

A positive negative exponential relationship was found between the two forms of concrete after fitting the data. The equation for this relationship is shown in Equation (4) and R^2 is 0.97. Therefore, the earlier the existing concrete was repaired in the project, the better the effect. The concrete's age was over 28 days, and the age of the repaired interface had little effect on its flexural strength.

$$f_f = 0.93\exp(-T/18.11) + 3.22 \quad (4)$$

where f_f is the flexural strength of the interface (MPa), and T represents the curing age of the existing concrete (d).

4. Analysis of Results

4.1. Degrees of Influence of Various Influencing Factors on New and Old Concrete

It can be concluded, based on the research presented above, that the flexural strength of the interfaces between the new and the old concrete was influenced by factors like the roughness, the type of interfacial agent, and the curing age. However, in actual repair projects, it might not be possible to consider these factors together for some reason. It may therefore be more helpful to guide related repair work if these factors can be quantified, as they affect the bending strength of the interface.

The difficulties above can be resolved very effectively through univariate variance analysis, based on previous findings [6,14,45]. With this method, F-values can be calculated for the interface between old and new concrete based on the impact of various factors on flexural strength. In Table 7, we present the calculation program and the results of the analysis of variance for the three influencing factors.

Table 7 clearly shows that the F values for the interfacial roughness, type of interfacial agent, and curing age of the new and old concrete were 49.37, 83.56, and 33.05, respectively. The data showed that the interfacial agent had the greatest impact on the flexural strength of the new and old concrete, followed by the interfacial roughness, and the existing concrete's curing age had the smallest impact. Therefore, it is recommended to use interfacial agents when repairing concrete in practical engineering projects.

4.2. Formula for Flexural Bearing Capacity of New and Old Concrete

Based on the analysis above, only a single factor was considered to influence the flexural strength of old and new concrete. In the actual repair work, the flexural strength of the interface between the new and the old concrete was often influenced by multiple factors. The comprehensive effects of the interfacial roughness, interfacial agents, and differences in curing ages were not considered in this experiment. In the process of repairing existing concrete, the flexural strength of new concrete can be determined through flexural tests. However, it is difficult to determine the flexural strength of old and new concrete interfaces, which requires the theory to be extended. The idea of establishing the equation while taking multiple factors into account is similar to those of Ding [46] and Huang [14]. Therefore, based on the flexural strength of the new concrete and considering three influencing factors, a formula for calculating the flexural strength of the interface between the new and the old concrete was derived.

$$f_f = K\alpha_1\alpha_2\alpha_3f_c \quad (5)$$

In the equation, f_f is the flexural strength of the interface between the full lightweight ceramsite concrete and the ordinary concrete (MPa), K is the correction factor, α_1 is the influence coefficient of the interfacial roughness, α_2 is the influence coefficient of the interfacial agent, α_3 is the influence coefficient of the difference in curing age between the new and the old concrete, and f_c is the compressive strength (MPa) of full lightweight ceramsite concrete.

Table 7. Calculation formulas and results for each parameter.

Calculation Parameters	Calculation Formula	Interfacial Roughness	Interfacial Agent	Difference in Curing Age between New and Old Concrete
Within-group sum of squares	$SSE = \sum_{j=1}^m \left[\sum_{i=1}^{n_j} (x_{ij} - \bar{x}_j)^2 \right]$	0.0348	0.03	0.1028
Sum of squares of deviations between groups	$SSA = \sum_{j=1}^m n_j (\bar{x}_j - \bar{x})^2$	0.6519	1.0029	1.3617
Within-group mean square deviation	$MSE = \frac{SSE}{n-m}$	0.0044	0.003	0.0103
Mean-square deviation between groups	$MSA = \frac{SSA}{m-1}$	0.2173	0.2507	0.3404
F value	$F = \frac{MSA}{MSE}$	49.37	83.56	33.05
$F_{0.025}(m-1, n-m)$		14.54	8.84	8.84
$F_{0.05}(m-1, n-m)$		8.85	5.96	5.96
$F_{0.1}(m-1, n-m)$		5.25	3.92	3.92
Level of influence	According to the F-distribution table If $F > F_{0.025}$, this factor has an extremely significant impact on the flexural strength. If $F_{0.025} > F > F_{0.05}$, then this factor has a significant impact on the flexural strength. If $F_{0.05} > F > F_{0.1}$, then this factor has almost no effect on the flexural strength. If $F < F_{0.1}$, then this effect has no effect on the flexural strength.	Extremely significant impact	Extremely significant impact	Extremely significant impact

4.2.1. Influence Coefficient of Interface Roughness α_1

The 28-day-cured full lightweight ceramsite concrete had a compressive strength of 53.66 MPa, according to Table 5. It is possible to calculate the flexural strength of the interface between new and old concrete cured for 28 days with different interfacial-roughness values by using Equation (6).

$$\alpha_1 = \frac{-0.05h^2 + 0.3h + 2.86}{53.66} = -0.00093h^2 + 0.00559h + 0.0533 \quad (6)$$

where α_1 is the influence coefficient of interfacial roughness, and h is the interfacial roughness (mm).

4.2.2. Influence Coefficient of Interface Agent α_2

The values of the different interfacial agents in Table 6 were calculated by combining the data from groups J-0 to J-4. In the absence of interfacial agents, α_2 was taken as 0.0563. The α_2 was 0.0606, 0.0630, 0.0660, and 0.0704 when cement paste, silica fume, polymers, and epoxy resin were used as the interfacial agents.

4.2.3. Influence Coefficient of Difference in Curing Age between New and Old Concrete α_3

The same determination process for α_1 , α_3 was obtained by a simple calculation, as shown in Equation (7).

$$\alpha_3 = \frac{0.93\exp(-T/18.11) + 3.22}{53.66} = 0.0173\exp(-T/18.11) + 0.0600 \quad (7)$$

where α_3 is the influence coefficient of the difference in curing age between new and old concrete, and T is the difference in curing age (d) between new and old concrete.

4.2.4. Determination of the Formula for Calculating the Flexural Strength of the Interface between New and Old Concrete

Firstly, Equations (6) and (7) were substituted into Equation (5) to obtain Equation (8). Based on this analysis, the K value was 266.959 and the R^2 was 0.99. A final calculation was performed using Equation (9) to determine the flexural strength at the interface between the new and the old concrete.

$$f_f = K(-0.00093h^2 + 0.00559h + 0.0533)\alpha_2(0.0173\exp(-T/18.11) + 0.0600)f_c \quad (8)$$

$$f_f = 266.959(-0.00093h^2 + 0.00559h + 0.0533)\alpha_2(0.0173\exp(-T/18.11) + 0.0600)f_c \quad (9)$$

where f_f is the flexural strength of the interface between the full lightweight ceramsite concrete and the ordinary concrete (MPa), h is the interfacial roughness (mm), and T is the difference in curing age between the new and the old concrete (d). When the interfacial agent is not used, the influence coefficient of the interfacial agent is taken as 0.0563. When using interfacial agents such as cement paste, silica-fume cement paste, polymer interfacial agent, and epoxy-resin interfacial agent, the influence coefficients of the interfacial agents are 0.0606, 0.0630, 0.0660, and 0.0704, respectively. The f_c is the compressive strength (MPa) of the full lightweight ceramsite concrete.

4.3. Calibration of Formulas

In order to calibrate the formula for calculating the flexural capacity of old and new concrete, as shown in Table 8, we collected four sets of data on the flexural properties of old and new concrete and substituted them into Equation (9). Finally, the calibrated data were compared with the original data. The data were taken from [47]. As shown in Figure 9, the change trends of the test results and calibration results were consistent. This indicated that the calibration results were accurate. However, there was a certain

gap between the two flexural strength values. This may have been due to factors such as different aggregate particle sizes [48], different mix proportions [49] and different curing conditions for lightweight aggregate concrete [50]. In view of the results presented, the formula can be used in similar case studies.

Table 8. Calibration data for new and old concrete flexural capacity equations.

Group Number	Compressive Strength of Lightweight Aggregate Concrete (MPa)	Interfacial Roughness (mm)	Interfacial Agent	Curing Age (d)	Measured Value (MPa)	Calibration Value (MPa)
a	43.27	2	cement paste	28	1.03	2.71
b	53.70	1.5			1.07	2.96
c	53.70	2			1.52	3.36
d	63.50	2			2.01	3.97

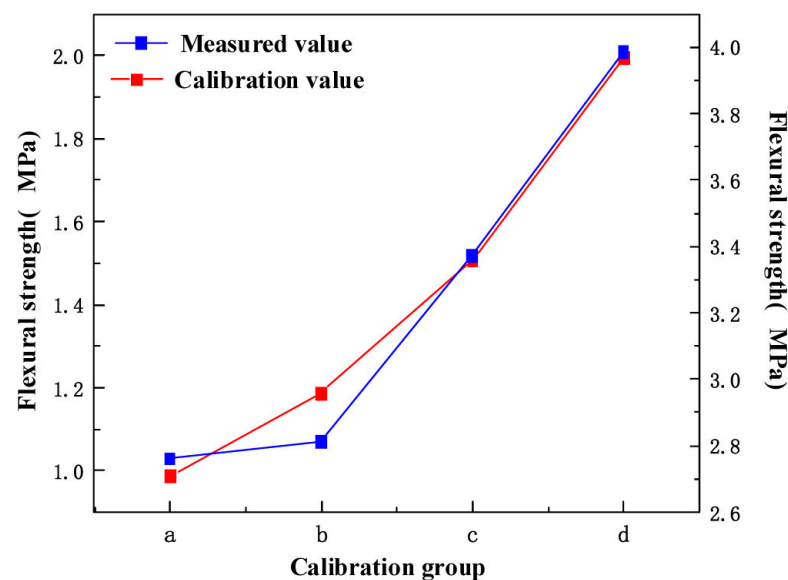


Figure 9. Calibration of flexural-strength-bearing-capacity equations for old and new concrete.

5. Conclusions

The following conclusions can be drawn from the above results and discussion.

- (1) The repair of C45 ordinary concrete with LC50 lightweight ceramsite concrete can make the flexural strength of the repaired structure reach about 75% of that of an existing concrete structure. Moreover, the composite structure can fulfil relevant specifications after repair.
- (2) As the interfacial roughness increased, the flexural strengths of the new and old concrete increased to varying degrees. When the roughness was 3 mm, the flexural strength reached its maximum value. However, if the treatment level of the interfacial roughness continued increase, this might have led to microcracks at the interface. This would have destroyed the integrity of the existing concrete and eventually led to a decrease in the flexural strength at the interface. Therefore, when the roughness of the new and old concrete interface was processed, the interfacial roughness should be reasonably controlled.
- (3) By comparing the repair effects of different interfacial agents on the flexural strength of the interface between the new and the old concrete, it was found that the coating interfacial agent effectively improved the flexural performance of the new-and-old-concrete interface. In particular, the epoxy-resin interfacial agent had the greatest effect on the flexural strength of the interface, restoring the flexural strength of the specimen to 77% before damage occurred.

- (4) The existing concrete's curing age and its interface's flexural strength exhibited a negative exponential relationship. When the curing age was less than 28 days, the flexural strength of the interface decreased rapidly. After 28 days of curing, the flexural strength decreased slowly with the increase in the age until it tended to become stable.
- (5) Through the single-variable analysis of variance, it was found that the order of influence of three factors on the flexural strength of the interface was interfacial agent > interfacial roughness > existing concrete's curing age. Therefore, in the actual repair project, the use of interfacial agents to repair the interface should be given priority.
- (6) The flexural strength of the interface between full lightweight ceramsite concrete and ordinary concrete was calculated using a formula. Three influencing factors were also taken into account in the formula. This study aimed to provide a scientific reference for the reinforcement of concrete projects with full lightweight ceramsite concrete.

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Article

Load Transfer Mechanism and Bond–Slip Behavior of Recycled Concrete-Encased Steel (RCES) Subjected to Cyclic Loading

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Abstract: RCES composite structures, which combine the advantages of high bearing- and good anti-seismic capacity of ordinary steel reinforced concrete (SRC) with the characteristics of energy-saving and environmental protection of recycled concrete, are beneficial to promote the use of recycled concrete. However, the bond–slip behavior and load transfer mechanism are essential issues of RCES composite structures. This paper primarily focused on the load transfer mechanism and bond–slip behavior of RCES composite structures subjected to cyclic loading. A total of 14 specimens, which were designed with different replacement ratios of recycled concrete, cover thicknesses of H-shaped steel, transverse reinforcement ratios, and recycled concrete strengths, were tested to investigate the load transfer mechanism and interface damage. The results indicate that the whole loading procedure can be divided into four phases and four limit points. The bonding shear damage of a specimen develops rapidly, and most damage happened during 0~0.2 mm slip. The bonding stress values of chemical bonding stress τ_{ca} , friction resistance stress τ_f , and mechanical biting stress τ_m were calculated. Moreover, in order to reflect the influence of cyclic loading, a degradation factor ξ was proposed. The mean values of chemical bonding stress τ_{ca} : friction resistance stress τ_f : mechanical biting stress τ_m are 1:0.187:0.696, in which the chemical bonding stress is the largest, friction resistance stress is the second, and the mechanical biting stress is the least.

Keywords: recycled concrete-encased steel (RCES); bond–slip mechanism; full state analysis; interface damage mechanism; cyclic loading

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1. Introduction

In recent years, the construction industry has seen a rapid development. However, the rapid development of construction consumes a lot of non-renewable natural resources such as sand and stone. On the other hand, the demolition of many old buildings generates an astonishing amount of construction waste due to the rapid urbanization [1]. These wastes have a significant impact on the environment and mostly contribute to landfill saturation. In order to solve this problem, it is necessary to recycle the construction waste and reduce environmental degradation, as well as the negative effects due to construction processes [2]. Recycled concrete, which is made by using recycled coarse aggregates (RCA) to replace natural aggregate partially or completely, is believed to be a good way to solve this problem [2]. Recently, many countries have attached great importance to environmental protection and are paying great attention to research on recycled concrete. A larger number of research studies, performed worldwide, have focused on the mechanical properties or recycled concrete [3–11]. Most of the studies show that although some mechanical properties of RCA are inferior to those of natural concrete, it is appropriate for recycled concrete to be used in structural engineering [12].

In order to popularize the application of recycled concrete, studies concerning the beams [13–16], columns [17,18], beam-column joints [12,19], and frames [20,21] were investigated. The research results of most previous studies show that the anti-seismic behavior and

failure mode of RCES structures are similar to those of ordinary reinforced concrete structures. However, the bearing capacity of RCES structures was reduced to some allowable extent. Nowadays, SRC structures have already been widely used in engineering construction due to their advantages, such as better seismic capacity and good ductility [22,23]. In this regard, RCES composite structures are proposed to broaden the application scope of recycled concrete [24]. It is believed that RCES composite structures, which combine the shape steel and recycled concrete, can achieve a better seismic performance, as the attractive features of SRC can make up for the shortcomings of recycled concrete. However, the bond-slip capacity is a key problem of RCES composite structures, which provide enough shear resistance at the interface and ensure that H-shaped steel and recycled concrete are working together [25,26]. Zhang et al. (2019) [27] investigated the flexural capacity of composite beams and found that about 27–35% deflection underestimation happened if the interlayer slip was neglected. Hotta et al. (1977) [28] studied the influence of the lateral restraint stress of concrete on the bond-slip behaviors between steel and concrete. Zheng et al. (2016) [29] studied the bond mechanisms and failure modes of SRRC composite structures. Bai et al. (2016) [30] studied the bond-slip capacity of RCES composite structures by using the orthogonal method and pull-out test. The results of the above investigation showed that the bond-slip failure process of RCES composite structures were similar to that of ordinary steel-reinforced concrete, both of which undergo four stress phases: no slip phase, slip phase, failure phase, and residual phase. The concrete cover thickness of H-shaped steel and recycled concrete strength have positive effects on the average bonding strength, while the replacement ratio of recycled coarse aggregates lead to an allowable decrease in average bonding strength. Chen et al. (2013) [31] designed 22 SRRC test specimens and carried out pull-out tests, in which they found that the ultimate bond stress and residual bond stress increased with the compressive recycled concrete strength. The bond stress of different parts of H-shaped steel is different; the web adhesion stress is the smallest, followed by the outer flange, and the inner flange adhesion stress is the largest. The bonding force at the interface of an SRRC specimen is exponentially distributed along the longitudinal direction of H-shaped steel. Chen et al. (2013) [32] tested 11 SRRC specimens with different replacement ratios of recycled coarse aggregates and found that the interface energy dissipation capacity of SRRC specimens is not significantly affected by the replacement ratio of recycled coarse aggregates when the replacement ratio is less than 70%. However, when the replacement ratio is greater than 70%, the energy dissipation coefficient increases with the replacement ratio. The shear stiffness of the interfacial bond decreases with the increase in the replacement ratio. Finally, a τ - s constitutive model was proposed. Most of the previous research, especially for the studies mentioned above, is focused on the investigation of bond strength and stress. There is little research on the degradation of bond capacity and no research on the full-stage analysis of load transfer mechanism at the interface between recycled concrete and steel. Studies generally indicate that the bond stress at the interface between concrete and H-shaped steel is composed of chemical bond stress, mechanical bite stress, and friction stress, but there are no quantitative calculation methods for the calculation of each part. Consequently, a series of RCES specimens were tested to investigate the load transfer mechanism, degradation of bond capacity, and to calculate the components of bond stress.

2. Test program

2.1. General

Materials used in this experiment include: (1) cement: ordinary Portland cement; (2) natural coarse aggregate (NCA): natural stones which were screened with continuous gradation from 0 to 25 mm; (3) recycled coarse aggregate (RCA): RCA comes from a recycled building material company; (4) fine coarse aggregate: the common river sand with a fineness modulus of 2.75; (5) water: city tap water; (6) water reducing agent: polycarboxylic type high performance water reducer; (7) channel steel: No. 10 U-shaped channel steel; (8) steel bar: $\Phi 6$ smooth bars were selected as the transverse reinforcement

(Φ means hot rolled plain steel bars with strength grade 300 MPa), Φ 12 ribbed steel bars were selected as the longitudinal reinforcement (Φ means hot rolled ribbed steel bars with strength grade 335 MPa); and (9) steel plate: Q235 steel plate with thickness of 6 mm. The mechanical properties of shape steel and steel bars were tested according to the Chinese standard GB/T228, and the results are shown in Table 1.

In order to ensure the quality of recycled concrete and investigate the influence of coarse aggregate on the mechanical properties of recycled concrete, a series of material property tests have been performed on NCA and RCA. The test results show that for the NCA and RCA used in the experiment, bulk density (kg/m^3) is 1580:1330, apparent density (kg/m^3) is 2800:2570, 24(h) water absorption (%) is 0.1:4.8, crush index (%) is 5.0:13.0, porosity (%) is 44:48, and mud content (%) is 0.4:0.6. It should be noted that both the bulk and apparent density of NCA is bigger than that of RCA, which means the bulk of recycled aggregates have more voids. The 24(h)-water absorption of RCA is 48 times that of NCA. Moreover, the crush index of RCA is higher than NCA which means the strength of RCA is lower than NCA.

Table 1. Mechanical property of shape steel and steel bars.

Shape Steel and Steel Bars		Yield Strength f_y/MPa	Yield Strain $\epsilon_y/10^{-6}$	Ultimate Strength f_u/MPa	Elastic Modulus E_s/MPa
Φ 6 smooth bars		445	1878	490	2.12×10^5
Φ 12 longitudinal bars		505	1927	715	2.09×10^5
Shape steel	Steel flange	359	1588	469	2.05×10^5
	Steel web	332	1450	429	2.13×10^5
	Steel plate	340	1201	445	2.15×10^5

2.2. Specimen Design

Fourteen RCES specimens were conducted, including 13 RCES cyclic loading specimens and 1 RCES pull-out loading contrast specimen. Six $150 \text{ mm} \times 150 \text{ mm} \times 150 \text{ mm}$ concrete test cubs were cast and tested after 28 days of standard curing. Demolding of both RCES specimens and concrete cub was carried out three days later after casting. Figure 1 shows the cross-section and elevation dimensions of typical RCES specimen.

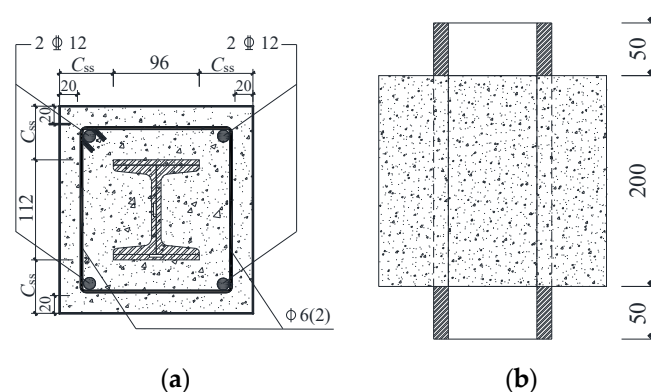


Figure 1. Design dimensions of typical RCES specimen, they should be listed as: (a) Cross-section of typical RCES specimen; (b) elevation of typical RCES specimen.

Table 2 shows the parameters investigated and details the mix properties of recycled concrete used in this paper. The effect of shrinkage was omitted due to the short-term test. The effect of steel surface treatment on adhesion was not studied in this paper and the steel used in this test was obtained directly from the factory. Four design parameters were mainly considered to investigate the bond capacity of RCES specimens as follows:

- Replacement ratio of recycled concrete (r): The chemical bonding force is an important part of the bond capacity at the interface. The magnitude of chemical bonding force depends on the amount and quality of cement colloid formed in the process of concrete mixing, curing, and finally coagulation. However, r has a great impact on the formation of cement colloid. Thus, specimens RCES-2, RCES-3, RCES-4, and RCES-5 with different r of 0%, 30%, 70%, and 100% were investigated. The other parameters remain constant.
- Thickness of recycled concrete cover (C_{ss}): The cover thickness provides much constraint to the interface and makes the slip difficult to happen. Specimens RCES-2, RCES-6, RCES-7, and RCES-8 with different concrete cover 50, 60, 70, and 80 mm were tested to investigate the relationship between bond capacity and restrain effect.
- Transvers reinforcement ratio (ρ_{sv}): The transvers reinforcement ratio also can provide constraint to the interface and improve the bond capacity to some extent. Specimens RCES-2, RCES-9, RCES-10, and RCES-11 were designed with different reinforcement ratios to study how the reinforcement improved the bond stress transferred at the interface.
- Compressive recycled concrete strength (f_c): The same as r , different concrete strengths require different mix proportions; different mix proportions influence the formation of cement colloid; which as a result affects the chemical bond stress. Therefore, some other factors, such as aggregate size, the type of cement, the additives and admixtures were omitted as the compressive strength was considered. In this test, specimens RCES-2, RCES-12, RCES-13, and RCES-14 with different compressive strengths of 31.90 MPa, 40.32 MPa, 45.44 MPa, and 50.05 MPa, respectively, were studied.

Table 2. Mix property and design details of test specimen.

Specimen	W/C	S_p (%)	Material Weight (kg·m ⁻³)							r (%)	C_{ss} (mm)	ρ_{sv} (%)	f_c (MPa)	f_{cu} (MPa)
			Cement	W	RCA	NCA	S	Added Water	Water Reducer					
RCES-01	0.45	36	406	183	1158	0	651	55.6	4.07	100	60	0.77	C40	40.32
RCES-02	0.45	36	406	183	1158	0	651	55.6	4.07	100	60	0.77	C40	40.32
RCES-03	0.45	36	406	183	0	1158	651	—	4.07	0	60	0.77	C40	42.96
RCES-04	0.45	36	406	183	347.4	810.6	651	16.68	4.07	30	60	0.77	C40	41.44
RCES-05	0.45	36	406	183	810.6	347.4	651	38.92	4.07	70	60	0.77	C40	41.06
RCES-06	0.45	36	406	183	1158	0	651	55.6	4.07	100	50	0.77	C40	40.32
RCES-07	0.45	36	406	183	1158	0	651	55.6	4.07	100	70	0.77	C40	40.32
RCES-08	0.45	36	406	183	1158	0	651	55.6	4.07	100	80	0.77	C40	40.32
RCES-09	0.45	36	406	183	1158	0	651	55.6	4.07	100	60	0.62	C40	40.32
RCES-10	0.45	36	406	183	1158	0	651	55.6	4.07	100	60	1.23	C40	40.32
RCES-11	0.45	36	406	183	1158	0	651	55.6	4.07	100	60	1.54	C40	40.32
RCES-12	0.47	37	389	183	1151	0	676	55.3	3.90	100	60	0.77	C30	31.90
RCES-13	0.40	36	450	180	1132	0	637	54.4	4.50	100	60	0.77	C45	45.44
RCES-14	0.35	35	505	177	1116	0	601	53.6	5.06	100	60	0.77	C50	50.05

Note: W represent water; C represent Portland cement; S_p represent sand ratio; r represents replacement ratio of recycled concrete; C_{ss} represent cover thickness of H-shaped steel; ρ_{sv} represent transvers reinforcement ratio; f_c represent the design compressive strength of recycled concrete.

2.3. Loading Procedure and Instrumentation

This test was conducted at the key laboratory of structural engineering and earthquake resistance in Xi'an University of Architecture and Technology. The test setup and a typical RCES specimen were illustrated in Figure 2. Four linear variable displacement transducers (LVDTs) were placed vertically on the concrete splint at the end of recycled concrete (two at the top and two at the bottom) to measure the relative bond-slip along the vertical direction, as the slip was distributed unevenly along the longitudinal direction. In order to measure the inner slip of RCES specimens, two adhesive slip sensors were placed on the one-third place outside of the steel flange. The load machine, LVDTs, and adhesive slip sensors were calibrated before used. The loading device is mainly composed by eight parts: (1) fixture; (2) high-strength screw; (3) adhesive slip sensor; (4) splint; (5) high-strength bolt; (6) LVDT; (7) recycled concrete; and (8) H-shaped steel. The top of RCES specimen is defined as the loading end, while the bottom of the RCES specimen where H-shaped

steel is fixed by the fixture and cannot move is defined as the fixed end, as presented in Figure 2. The H-shaped steel and recycled concrete were fixed by the bottom fixture and loading steel frame separately. As a result, the shear load at the interfacial transferred from concrete to steel through the bond effect.

The displacement control loading procedure was used in this test, as shown in Figure 3. The cyclic loading was loaded at a rate of 0.1 mm/s. Each level of displacement was loaded for three cycles to study the interface damage degradation. An amount of 1~2 kN was loaded at the beginning to keep all parts in close contact. Afterwards, the initial load was removed, and the loading was begun. The loading was terminated once 10 mm of relative slip was recorded at the fixed end.

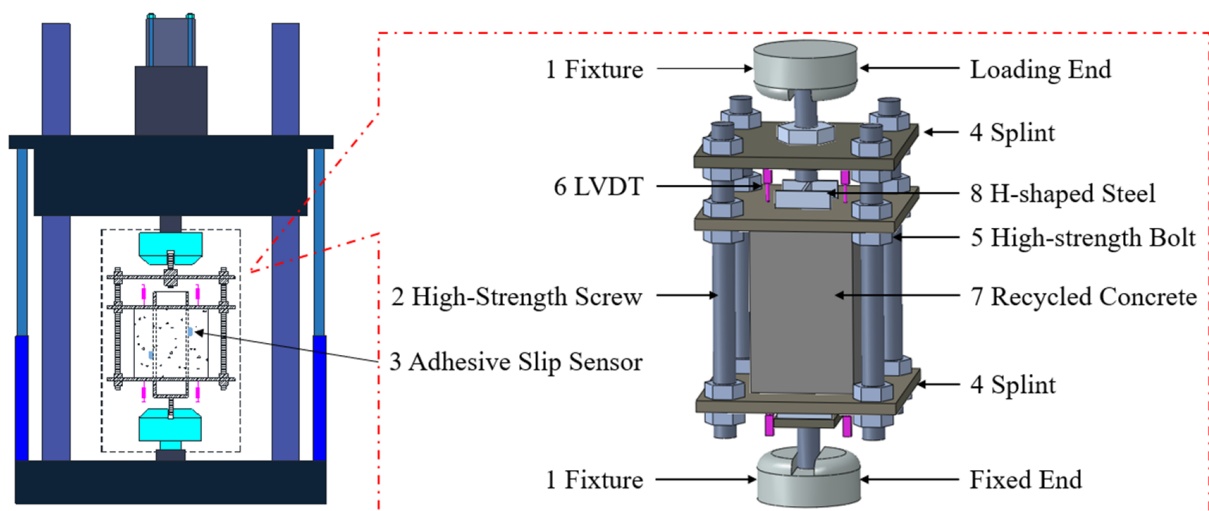


Figure 2. Test setup and loading specimen.

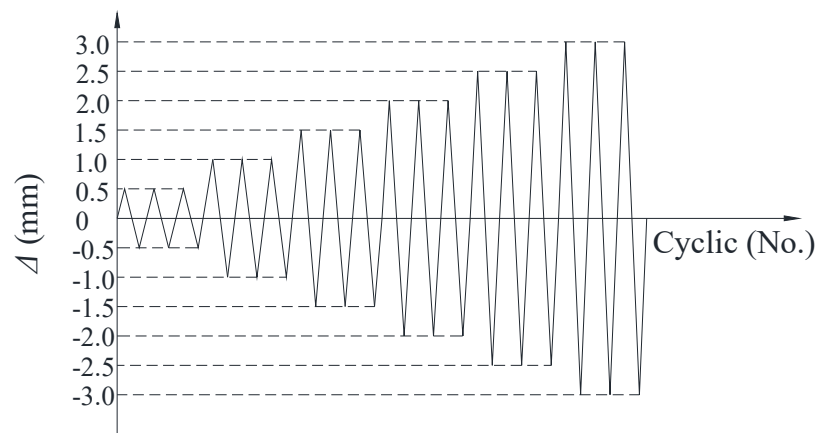


Figure 3. Loading procedure.

The distribution of strain gauges and adhesive slip sensors is presented in Figure 4. The measuring points are arranged by attaching strain gauges in the slot to prevent the strain gauges from being damaged during the loading process. Therefore, a composite H-shaped steel which is made up of two No. 10 channel steels and two 6 mm steel plates was used in this test.

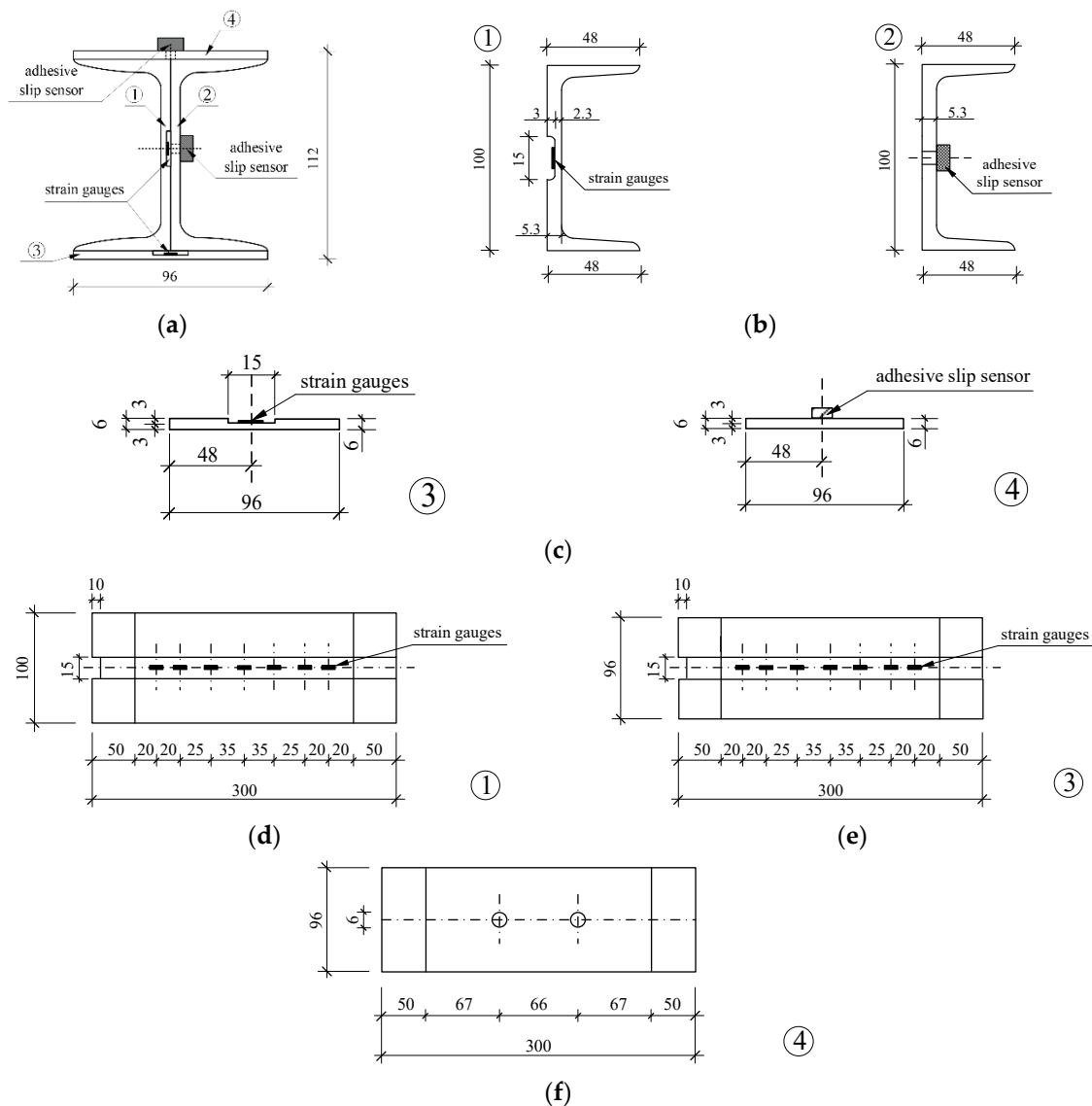


Figure 4. Measure arrangement of RCES specimen, they should be listed as: (a) Cross-section of H-shaped steel (mm); (b) Cross-section of channel steel (mm); (c) Cross-section of steel plate (mm); (d) Layout of strain gauges on web of channel steel (mm); (e) Layout of strain gauges on steel plate (mm); (f) Layout of adhesive sensors on steel plate (mm).

3. Results and Analysis

3.1. The Whole Loading Process

At the beginning of loading process, there was little damage at the bonding interface, the slip at both the top and bottom surface of specimen was very small, and the load increased dramatically. In this state, the transferred shear force at interface was mainly carried by chemical adhesive force and static friction resistance. However, the chemical adhesive force plays a major role. When the cyclic loading increased to 30~40% of ultimate load, the slip at the load end and fixed end gradually increased, specimen RCES-06, RCES-09, RCES-10, and RCES-12 showed obviously slip. As the cyclic load increased gradually, the other specimens begin to slip one after another. In contrast, RCES-03 and RCES-14 begin to slip later. This is because RCES-03 has a lower r and RCES-14 has a higher recycled concrete strength, both lower r and higher f_c affects the formulation of chemical adhesive stress and make it more difficult to slip at the interface. The slip increases with the increase in cyclic loading. At this stage, most H-shaped steel flanges showed concrete powder that

was ground due to the sliding at the interface as presented in Figure 5b,d,e. The concrete covers of a few specimens were crushed and peeled off, as shown in Figure 5h. As the bond damage increased, the cyclic loading began slow down. The chemical bonding stress was completely damaged until the cyclic loading reached to the peak value. Therefore, the shear force at interface was carried by the frictional resistance and mechanical bite stress. Then, the cyclic load decreased rapidly and the slip increased correspondingly. In the end, the cyclic load remained the same, and the bond interface was completely destroyed. As the anchorage length of H-shaped steel is small, no bonding cracks occurred in this test. Most specimens generally failed due to the pull of the steel out of the concrete. Some typical failure modes which happened during the loading process are shown in Figure 5.

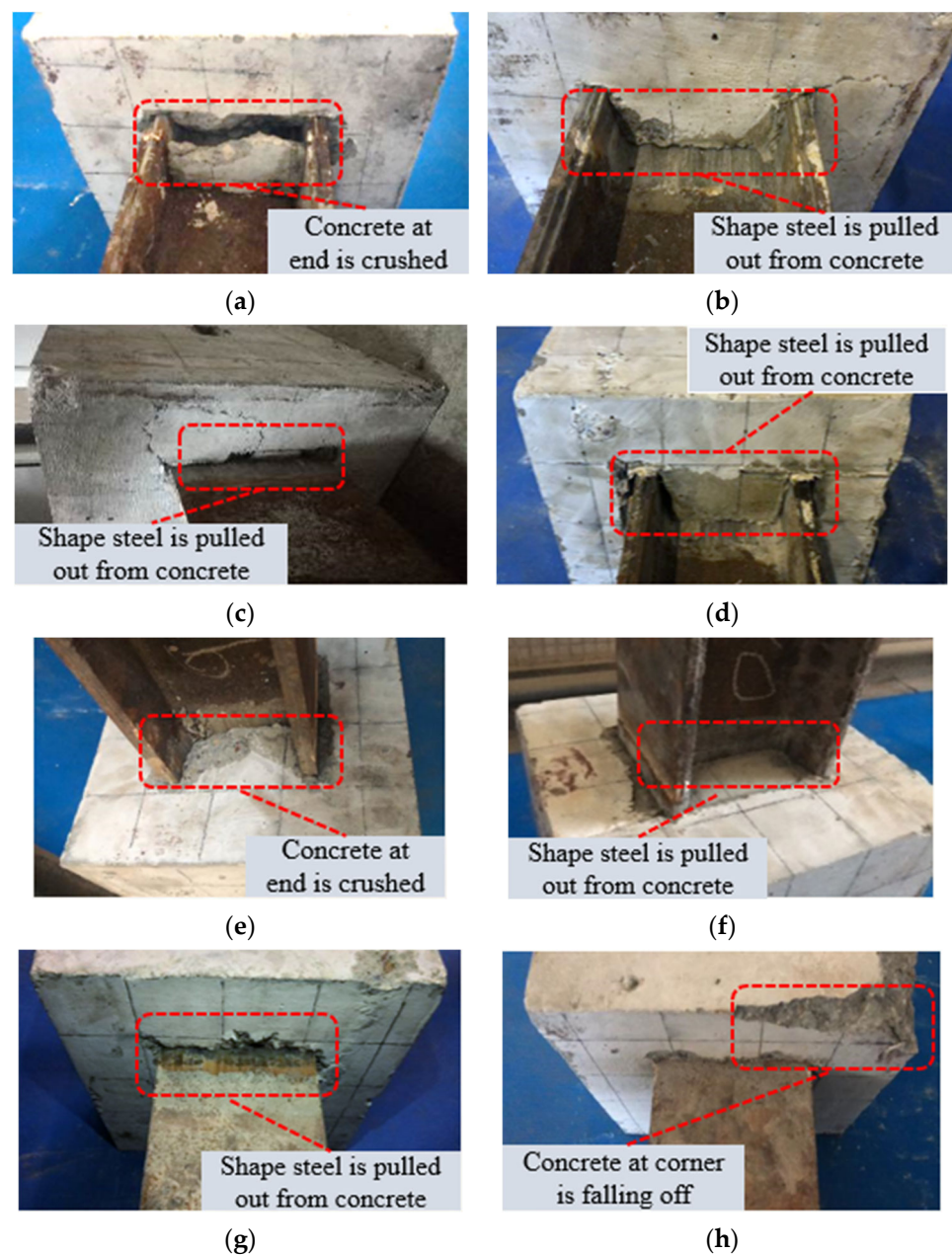


Figure 5. Typical failure modes of specimen, they should be listed as: (a) RCES-02; (b) RCES-03; (c) RCES-05; (d) RCES-08; (e) RCES-09; (f) RCES-10; (g) RCES-12; (h) RCES-13.

3.2. Analysis of Loading Procedure

The whole failure process of RCES specimens can be divided into two types by analyzing the full bond–slip skeleton curve, as shown in Figure 6. Four characteristic points were defined according to the variation of bonding force.

Micro slip point (A): The characteristic point A was defined using the geometric method as shown in Figure 7. Initially, a tangent line was made at point O which intersects the horizontal line BE at point E, then a vertical line was made through the E point that intersects the curve at point A, which is the micro slip point of the bond–slip curve. The bond–slip curve shows a linear relationship before point A [33]. The chemical bond stress reaches the ultimate value at point A.

Ultimate point (B): The peak point which the bond–slip curve reaches. As the cyclic loading increased gradually, the chemical adhesive stress was damaged completely. The transferred shear force at the interface is carried by the frictional resistance and mechanical bite stress.

Residual point (C): A line was drawn connecting point B and D. The point where the slope of tangent line in the bond–slip curve is equal to the slope of line BD is defined as the residual point C, as shown in Figure 7.

Failure point (D): The point reaches to 10 mm relative slip is defined as point D.

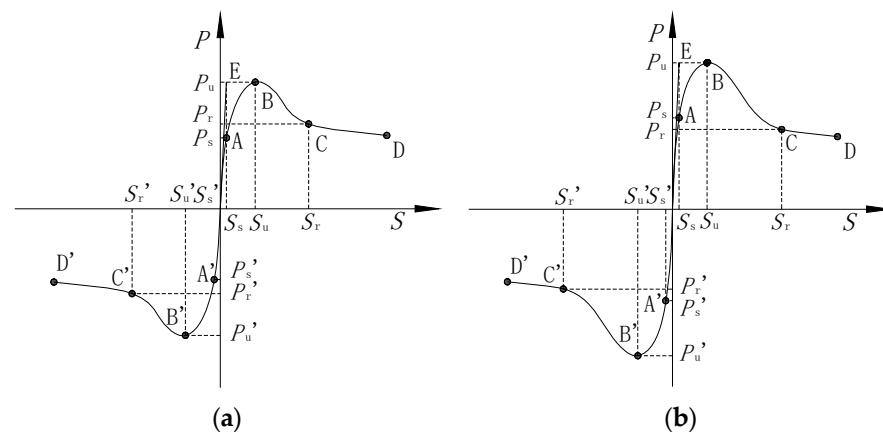


Figure 6. Failure process of bond–slip skeleton curve, they should be listed as: (a) $P_s < P_r$; (b) $P_s > P_r$.

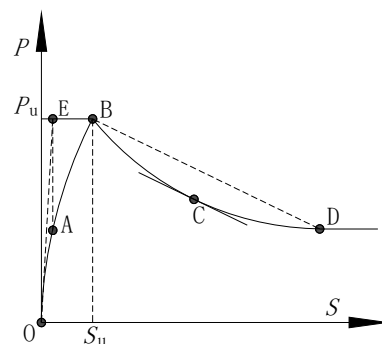


Figure 7. Method to defined characteristic point.

Correspondingly, there are four stages of the whole bond–slip curve: (1) micro slip phase (O–A); (2) slip development phase (A–B); (3) slip increases rapidly phase (B–C); and (4) residual phase (C–D).

- (1) Micro slip phase (O–A): The cyclic load increases linearly with the slip from 0. As shown in Figure 8a, the bonding stress at the interface increases gradually with the cyclic load. However, the cyclic load applied in this phase is relatively small; the transferred shear force at the interface is mainly carried by the chemical bonding stress. As the cyclic load continues increasing, the bonding stress at a certain point of the interface increases to the limit value of chemical bonding stress. At this time, the specimen is in a critical slip state, as shown in Figure 8b. With the increase in cyclic load, the cement colloid was damaged and obvious slip happened at the interface.
- (2) Slip development phase (A–B): In this phase, the bond–slip curve changes from a straight line to nonlinear line. Moreover, part of cement colloid was sheared and more damaged happened as shown in Figure 8c. The bonding slip zone diffused from fixed and loading end to the middle. The cement colloid damaged increased gradually so as to the slip zone as shown in Figure 8d. The chemical adhesive stress decreases gradually, while the mechanical bite stress and frictional resistance stress increase.
- (3) Slip increases rapidly phase (B–C): As shown in Figure 8e, the cement colloid on the interface will be completely damaged after the cyclic load reached to P_u . With the increase in cyclic loading, the chemical adhesive stress was lost, and the axial load dropped steeply.
- (4) Residual phase (C–D): In this phase, a concrete failure layer was formed as shown in Figure 8f. As the slip increased, the failure layer gradually grinds smooth. The mechanical bite stress gradually decreases and disappears, while the transferred shear force at the interface is carried by friction resistance stress.

Based on the analysis above, the bond stress, slippage and values of characteristic points were obtained and summarized in Table 3.

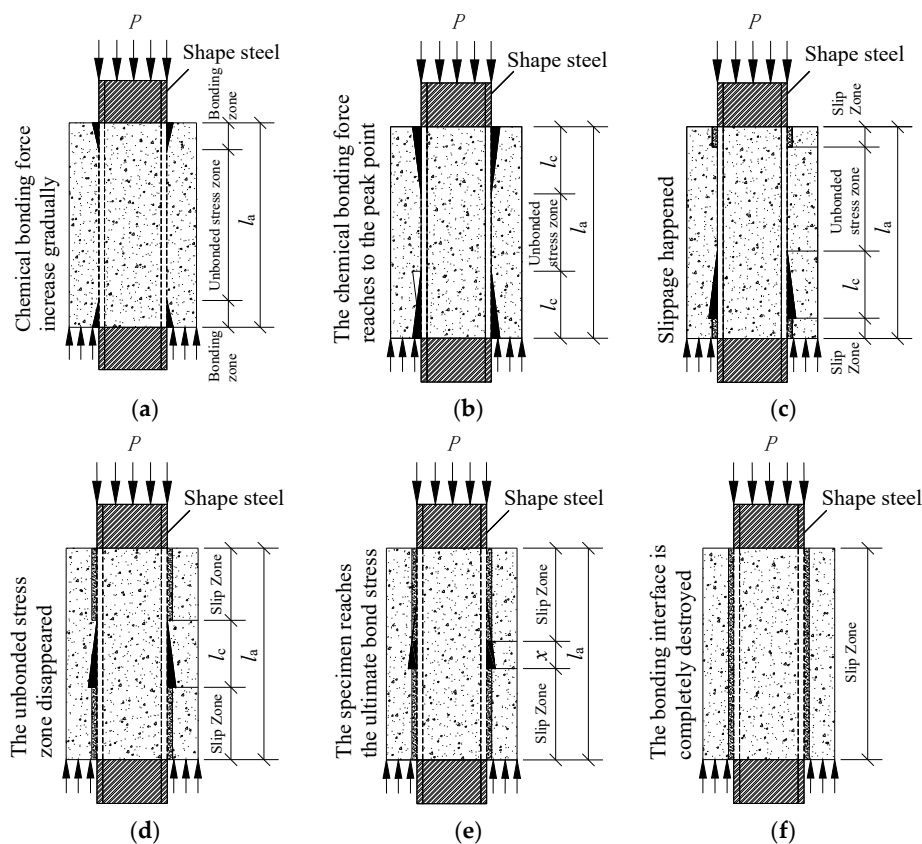


Figure 8. Failure process of specimen, they should be listed as: (a) Bonding stress increase; (b) slip critical state; (c) slippage happened; (d) the unbonded stress zone disappeared; (e) ultimate bonding state; (f) residual state.

Table 3. Values of characteristic bond strength.

Specimen	P_s (kN)	τ_s (MPa)	S_s (mm)	P_u (kN)	τ_u (MPa)	S_u (mm)	P_r (kN)	τ_r (MPa)	S_r (mm)
RCES-01	141.90	1.29	0.074	150.70	1.37	0.521	71.50	0.65	5.131
RCES-02	35.20	0.32	0.018	89.10	0.81	0.483	36.62	0.33	4.144
RCES-03	46.20	0.42	0.024	103.40	0.94	0.369	42.90	0.39	3.760
RCES-04	44.00	0.40	0.023	96.80	0.88	0.406	39.69	0.36	4.036
RCES-05	39.60	0.36	0.021	92.40	0.84	0.435	37.45	0.34	4.763
RCES-06	30.80	0.28	0.016	81.40	0.74	0.509	34.13	0.31	3.825
RCES-07	36.30	0.33	0.019	93.50	0.85	0.464	40.47	0.37	4.964
RCES-08	38.50	0.35	0.019	94.60	0.86	0.413	40.82	0.37	4.841
RCES-09	30.80	0.28	0.016	89.10	0.81	0.448	34.48	0.31	3.985
RCES-10	40.70	0.37	0.021	89.10	0.81	0.399	38.51	0.35	4.375
RCES-11	47.30	0.43	0.025	90.20	0.82	0.382	41.80	0.38	4.406
RCES-12	29.70	0.27	0.016	73.70	0.67	0.393	33.02	0.30	4.830
RCES-13	37.40	0.34	0.020	94.60	0.86	0.446	40.71	0.37	4.295
RCES-14	44.00	0.40	0.023	97.90	0.89	0.481	41.80	0.38	4.356

3.3. Interface Damage Analysis

In order to quantitatively describe and analyze the damage degree of bonding interface between concrete and steel, the secant stiffness K_i was used to define the damage factor D . The damage factor D can be calculated as Equation (1).

$$D = (K - K_i) / K \quad (1)$$

where: K is the initial bonding stiffness of specimen; D is the damage factor of specimen, $D = 0$ means there is no damage happened at the bonding interface; $D = 1$ indicates that the bonding interface is destroyed. The larger the value of D , the more serious the damage the bonding interface.

Figure 9 shows a relationship curve between damage factor and the slip value of a typical specimen. The whole bond–slip damage process can be divided into three phases according to the D – S curve. (1) Linear rapid damage phase (0~0.6): in this phase, the damage of specimen develops rapidly. D – S curve presenting a vertical line, D increases linearly with S . (2) Damage accumulation transition phase (0.6~0.9): damage factor D changes from linear to nonlinear and the development rate of damage slows down gradually. In this phase, the damage of the interface is serious, and the development of damage is basically complete. Usually, some bonding and splitting cracks will occur in this phase. (3) Damage stabilization phase (0.9~1.0): D – S curve shows a horizontal line parallel to x axis. Damage factor D does not change, and no new damage appeared.

Figure 10 shows the influence of different factors to D – S curves. (1) Replacement ratio of recycled concrete: r has a slight effect on the damage development of bonding interface as shown in Figure 10a. The D – S curves with different r will be different at the damage accumulation transition phase. This is because in this phase, most of the cement colloid on the bonding interface is broken, and at the same time, the recycled concrete appears to have bonding cracks, bonding splitting damage. (2) Cover thickness of H-shaped steel: at the initial damage stage, the D – S curves with different C_{ss} are basically the same. (3) Transvers reinforcement ratio: the transvers reinforcement ratio has little influence on the damage development of the bonding interface. At the initial loading stage, stirrups basically have little effect on the inhibition of damage. After the recycled concrete was cracked and damage, the stirrups provided lateral binding force to the recycled concrete and inhibited their mutual sliding. Stirrups enhanced the friction stress and mechanical bite stress, but had little influence on the damage development. (4) Strength of recycled concrete: the f_c has a great effect on the damage development of the bonding interface. Specimens with higher f_c had damage appear later. At the same damage degree, the specimen with higher strength had a small slip.

This is mainly because the higher the f_c is, the denser the concrete inside, the closer the bond between steel and concrete, and the interface is more difficult to damage.

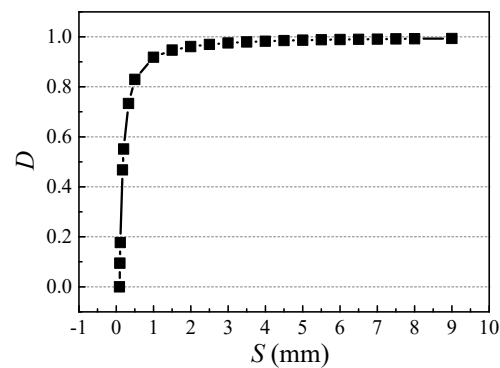


Figure 9. Damage factor vs. slip value curve.

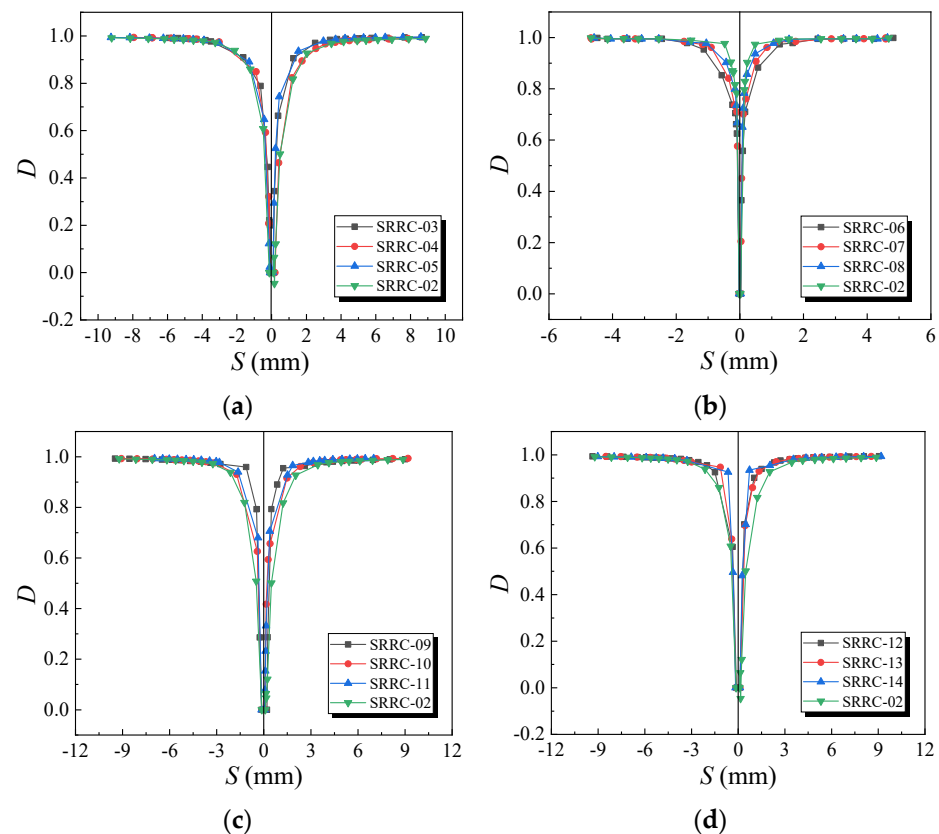


Figure 10. Influence of different factors on interface damage curves, they should be listed as: (a) Influence of r ; (b) Influence of C_{ss} ; (c) Influence of ρ_{sv} ; (d) Influence of f_c .

4. Composition and Solution of Bonding Stress

4.1. Distribution of Bonding Stress

The bond between steel and recycled concrete is composed of chemical bonding stress τ_{ca} , friction resistance stress τ_f and mechanical biting stress τ_m . The chemical bonding stress comes from the chemical adsorption on the bonding interface of cement colloid in recycled concrete after water absorption and hardening, which mainly exists in the area where there is no relative sliding on the bonding interface. When the bonding interface is relatively sliding, the cement colloid is sheared, and the chemical bonding force is greatly reduced. The friction resistance stress is the friction resistance caused by the non-smooth surface after the sliding. The frictional resistance is mainly determined by the normal stress

and friction coefficient on the bonding interface, and is mainly related to the transverse constraints such as the cover thickness of H-shaped steel, transverse reinforcement ratio, and the smoothness of the steel surface. The mechanical biting stress is mainly caused by the relative sliding of the bonding interface, and a layered concrete debris is generated after the cement colloid is sheared. The furrow effect occurs when relative slide happens. The mechanical biting stress depends mainly on the transverse restraint and the smoothness of the steel surface.

During the loading process, the bonding stress at the interface is not evenly distributed along the embedded length of the specimen. In order to analyze the stress transfer process of the bonding interface in detail, the embedded length l_a is divided into the slip zone and non-slip zone, as shown in Figure 11. The slip zone is the A region where the slip has already occurred, and the length is l_s . The non-slip zone is the region where no slip occurs, including the bonding diffusion zone B (the length of the bonding diffusion zone is defined as l_c) and the non-bonding stress zone C. Bonding diffusion zone B refers to a bonding diffusion length l_c which always exists in the non-slip zone during the process of shear transfer at the bonding interface. The bonding diffusion zone is located between the slip zone A and the no bonding stress zone C. Zone B moves from both ends of specimen to the middle with the repeated application of load until the bonding interface is completely destroyed.

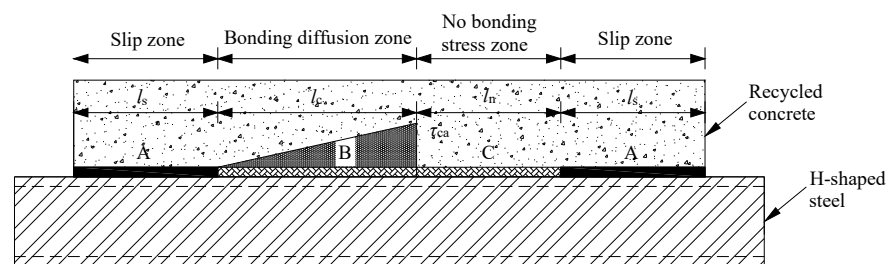


Figure 11. Division of the bonding interface.

4.2. Calculation of Bonding Stress

Cyclic loading has a great influence on the bonding stress at the interface. In the process of cyclic loading, continuous loading and unloading causes damage to the bonding interface, resulting in constant changes in the state of bonding interface, which affect the calculation of interface bonding stress. According to the test results of cyclic loading, the bonding strength of the specimen before reaching the peak value has a small degradation and basically presents a straight line, that is, the cyclic loading has a small effect on the bonding stress before peak load. Therefore, it can be approximated that cyclic loading has no effect on the chemical bonding stress τ_{ca} , friction resistance stress τ_f , and mechanical biting stress τ_m before peak loading. However, after peak load, especially in the region between the peak point and residual point, the bonding strength of the specimen degrades greatly, and the cyclic load has a great influence on the bonding stress of the interface. In this phase, the contact surface becomes smooth gradually due to repeated grinding, and the restraint effect of the lateral recycled concrete and transverse stirrups gradually weakens, which leads to the decrease in friction resistance stress τ_f and mechanical biting stress τ_m . Therefore, it can be considered that the mechanical biting stress and friction resistance stress of the bonding interface are weakened by cyclic loading, while the chemical bonding stress is not affected. In order to reflect the influence of cyclic loading on bonding stress, a degradation factor of bonding stress ξ was proposed. ξ can be calculated by Equation (2). Then the bonding stress composition of different characteristic loading points have the relationships shown in Equations (3) and (4).

$$\zeta = \frac{\tau_u - \tau_r}{\tau_u} \quad (2)$$

$$\tau_{ca} = \tau_{ca}^s = \tau_{ca}^u \quad (3)$$

$$(\tau_f + \tau_m) = (\tau_f^s + \tau_m^s) = (\tau_f^u + \tau_m^u) = \zeta(\tau_f^r + \tau_m^r) \quad (4)$$

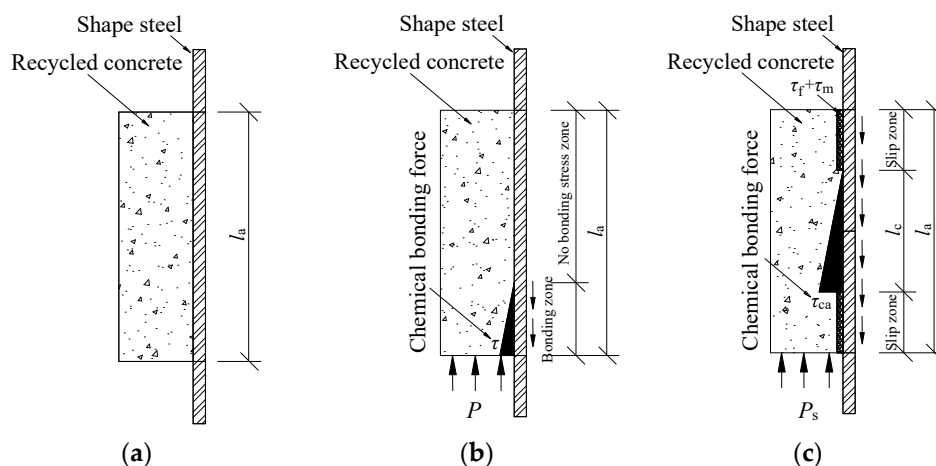
where: τ_{ca}^s is the chemical bonding stress in micro slip phase; τ_{ca}^u is the chemical bonding stress at peak load; $(\tau_f^s + \tau_m^s)$ is the friction resistance stress and mechanical biting stress in micro slip phase; $(\tau_f^u + \tau_m^u)$ is the friction resistance stress and mechanical biting stress at peak load; and $(\tau_f^r + \tau_m^r)$ is the friction resistance stress and mechanical biting stress in residual phase.

Based on the analysis of Section 3.2, the load–slip skeleton curve can be divided into four different characteristic loading points and four loading phases. Correspondingly, there are five critical states in the whole loading process: (a) No damage stage; (b) A state; (c) B state; (d) C state; and (e) D state, as shown in Figure 12.

At the origin point O, the load was just started to load, and no shear force is transferred on the interface. There is no damage occurred in the contact surface and the bond stress distribution is shown in Figure 12a. with the axial load increase, the chemical bonding stress τ_{ca} and axial load balance each other appear in the bonding diffusion zone at the ends of specimen as shown in Figure 12b. With the application of cyclic loading, the bonding diffusion zone moves from both ends to the middle. The axial load increase linearly with the increase in slip zone. When the axial load increases to point A, the bonding diffusion zone has already moved to the middle of the specimen, and it is about to slide. The bonding stress distribution of the interface is shown in Figure 12c. At this critical point, the shear force transferred by the bonding interface is borne by the chemical bonding stress τ_{ca} in the bonding diffusion zone, friction resistance stress τ_f , and mechanical biting stress τ_m in slip zone. Based on the balance condition of force, the bonding stress at the interface can be calculated by Equation (5).

$$P_s = (\tau_f + \tau_m) \cdot (l_a - l_c) \cdot C_a + \int_0^{l_c} \frac{\tau_{ca}}{l_c} \tau_{ca} d\tau \quad (5)$$

where: l_a is the embedded length of H-shaped steel; l_c is the length of bonding diffusion zone; and C_a is the perimeter of steel cross-section.



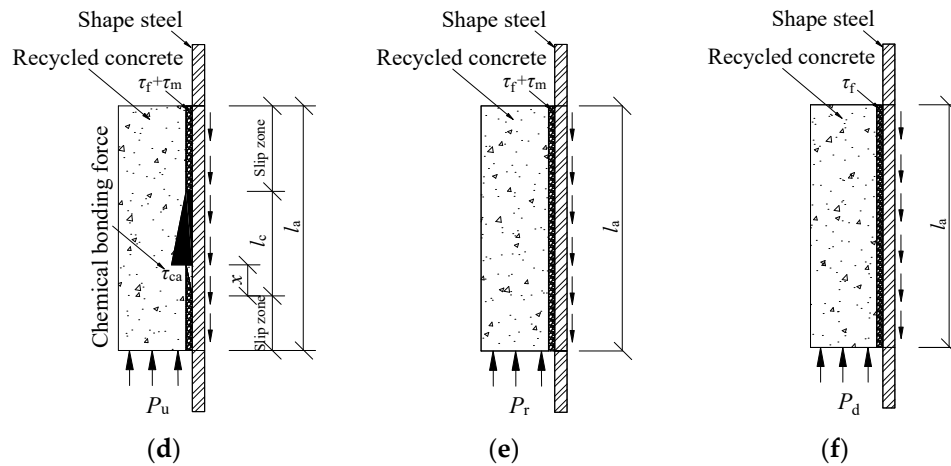


Figure 12. Diagram of specimen under different critical states, they should be listed as: (a) No damage stage; (b) Stress development state; (c) A state; (d) B state; (e) C state; (f) D state.

With the axial load increase, slip occurred in the bonding diffusion zone and the load–slip curve showed nonlinear growth. The increase rate of the axial load with the increase in slip zone slowed down. When the axial load increases to point B, part of the bonding diffusion zone has already slipped, the chemical bonding stress is reduced, while the friction resistance stress and mechanical biting stress are correspondingly increased. The area enclosed by the bonding stress curve and the embedded length of interface is the largest when the combined force of friction resistance stress and mechanical biting stress is equal to that of chemical bonding stress. At this time, the specimen reaches the peak value P_u . The relationship between peak load and bonding stress are shown in Equations (6)–(9).

$$P_u = (\tau_f + \tau_m) \cdot [l_a - (l_c - x)] \cdot C_a + \int_x^{l_c} \frac{\tau_{ca}}{l_c} \cdot \tau \cdot C_a d\tau \quad (6)$$

$$P_u = (\tau_f + \tau_m) \cdot (l_a - l_c + 0.5x) \cdot C_a + 0.5 \cdot \tau_{ca} \cdot l_c \cdot C_a \quad (7)$$

$$\frac{x}{l_c} \tau_{ca} = (\tau_f + \tau_m) \quad (8)$$

$$x = (\tau_f + \tau_m) \cdot l_c / \tau_{ca} \quad (9)$$

where: x is the sliding length of bonding diffusion zone in B critical state.

After B critical state, the bonding diffusion zone slips quickly, and the load drops rapidly. When the axial load increases to point C, all the bonding interfaces are destroyed, and the bonding diffusion zone disappears. The length of slip zone l_s is equal to the embedded length of steel l_a , and the chemical bonding stress completely disappears. At the C state, the relationship between axial load and bonding stress are shown in Equation (10).

$$P_r = \xi_r (\tau_f + \tau_m) \cdot l_a \cdot C_a \quad (10)$$

where: ξ_r is the bond stress degradation factor corresponding to residual points.

The bond interface damage is basically complete after C critical state. When the axial load increases to the critical point D, the bonding interface is ground very smooth, and the mechanical biting stress generated by the furrow phenomenon is lost. The shear force transferred by the bonding interface is only borne by the friction resistance stress. Therefore, the relationship between the axial load and bonding stress is shown in Equation (11).

$$P_d = \xi_d \tau_f \cdot l_a \cdot C_a \quad (11)$$

where: ξ_d is the bond stress degradation factor corresponding to failure points.

The chemical bonding stress τ_{ca} , friction resistance stress τ_f , and mechanical biting stress τ_m can be obtained by substituting the characteristic bond strength into Equations (5)–(11), and the results are shown in Table 4.

Table 4. Composition and solution of bonding stress.

Specimen	l_c (mm)	l_c/l_a	x (mm)	τ_{ca} (MPa)	τ_m (MPa)	τ_f (MPa)	τ_m/τ_{ca}	τ_f/τ_{ca}
SRRC-02	190.72	95%	168.42	0.604	0.100	0.443	0.165	0.733
SRRC-03	179.92	90%	145.42	0.773	0.075	0.560	0.097	0.724
SRRC-04	182.99	92%	141.60	0.747	0.109	0.479	0.146	0.641
SRRC-05	191.76	96%	152.25	0.687	0.094	0.460	0.137	0.670
SRRC-06	181.92	91%	179.83	0.503	0.142	0.363	0.282	0.723
SRRC-07	181.88	91%	174.30	0.596	0.123	0.476	0.207	0.799
SRRC-08	174.36	87%	166.47	0.613	0.094	0.510	0.154	0.833
SRRC-09	223.92	112%	188.64	0.596	0.089	0.422	0.149	0.708
SRRC-10	162.31	81%	144.36	0.631	0.088	0.482	0.140	0.764
SRRC-11	126.47	64%	123.95	0.621	0.109	0.510	0.176	0.820
SRRC-12	179.40	90%	167.05	0.485	0.118	0.371	0.244	0.765
SRRC-13	176.20	88%	172.25	0.597	0.123	0.479	0.207	0.803
SRRC-14	168.22	84%	146.41	0.699	0.145	0.474	0.207	0.678
Average	178.01	89%	152.67	0.685	0.129	0.460	0.187	0.696

4.3. Parameter Analysis

4.3.1. Replacement Ratio of Recycled Concrete

Figure 13a shows the relationship between l_c/l_a (the length of bonding diffusion zone vs. the embedded length of steel) and the replacement ratio of recycled concrete under the same conditions of other analysis parameters. It can be seen from Figure 13a that the length of the bonding diffusion zone accounts for a large proportion of the embedded length of the specimen, reaching to more than 90%. This is partly due to the smaller embedded length designed in this test. l_c/l_a increases with the replacement ratio of the recycled concrete, but not obviously, which indicates that the length of bonding diffusion zone with different replacement ratios of recycled concrete are not the same. The length of the bonding diffusion zone of ordinary concrete is the smallest, which indicating that the cement colloid formed by ordinary concrete has a better adhesive effect. The cement colloid effect gradually decreased with the increase in the replacement ratio.

Figure 13b shows the influence of the replacement ratio on the chemical bonding stress and friction resistance stress. Both chemical bonding stress and friction resistance stress decrease with the increase in replacement ratio. However, the replacement ratio has little effect on friction resistance stress.

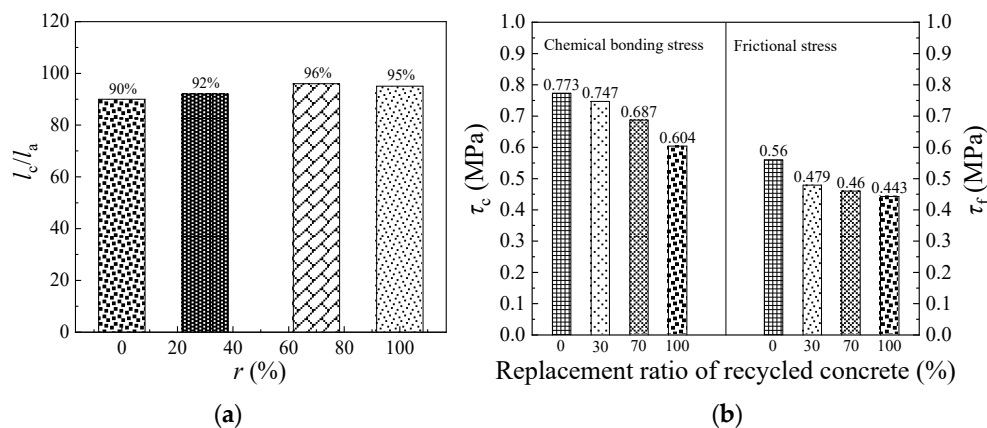


Figure 13. The effect of replacement ratio of recycled concrete on the bonding capacity, they should be listed as: (a) The effect of replacement ratio of recycled concrete on l_c/l_a ; (b) effect on the chemical bonding stress and friction resistance stress.

4.3.2. Cover Thickness of Shape Steel

The influence of cover thickness of shape steel on the length of bonding diffusion zone vs. the embedded length of steel is shown in Figure 14a. It can be seen from Figure 14a that the l_c/l_a increases first then decreases with the increase in cover thickness of the shaped steel. The maximum l_c/l_a value is 95% in SRRC-02, and the minimum l_c/l_a value is 87% in SRRC-08. Within a certain range, increasing the cover thickness of shape steel is beneficial to the bonding performance of the bonding interface.

Figure 14b shows the influence of the cover thickness of shape steel on the mechanical biting stress and friction resistance stress. The cover thickness of shape steel has a more significant influence on the friction resistance stress. The friction resistance stress gradually increases with the increase in the cover thickness of shape steel. The friction resistance stress corresponding to the four cover thicknesses (50, 60, 70 and 80 mm) are 0.363 MPa, 0.443 MPa, 0.476 MPa, and 0.510 MPa, respectively. That is, for every 10 mm increase in the cover thickness, the friction resistance stress will increase by 0.038 MPa on average. This is mainly because the cover thickness provides a strong lateral constraint for the bonding interface, increasing the normal pressure to the bonding interface, and as a result increasing the friction resistance.

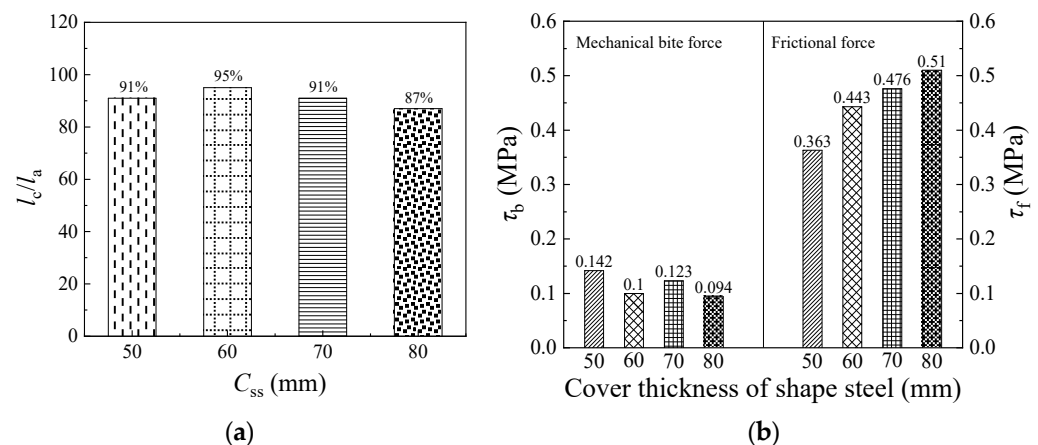


Figure 14. The effect of cover thickness of steel on the bonding capacity. (a) The effect of cover thickness of steel on l_c/l_a ; (b) effect on the mechanical biting stress and friction resistance stress.

4.3.3. Reinforcement Ratio

Figure 15a shows the influence of reinforcement ratio on l_c/l_a . Compared with replacement ratio of recycled concrete and the cover thickness of shape steel, the reinforcement ratio has a significant influence on l_c/l_a . l_c/l_a decreases significantly with the increase in the reinforcement ratio, the l_c/l_a value of SRRC-09 is 112%, which is the biggest, the l_c/l_a value of SRRC-11 is 64%, which is smallest. It indicates that the reinforcement ratio has advantages in decreasing the bonding diffusion zone.

The influence of the reinforcement ratio on the chemical bonding stress τ_{ca} , friction resistance stress τ_f , and mechanical biting stress τ_m is shown in Figure 15b. Both chemical bonding stress and friction resistance stress increase with the increase in the reinforcement ratio. When the reinforcement ratio increases from 0.62% to 1.54%, the chemical bonding stress increases from 0.596 MPa to 0.621 MPa, with an increase of 0.025 MPa; while the friction resistance stress increases from 0.422 MPa to 0.510 MPa, with an increase of 0.088 MPa. Therefore, the reinforcement ratio has more influence on the friction resistance stress, but not a significant impact on the mechanical biting stress.

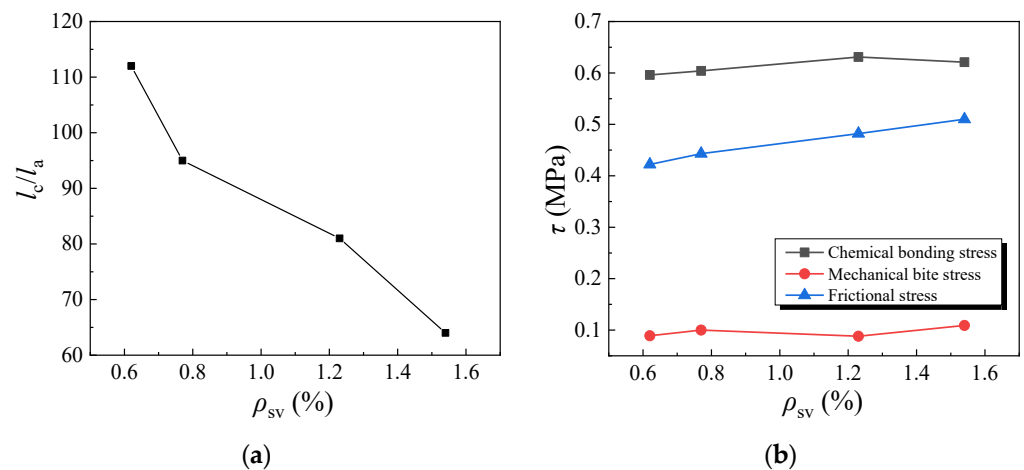


Figure 15. The effect of reinforcement ratio on the bonding capacity. (a) The effect of reinforcement ratio on l_c/l_a ; (b) the effect of reinforcement ratio on the bonding stress.

4.3.4. Recycled Concrete Strength

Figure 16a shows the influence of recycled concrete strength on l_c/l_a . l_c/l_a decreases with the increase in recycled concrete strength to some extent, but not too much. Figure 16b shows the influence of recycled concrete strength on the chemical bonding stress and friction resistance stress. Both the chemical bonding stress and friction resistance stress increase with the increase in recycled concrete strength. As can be seen from Figure 16b, the recycled concrete strength has a relative great impact on the chemical bonding stress than friction resistance stress. The chemical bonding stress of SRRC-12 is 0.485, which is the smallest, while the chemical bonding stress of SRRC-14 is the biggest. The chemical bonding stress increases by 0.214 MPa when the concrete strength increased from C30 to C50. However, the influence of recycled concrete stress on frictional resistance is weak compared the impact on chemical bonding stress.

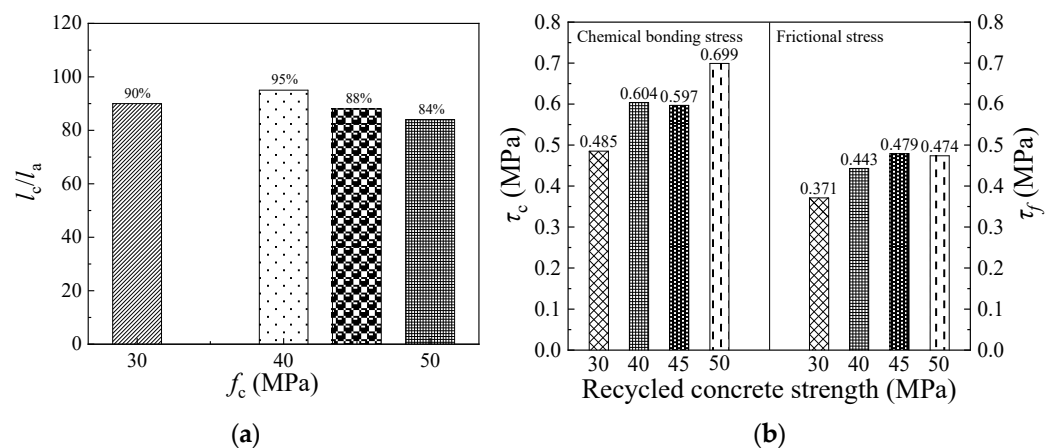


Figure 16. The effect of recycled concrete strength on the bonding capacity. (a) The effect of recycled concrete strength on l_c/l_a ; (b) the effect of recycled concrete strength on chemical bonding stress and friction resistance stress.

5. Summary and Conclusions

This paper investigated the bond capacity and load transfer mechanisms between H-shaped steel and recycled concrete under cyclic loading. Fourteen specimens were tested and performed to carry out a full-stage analysis. A degradation factor of bonding stress ζ was proposed to investigate the bond degradation at the interface under cyclic loading. The following conclusions can be drawn based on the results and analysis of the test:

- Based on the effect of shear transfer on the bond capacity, the whole loading procedure can be divided into four phases with four limit points as follows: Micro slip phase (OA)→micro slip point (A)→slip development phase (AB)→ultimate point (B)→slip increasing rapidly phase (BC)→residual point (C)→residual phase (CD).
- The bond–slip damage process can be divided into three phases: (1) linear rapid damage phase (0~0.6); (2) damage accumulation transition phase (0.6~0.9); and (3) damage stabilization phase (0.9~1.0). The bonding damage develops rapidly, which reaches over 50% when the slip at interface is about 0.2 mm.
- The bonding stress between H-shaped steel and recycled concrete is composed of chemical bonding stress τ_{ca} , friction resistance stress τ_f , and mechanical biting stress τ_m . The mean values of the chemical bonding stress τ_{ca} : friction resistance stress τ_f : mechanical biting stress τ_m is 1: 0.187: 0.696. In the composition of bonding stress, the chemical bonding stress is the largest, friction resistance stress is the second-largest, and the mechanical biting stress is the least.
- The average ratio of the length of bonding diffusion zone vs. the embedded length of steel l_c/l_a is 89%. l_c/l_a increases with the increase in the replacement rate of recycled aggregate, and decreases with the increase in the thickness of the section steel protective layer, the reinforcement ratio and the strength of recycled concrete. The chemical bonding stress τ_{ca} and friction resistance stress τ_f decrease with the increase in the replacement ratio of recycled concrete, and increase with the increase in cover thickness of shape steel, reinforcement ratio, and recycled concrete strength. The effect of each design parameter on the mechanical biting stress is not obvious.

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Article

Improvement of 3D Printing Cement-Based Material Process: Parameter Experiment and Analysis

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Abstract: Three-dimensional printing concrete is a digital and automating construction technology, which is expected to solve a series of problems existing in the traditional construction industry, such as low automation, high labor intensity, low efficiency and high risk. However, there are still many technical and operational challenges. The purpose of this paper is to provide insights into the effects of process parameters on the geometry and stability of the printed layer. Firstly, a theoretical model is established to analyze the structure of the printed layer under different nozzle speeds, material flow rates and nozzle offset. Secondly, a slump test is carried out to select the optimal ratio suitable for 3D cement printers, and the specimen is printed under various conditions. Finally, based on the obtained parameters, multiple nozzles are used for printing, and a pressure value suitable for each nozzle in the nonlinear path is calculated. The experimental results show that theoretical model can sufficiently verify printing structure in different parameter intervals, and the process parameters (nozzle speed, material flow rate and nozzle offset) can be changed to achieve the best effect of cement-based material forming structure.

Keywords: 3D printing; cement-based mortar; theory model; process parameters; multi-nozzle structure

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1. Introduction

The construction industry accounts for about 10% of total world output, but productivity in construction has barely increased since 1945 [1–3]. Compared with other industrial sectors (such as manufacturing, retail and agriculture), the productivity of the construction industry is extremely low: there are problems such as low technological content of production mode and lagging technological innovation. Meanwhile, problems such as the aging population and rising labor costs are gradually aggravated. Therefore, many researchers are urgently exploring solutions to change this situation.

With advent of Industry 4.0 [4], improving the degree of automation in the manufacturing stage of the construction process is expected to solve the problem of low productivity [5]. In this context of rapid development, construction 3D printing is an effective way to improve the current situation of the construction industry. Because the construction process has better degrees of freedom, lower labor costs and faster productivity, it has received wide attention [6–10]. At present, construction 3D printing has contour crafting [11,12], D-shape [13], concrete printing [14] and construction methods that are applicable to the field of architecture, and have also been successfully applied to artificial bridge construction, military shelters, rapid disaster relief shelters [15–18] and complex structures (an ultra-high-performance structure with complex geometric shapes designed by Gosselin et al. [19]). Different mechanical mechanisms and process parameters have certain influence on the shape, stability, material accumulation and fracture of the printed parts, that is, there are

different forming effects, the geometric shape and local stability of the extrusion layer are key factors affecting the final deforming effect.

Many researchers have studied the influence of printing parameters on forming mechanism and obtained the corresponding optimal process parameters. Farahbakhsh, M et al. [20], by manipulating process parameters, the layer geometry was changed to simulate densification, and friction between successive layers was enhanced by periodically increasing the printed area or enhancing compaction during printing (Figure 1a,b). Carneau, P et al. [21] provide insights on the effect of printing parameters on the geometry and local stability of the extruded layer. The upper and lower limit requirements for controlling lamination are discussed from the influence of geometric and motion parameters, as well as from the study of the properties of fresh materials. (Figure 1c). Huang, X et al. [22] introduced the influence of printing parameters on molding quality and interlayer bond strength through single-factor experiment, and obtained corresponding printing parameters (Figure 1d). Chen, Y et al. [23] studied the influence of time interval and nozzle distance on the interlayer bond strength, and the results showed that extending the time interval could reduce bond strength, while only increasing the nozzle distance had limited influence on the bond strength (Figure 1e). Elistratkin, M et al. [24] studied the combined influence of factors such as layer thickness, nozzle speed and screw rotation speed on the extrusion and printing track parameters of building materials, and established the relationship between the above factors to improve the stability of the printing process and the final structural quality (Figure 1f). Based on the above discussion, the process parameters are the key technical indexes for building structure printing, which not only affect the interlayer bonding effect, but also have a certain impact on the appearance of the printed product. Therefore, it is necessary to carry out the research on the matching of the printing process parameters.

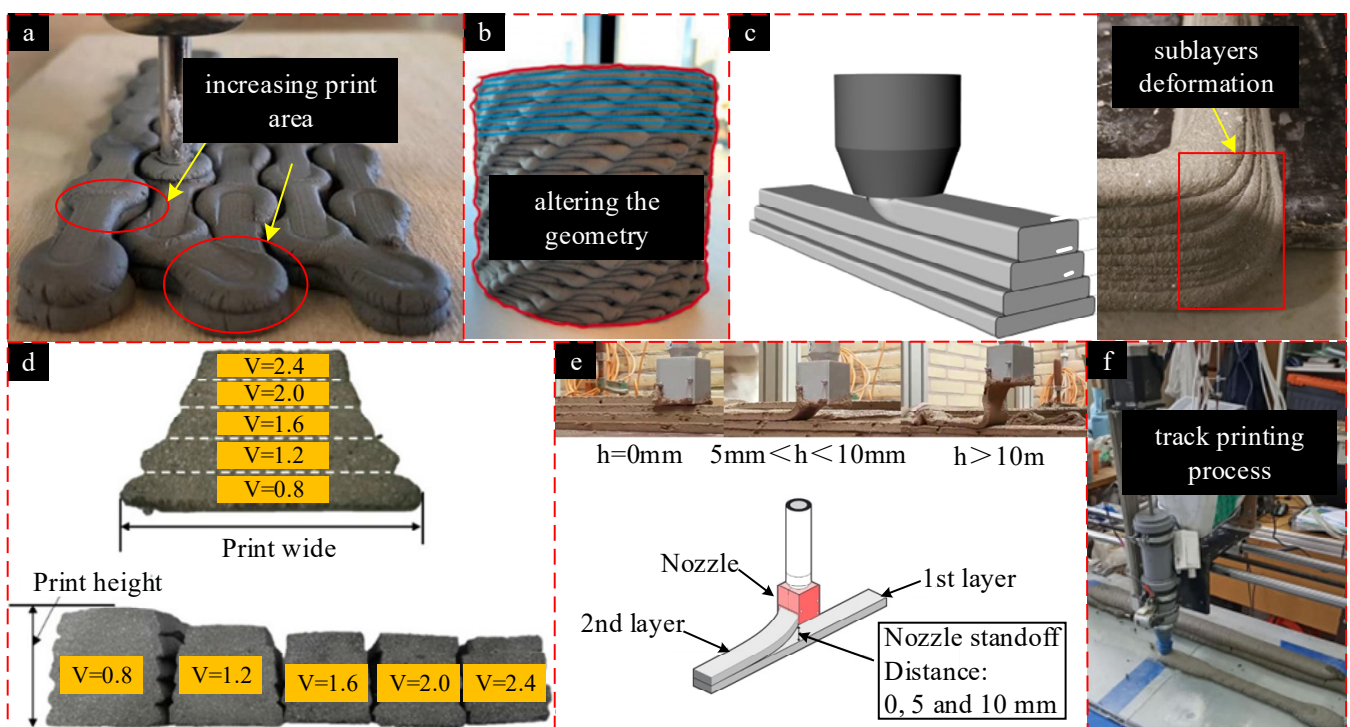


Figure 1. Effect of different printing parameters. (a) Increased surface contact between successive layers [20], (b) Altering the geometry [20], (c) Over pressing of the sublayers [21], (d) Influence of printing speed on concrete size [22], (e) Increasing the nozzle standoff distance could lead to the inaccurate layer deposition [23], (f) Track printing process [24].

In this paper, the influence of process parameters on material fluidity and molding effect was studied by single-factor experiment, taking tailing sand content, resting time, nozzle speed, pressure and nozzle offset as influencing factors, and slump, embryo strength,

layer height and layer width as combining indicators. The self-developed 3D concrete printing equipment was utilized to analyze the matching relationship of print parameter, which can be adjusted to optimize printing parameter combination was obtained and provided reference for related research.

2. Materials and Methods

2.1. Materials

In order to determine the optimum sand content, a mixing ratio test was designed. P.O 42.5 ordinary Portland cement was selected as the main cementitious, and the first grade fly ash, S95 grade slag powder and SF96 grade silicon powder were used as admixture to improve the micro pore structure, bonding performance, cement-based strength and pumping performance of mortar [25]. In addition to the above cementing materials, the chemical admixture used hydroxypropyl methyl cellulose ether and sodium gluconate retarder, which can greatly improve the plasticity and water retention of mortar and prevent drying and cracking. Tailing sand was selected for fine aggregate, and its physical properties were shown in Table 1. In order to ensure accuracy of water–binder ratio, tailing sand should be dried and sealed before use to ensure that the moisture content is 0. The water used for the experiment was tap water or medium water (medium water generally refers to reclaimed water, waste water or rainwater after proper treatment, to reach a certain quality index, can be used for beneficial water), and the printing experiment were carried out at room temperature. The process parameters were shown in Table 2.

Table 1. Physical properties of tailing sand.

Fine Aggregate	Fineness Modulus	Average Particle Size (mm)	Bulk Density (kg/m ³)	Mud Content (%)
Tailing sand	0.7	0.15	1400	0.4
Admixture	Fineness (45 micron standard sieve)	Average particle size (μm)	Density (g/cm ³)	Water content (%)
Fly ash	≤5%	5	2.6	≤1
Slag powder	≤3%	38	2.8	≤1
Silicon powder	≤3%	5	2.6	≤3

Table 2. Experimental material parameters.

Tailing Sand/%	Water/Cement Ratio	Cement/%	Mineral Admixture/%	Thickener/%	Retarder/%
0	0.39	80.2	19.8	0.05	0.3
10	0.39	80.2	19.8	0.05	0.3
20	0.39	80.2	19.8	0.05	0.3
30	0.39	80.2	19.8	0.05	0.3
40	0.39	80.2	19.8	0.05	0.3
50	0.39	80.2	19.8	0.05	0.3

The mixing process is as follows: Firstly, weigh out cement, admixtures, sand and admixtures. Secondly, first the cement and admixture mix and stir evenly, then pour the sand into the mix evenly, and then pour the admixture into the mixture, stir for 2–3 min after all mixing. Finally, gradually add water and stir.

2.2. Methods

2.2.1. 3D Cement Printing Equipment

A self-developed four-axis motion 3D cement printer was used in the experiment (Figure 2a,b). The X-axis, Y-axis and Z-axis motion mechanism was composed of linear modules. Stepping motors were used to control A axis rotation. Numerical control technology was used to control the stroke accurately. The printing system of the equipment was divided into two parts: one is the extrusion device, including air source (Figure 2c), ink cartridge (Figure 2d) and nozzle (Figure 2e). The other part is the Mach3 control system

that controls the operation of equipment (Figure 2f). The specification of the 3D printing system was show in Table 3.

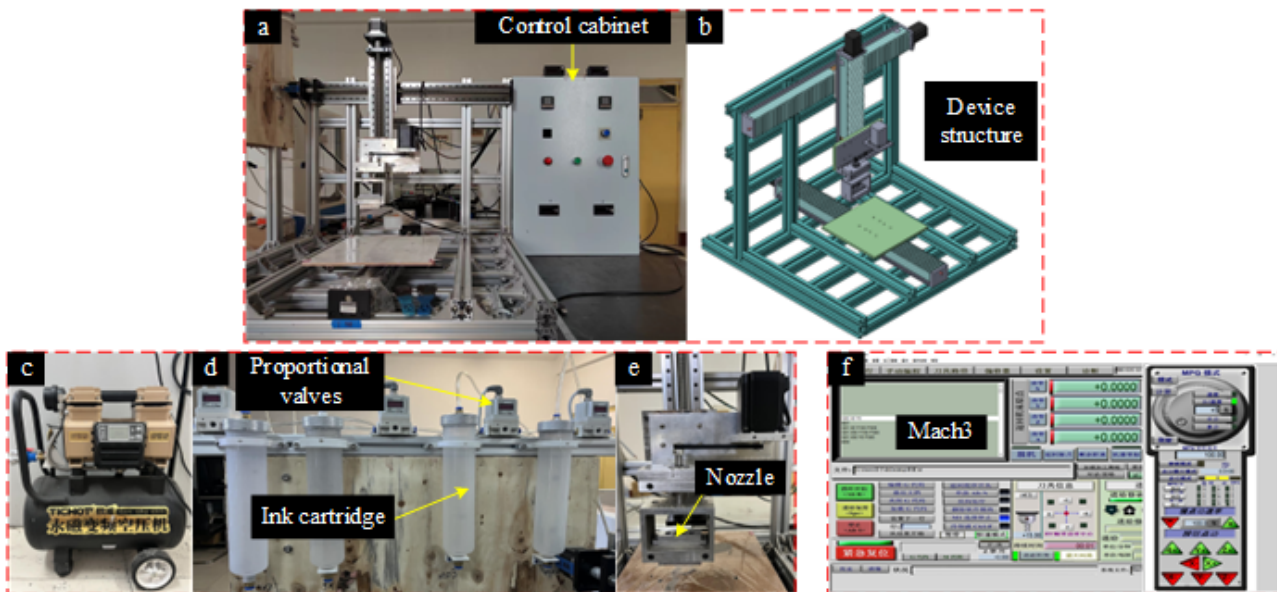


Figure 2. Three-dimensional cement printing equipment. (a) Three-dimensional concrete printing device, (b) Device structure, (c) Air source, (d) Ink cartridge, (e) Nozzle, (f) Control system.

Table 3. 3D printing system specifications.

Parameters	Value
Size specification (X × Y × Z)	800 × 800 × 600 mm ³
Walking accuracy	±1 mm
Print speed	0–3000 mm/min
Pressure range	0–1.0 MPa

2.2.2. Theory

Figure 3 summarizes the influence of cement-based material flow rate and nozzle speed on printing molding. The cement-based material can be squeezed through pressure control system, and the nozzle speed was controlled by stepper motor. With intermediate cement-based material flow-rate and nozzle speed, the width of the print layer will be similar to the nozzle diameter. In this case, the printing process can obtain more ideal specimen (Figure 3a). With a high flow-rate of cement-based material or low nozzle speed, the width of the print layer was often larger than the nozzle diameter (Figure 3b). A wider path will strengthen the bond between the layers and consequently achieve better mechanical properties, and on the other hand, the printing process can easily lead to material accumulation in narrow, non-linear paths. With a low flow-rate or high speed, the width of the print layer will be similar to or smaller than the nozzle diameter (Figure 3c). However, when the flow-rate was reduced (pressure value was less than 0.045 MPa), the cement-based material creates friction with the bottom surface, causing a dragging phenomenon and subsequent fracture (Figure 3d). The relationship between nozzle velocity and X, Y axis velocity is:

$$F = \sqrt{V_x^2 + V_y^2} \text{ (mm/s)} \quad (1)$$

$$\frac{F}{V_x} = \frac{S}{L_x} \quad (2)$$

$$\frac{F}{V_y} = \frac{S}{L_y} \quad (3)$$

where F is the theoretical speed of nozzle, in mm/s. S is the theoretical displacement of nozzle, in mm. V_x is the actual speed in the X-axis direction, in mm/s. L_x is the actual displacement in the X-axis direction, in mm. V_y is the actual speed in the Y-axis direction, in mm/s and L_y is the actual displacement in the Y-axis direction in mm.

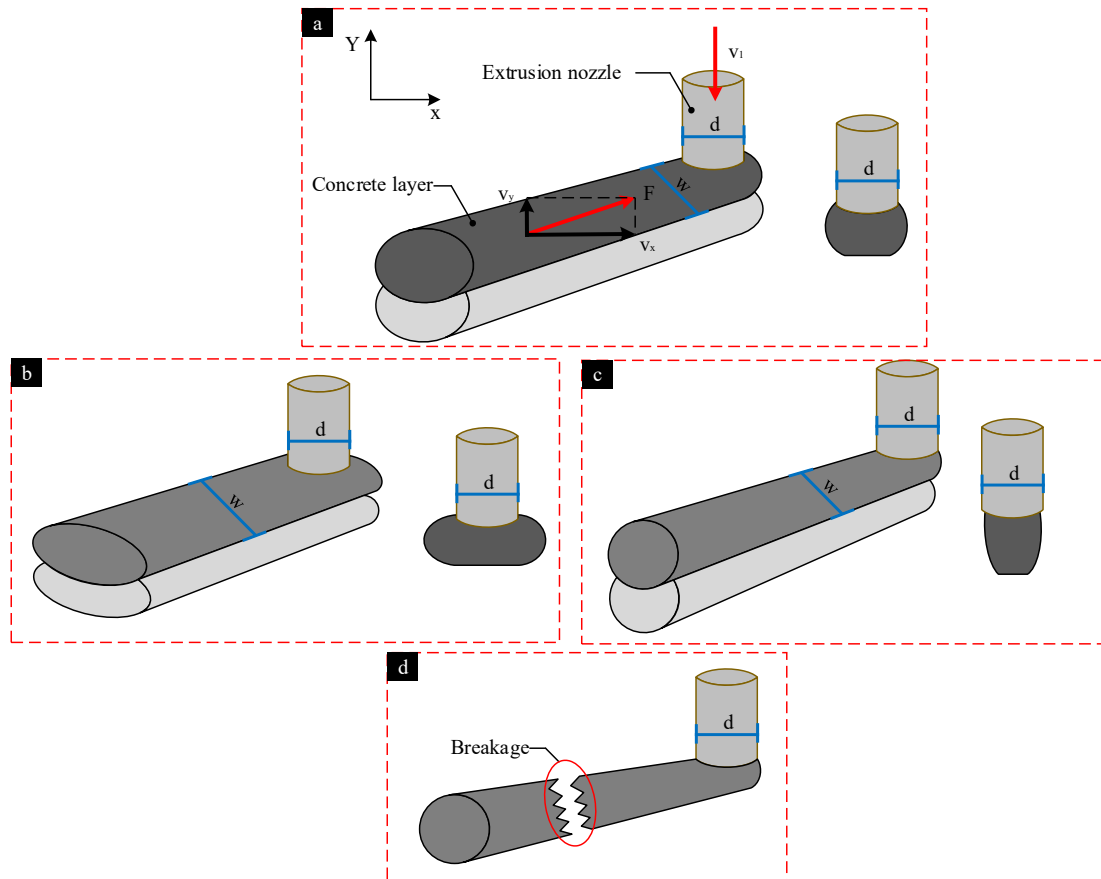


Figure 3. Influence of flow-rate of material and printing nozzle speed on molding. (a) Ideal state, (b) High flow-rate of material or low printing nozzle speed, (c) Low flow-rate of material or high printing nozzle speed, (d) Breakage state.

The material extrusion speed V_1 is:

$$t = \frac{S}{F} \text{ (s)} \quad (4)$$

$$A = \pi r^2 b \text{ (mm}^3\text{)} \quad (5)$$

$$V_1 = \frac{A}{t} \text{ (mm}^3\text{/s)} \quad (6)$$

where t is the printing time, in s. b is the height of material falling in the ink cartridge during the printing time, in mm. r is the radius of the ink cartridge, in mm and A is the volume of material extruded during the printing time in mm³.

Figure 4 summarizes the influence of nozzle height on printing molding under optimal material flow-rate and nozzle speed. If the nozzle offset (H) was much larger than the nozzle diameter (d), the material gradually tilts forward, resulting in a layer height consistent with the nozzle offset, and the cross-section is trapezoidal (Figure 4a). In this case, the inter-layer contact area was smaller, and the inter-layer bond strength was weaker [21]. If H was equal to d , the surface will be slightly flattened by the nozzle, and a large inter-layer contact area can be obtained without destroying the shape (Figure 4b). If H was much smaller than d , the nozzle will push the cement-based material to both sides, forming thin layer (Figure 4c).

In this case, there may be some cracks and other defects in the spilled concrete on both sides. The relationship between nozzle offset and specimen width as

$$H = h \text{ (mm)} \quad (7)$$

$$w = \frac{A}{SH} \text{ (mm)} \quad (8)$$

where h is layer height, in mm. H is the nozzle offset and w is layer width in mm. When the volume of extruded material was constant, the layer height was inversely proportional to the layer width.

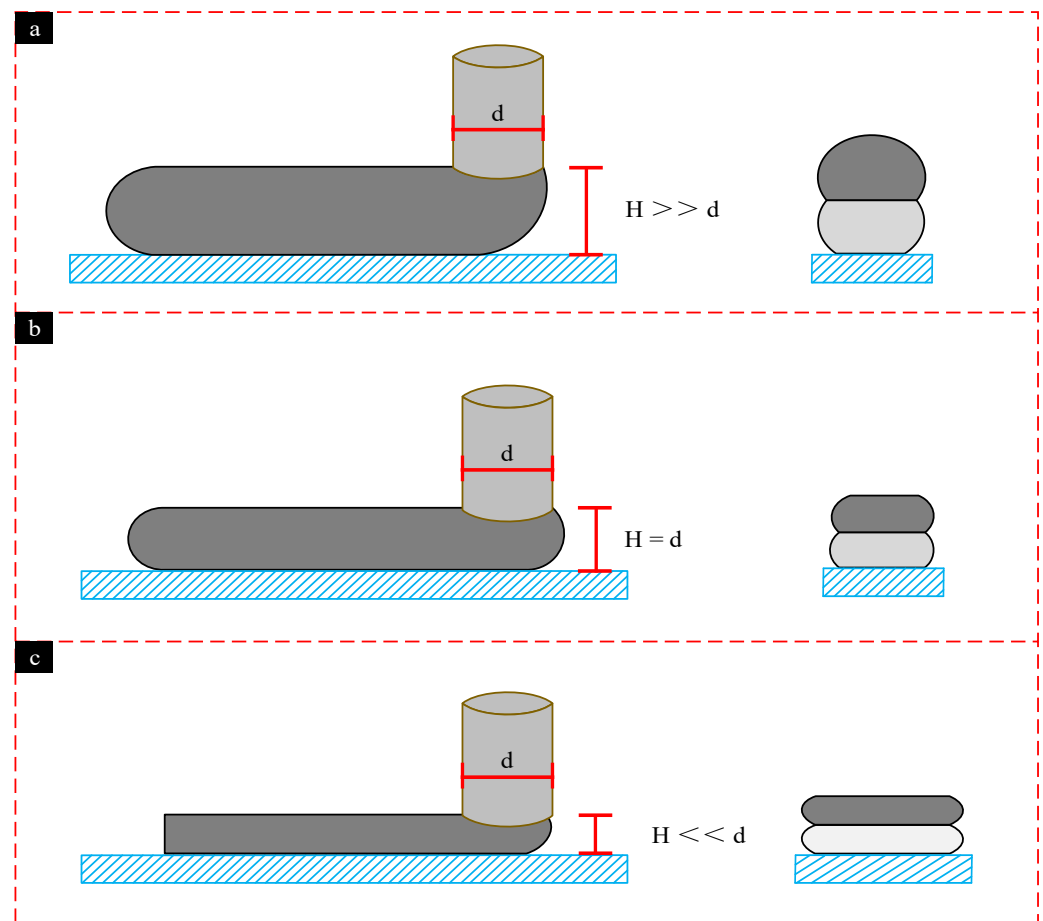


Figure 4. Influence of nozzle offset on molding. (a) The nozzle offset is much higher than the nozzle diameter, (b) The nozzle offset is equal to the nozzle diameter, (c) The nozzle offset is much smaller than the nozzle diameter.

2.2.3. Experimental Procedure

By controlling the content of tailing sand, the cement-based materials have well fluidity, extrudability and constructability. The tailing sand content was set to 0%–50% when the other ratios were unchanged (gradient value of 10%), as show in Table 2. The mixed cement-based material was loaded into a 100 × 100 mm cylinder (Figure 5a) and then tamped with a tamper stick. The cylinder was removed, and the specimen was gradually pressurized until the specimen severely deformed (Figure 5b). The pressure was then converted to strength (Pressure divided by contact area, $F = mg$, $P = F/S$). After the optimal content of tailing sand was selected, the slump and strength were tested after 0, 5, 10, 15 and 20 min, respectively, to select the optimal printing time.

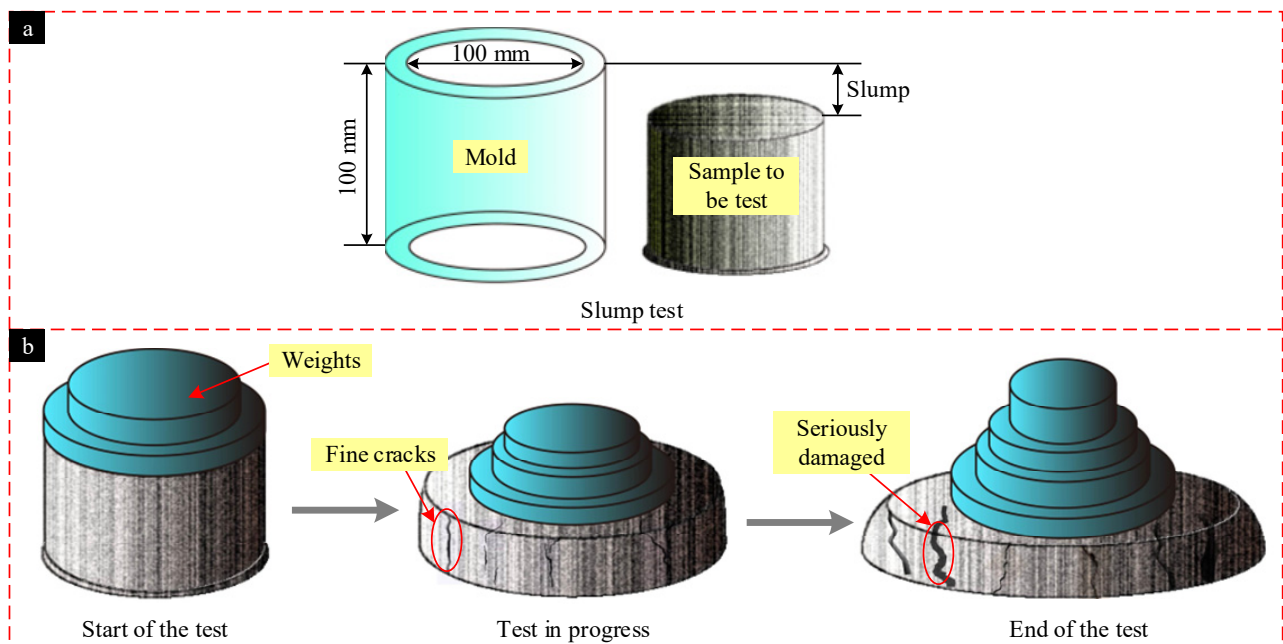


Figure 5. Slump and strength test theoretical model. (a) Slump test, (b) The test process of strength.

The theoretical model was verified under the support of optimal mix ratio. Experiments were designed to analyze the printing effect of cement-based materials under different parameters. Firstly, the influence of nozzle speed on molding effect was analyzed under the same pressure. The pressure was set to 0.045 MPa, and the nozzle speed was 1.667–13.333 mm/s (gradient value of 1.667 mm/s), as shown in Table 4. Secondly, the influence of pressure on the molding effect was analyzed at the same nozzle speed. The nozzle speed was set to 8.333 mm/s, and the pressure was set to 0.045–0.144 MPa (gradient value of 0.009 MPa), as shown in Table 5. Finally, the influence of different nozzle height on the molding effect was analyzed. The nozzle speed was set to 8.333 mm/s, the pressure was set to 0.117 MPa, the nozzle diameter was set to 10 mm and the nozzle height was set to 3, 5, 7.5, 10, 12 and 14 mm, as show in Table 6.

Table 4. Different nozzle speed under the same pressure.

Number	Nozzle Speed (mm/s)	Pressure Value (MPa)	Number	Nozzle Speed (mm/s)	Pressure Value (MPa)
1	1.667	0.045	5	8.333	0.045
2	3.333	0.045	6	10	0.045
3	5	0.045	7	11.667	0.045
4	6.667	0.045	8	13.333	0.045

Table 5. Different pressure at the same nozzle speed.

Number	Pressure Value (MPa)	Nozzle Speed (mm/s)	Number	Pressure Value (MPa)	Nozzle Speed (mm/s)
1	0.045	8.333	7	0.099	8.333
2	0.054	8.333	8	0.108	8.333
3	0.063	8.333	9	0.117	8.333
4	0.072	8.333	10	0.126	8.333
5	0.081	8.333	11	0.135	8.333
6	0.09	8.333	12	0.144	8.333

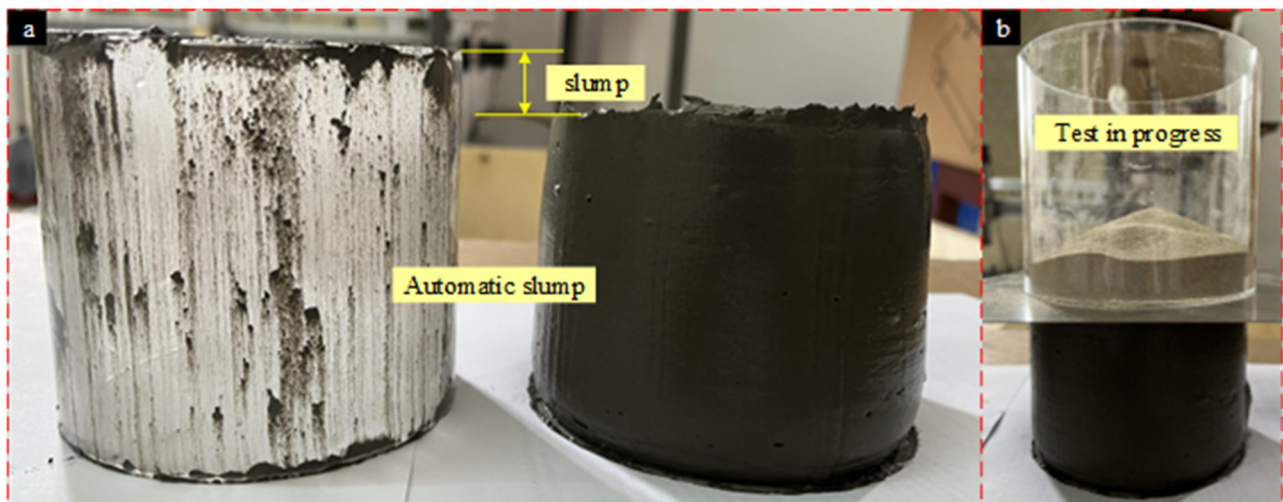
Table 6. Different nozzle offset at the same nozzle diameter.

Number	Nozzle Offset (mm)	Nozzle Diameter (mm)	Number	Nozzle Offset (mm)	Nozzle Diameter (mm)
1	3	10	4	10	10
2	5	10	5	12	10
3	7.5	10	6	14	10

3. Analysis and Discussion

3.1. Slump Analysis

Cement-based materials were usually extruded by nozzles, and fluidity was a key parameter to evaluate the extrudability of materials. Therefore, the good fluidity in the printing process can ensure that cement-based material was smoothly extruded continuously from the nozzle without break, and was easy to deposit and molding. Fluidity was usually expressed in terms of slump: the test process of slump and strength is shown in Figure 6. Slump refers to the height of automatic deposition of cement-based material (Figure 6a), and strength is converted by applied pressure (Figure 6b). The variation of strength and slump with tailing sand content was shown in Figure 7a. The slump of specimens decreased with the increase in tailing sand content, which indicated that constructability of cement-based materials can be effectively improved while the fluidity is reduced. At the same time, it can be seen from the figure that the strength increases almost linearly with the increase in tailing sand content. Combined with strength and slump analysis, the best printing results were obtained when the tailing sand content was 40%. The changes in slump and strength with time are shown in Figure 7b. With the increase in time, the higher strength of material, the better constructability of material. However, with the increase in time, the slump gradually decreased. When the time was longer than 15 min, the material was almost no slump, which will cause pipe blockage and was not conducive to printing. Therefore, 10 min after the stirring was the optimal printing time.

**Figure 6.** The test process. (a) Slump, (b) Strength.

3.2. Influence of Nozzle Speed and Pressure on Molding Effect

The extrudability of cement-based materials plays an important role in the experimental process, the pressure extrusion system is used to realize the on-demand extrusion of cement-based materials. Under high pressure conditions, the ink cartridges can burst due to excessive internal pressure and there are certain risks, cement-based materials were difficult to control, and material was difficult to extrude under low pressure. Therefore, in order to make the cement-based materials' extrusion stable and controllable, the pressure

was set to 0.045 MPa. At different nozzle speeds, the specimen morphology has different widths and heights. The width and height of the print layer variation with the nozzle speed are shown in Figure 8a. With the increase in nozzle speed, the width and height showed a downward trend.

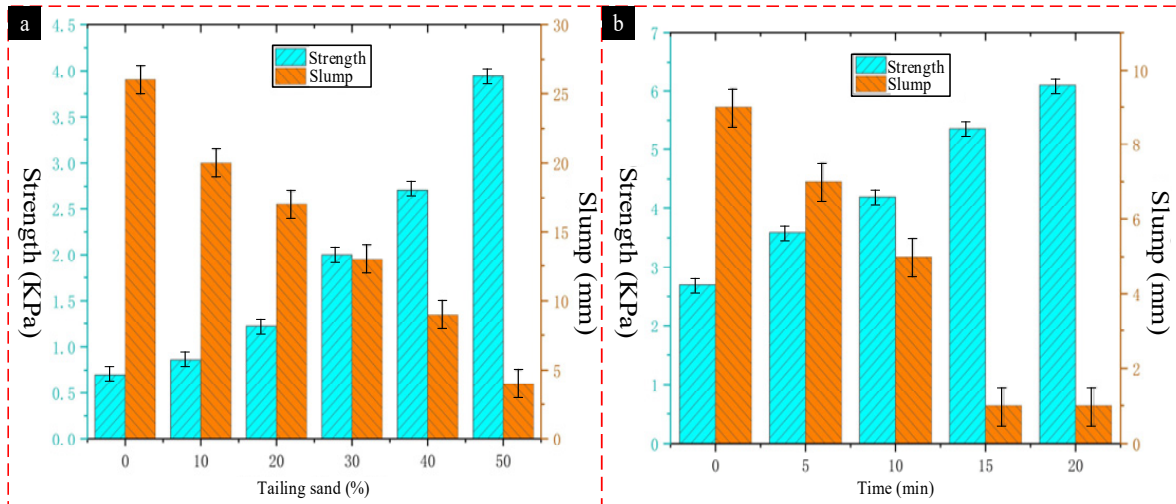


Figure 7. Variation of strength and slump ($n = 8$). (a) Variation of strength and slump with tailing sand content, (b) Variation of strength and slump with time.

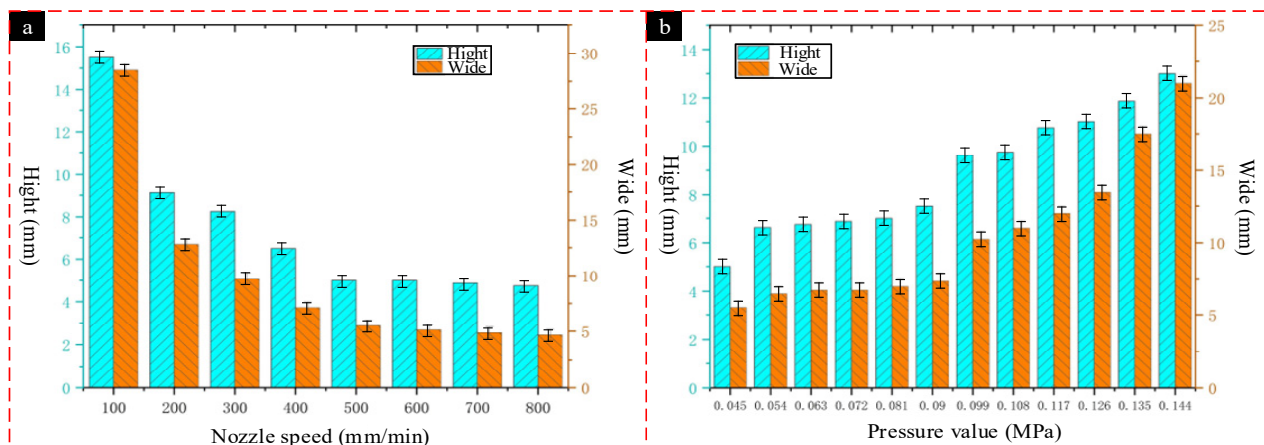


Figure 8. Cement-based material molding relationship. (a) Variation of layer height and wide with nozzle speed under the same pressure, (b) Variation of layer height and wide with pressure under the same nozzle speed.

In order to study the Influence of extrusion pressure on molding effect of cement-based materials and the optimal ratio of nozzle speed and extrusion speed, the nozzle speed was set to 8.333 mm/s (F500) according to the result in Figure 8a. Specimen had different morphology under different pressures. The variation of layer width and layer height with pressure are shown in Figure 8b. With the increase in pressure, the layer width and height showed an upward trend.

The printing effect of cement-based materials under different parameters is shown in Figure 9. When the nozzle speed was less than 3.333 mm/s (F200) and the pressure value was greater than 0.144 MPa, materials accumulation phenomenon will occur (Figure 9a). The surface of the specimen printed at low speed and high pressure was uneven, and materials accumulation can be obviously seen on the printing path (Figure 9d). However, an increase in layer width leads to increase in the contact area between layers, so the constructability was particularly outstanding in this case. On the contrary, when the

nozzle speed was greater than 8.333 mm/s (F500) and the pressure value was less than 0.081 MPa, the shape of the printing specimen was not sufficient to support the printing of the next layer (Figure 9c). The specimens printed at high speed and low pressure have poor construction performance and were prone to collapse during printing (Figure 9f). When the nozzle speed was 8.333 mm/s and the pressure value was 0.117 MPa, the surface of the printing specimen was smooth and flat, and there was no material accumulation. At the same time, specimens has good constructability and ideal molding effect (Figure 9b,e).

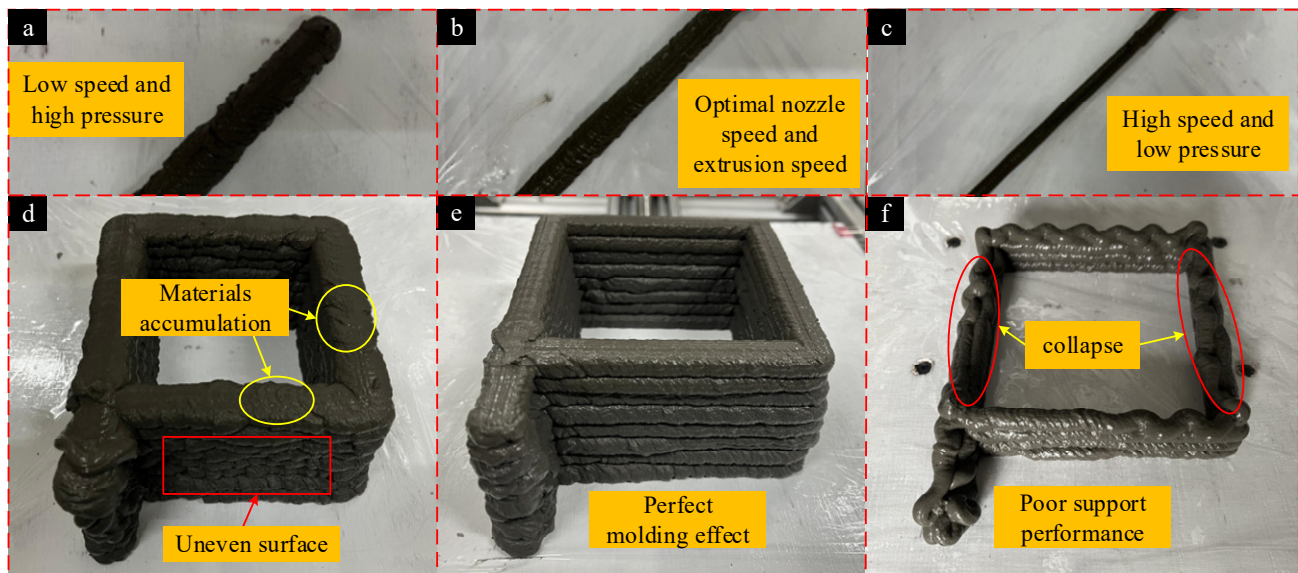


Figure 9. Printing effect of cement-based material under different parameters. (a) Shapes at low speed and high pressure, (b) Shape under the best matching parameters, (c) Shape at high speed and low pressure, (d) Printing molding effect under low speed and high pressure, (e) Printing molding effect under the best matching parameters, (f) Printing molding effect under high speed and low pressure.

3.3. Influence of Nozzle Offset on Molding Effect

Based on Section 3.2, the nozzle speed was set to 8.333 mm/s, the pressure was set to 0.117 MPa, and the nozzle diameter was set to 10 mm. Different nozzle offset lead to different deposition shapes. The changes in the layer width and layer height with the nozzle offset are shown in Figure 10. With the increase in the nozzle offset, the layer height showed an upward trend. When the nozzle offset was greater than or equal to the nozzle diameter, the layer height was consistent with the nozzle offset. According to Equation (8), the layer height was inversely proportional to the layer width, so the layer width showed downward trend.

The molding effect is shown in Figure 11. When the nozzle offset was less than 7.5 mm, compression condition of specimen was extremely serious, so the layer width was large (Figure 11a). When the nozzle offset was low for printing, the layer contact area was increased, and the constructability was improved. However, due to the severe extrusion of the printing layer, the materials will overflow on both sides of the printing path (Figure 11d). On the contrary, when the nozzle offset was greater than 10 mm, the layer height was larger and the layer width was smaller, which was not conducive to the support function of the lower layer to the upper layer, and higher the nozzle offset leads to inaccurate deposition, therefore, the constructability was poor (Figure 11c,f). In relative terms, the nozzle offset was set to be slightly smaller than the nozzle diameter. On the premise that the printing layer was not severely compressed, ensures good constructability, and at the same time, there was no material spillage during the printing process (Figure 11b,e).

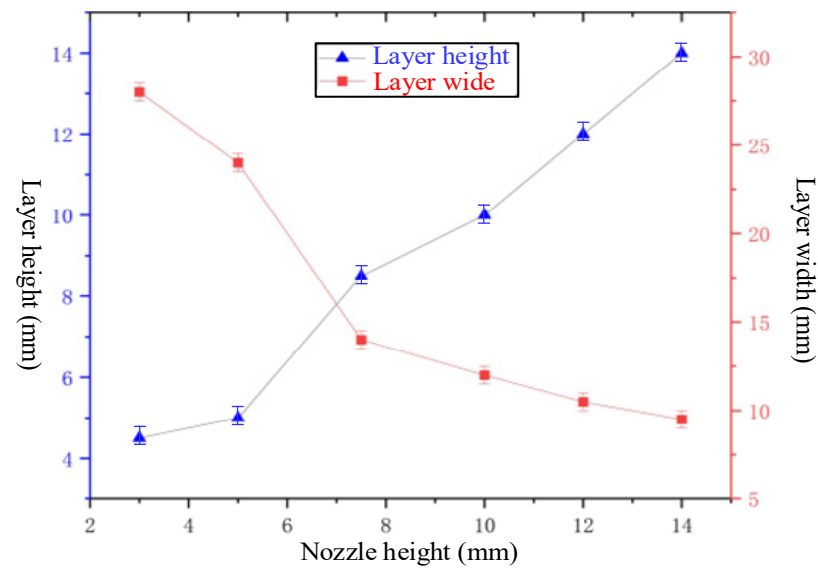


Figure 10. Effect of nozzle offset on layer height and layer width.

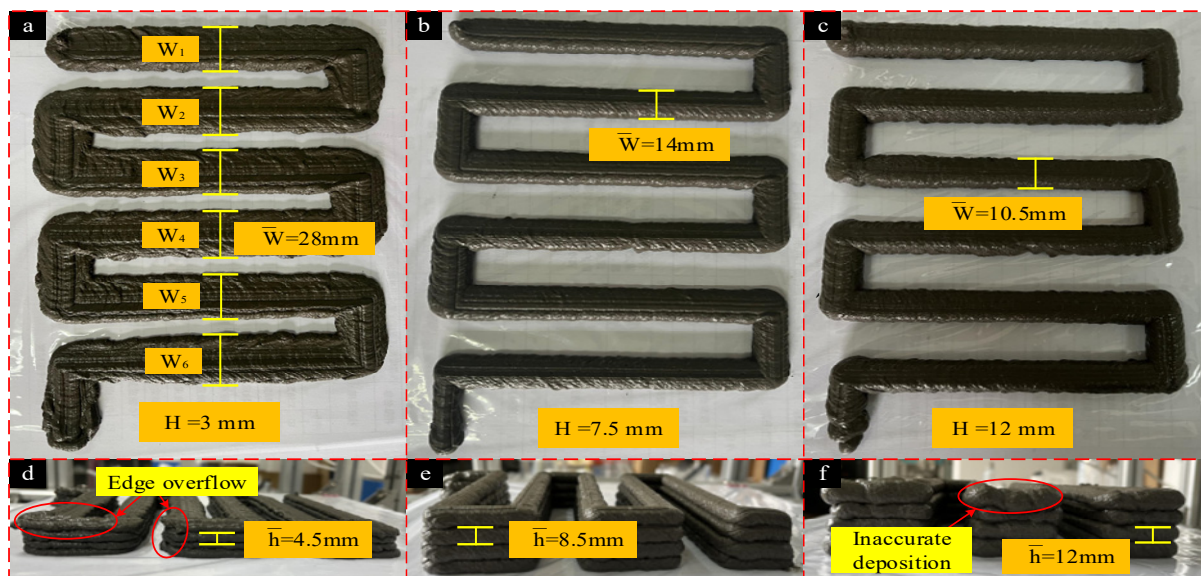


Figure 11. Printing effect of cement-based material under different nozzle offset. (a) Width of the print layer when the nozzle offset is 3 mm, (b) Width of the print layer when the nozzle offset is 7.5 mm, (c) Width of the print layer when the nozzle offset is 12 mm, (d) Height of the print layer when the nozzle offset is 3 mm, (e) Height of the print layer when the nozzle offset is 7.5 mm, (f) Height of the print layer when the nozzle offset is 12 mm.

3.4. Effect of Different Pressure on Multi-Nozzle Printing Molding

The nozzle device of the printer used in this paper can switch between single-nozzle extrusion and multi-nozzle extrusion by Mach3 control board. When multi-nozzle printing was carried out, different nozzles should be set different pressure values on the non-linear path to ensure that there was no materials accumulation. Based on the results of Sections 3.2 and 3.3, the nozzle speed of 8.333 mm/s, the pressure of 0.117 MPa, and the nozzle height of 7.5 mm were set as the basic parameters.

The molding effect of multi-nozzle printing as shown in Figure 12. When printing with double nozzles, the nozzle spacing can be adjusted, and the extrusion speed of each nozzle can be controlled by pressure, so as to meet the printing requirements of complex components. As shown in Figure 12a, the pressure value of inner ring was 0.099 MPa, and the pressure value of outer ring was 0.117 MPa. Therefore, the forming effect of the inner

and outer rings was inconsistent. When the three nozzles are printed without spacing, the pressure value in the linear path was unified to 0.117 MPa. However, in the non-linear path, because the paths of the three nozzles are different, the pressure value of the nozzles should be adjusted to achieve the effect of no materials accumulation (Figure 12b). The relationship between material extrusion speed and arc length as

$$V_1 = \frac{lh w}{t} \text{ mm}^3/\text{s} \quad (9)$$

where l is the arc length, in mm. layer height and layer width were fixed values, the volume of material required to calculate the nonlinear path, and the printing time was the same. Therefore, extrusion speed of different paths was obtained, and pressure value of each nozzle was determined according to the extrusion speed.

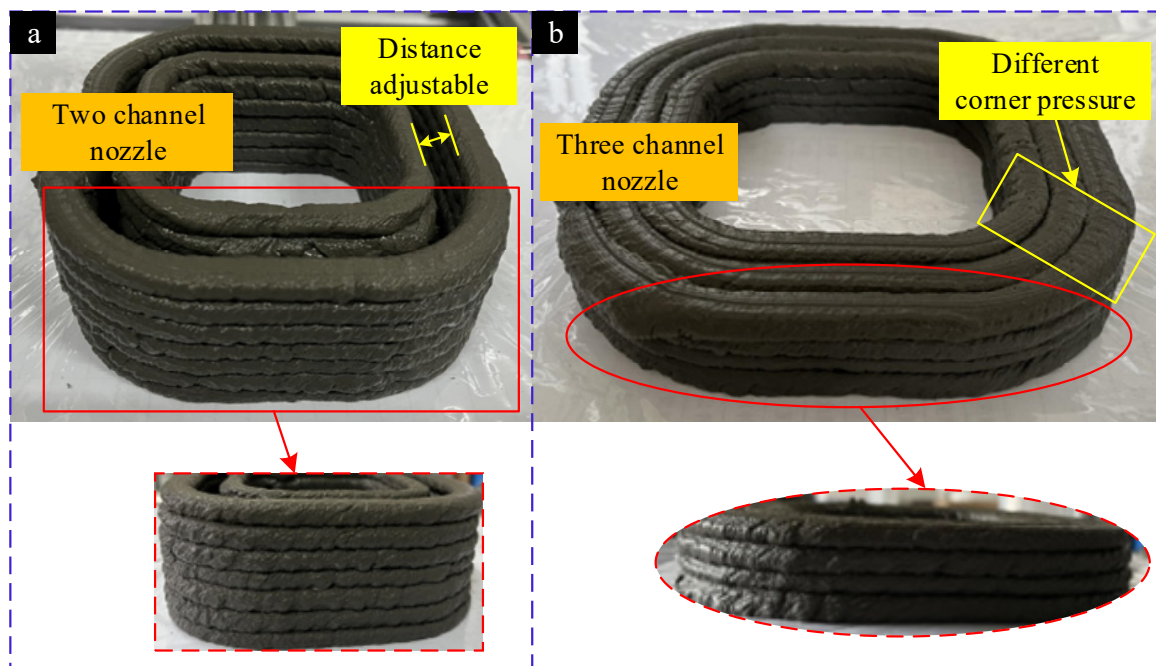


Figure 12. Multi-nozzle extrusion printing effect. (a) Double nozzle printing, (b) Three nozzle printing.

4. Conclusions

The 3D printing concrete process opens up a novel construction method, which will improve the complexity, diversity and geometric accuracy of products, and bring higher production efficiency to the construction industry. This paper aimed to investigate the influence of different printing parameters on the material printing molding through theoretical analysis and experimental methods. The conclusions are as follows:

- (1) The success of the printed structure depends on the suitability of the properties of the cement-based material. Through strength and slump experiments, the optimal mix ratio suitable for 3D building printer was obtained, which can provide technical support for subsequent experiments.
- (2) According to the influencing factors such as nozzle speed, material flow rate and nozzle offset, the theoretical models under different process parameters were analyzed, which can provide theoretical support for the subsequent parameter experiments.
- (3) The experimental results show that the theoretical model can fully verify the print structure in different parameter intervals, the layer width and layer height can be controlled by changing the related parameters, which can effectively control the nozzle speed, material flow and nozzle offset to achieve the best effect of cement-based material forming structure.

Author Contributions: Z.L.: Data curation, Writing—Original draft preparation. H.L.: Conceptualization, Methodology, Writing—Reviewing and Editing. X.C.: Visualization, Software. P.N.: Writing—Reviewing and Editing. X.Y.: Investigation. G.Z.: Validation, Writing—Reviewing and Editing. H.S.: Formal analysis. W.J.: Data curation. All authors have read and agreed to the published version of the manuscript.

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Article

Research on Dynamic Response and Construction Safety Countermeasures of an Adjacent Existing Line Foundation under the Influence of a New Railway Line

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Abstract: The excavation of a new high-speed railway causes the side slope adjacent to the existing line foundation to become airborne, and the excessive dynamic deformation or cumulative deformation caused by the dynamic load of trains will affect the normal service of the subgrade, even leading to its instability. To date, there are no relevant experimental data regarding this, and there is also a lack of corresponding specifications. The only available numerical simulation research results need to be verified in practice. Therefore, this study relies on the Shanghai–Nanjing intercity high-speed railway construction project adjacent to the existing Beijing–Shanghai line to carry out a subgrade dynamic response test to ensure the safe operation of the existing line. The test obtained the vibration displacement, frequency, acceleration, and other parameters of the existing subgrade construction in three stages: subgrade excavation, pile formation, and subgrade filling. From the test results: During the test period, the vertical surface vibration displacement and vibration acceleration have a certain attenuation along the depth direction. In the stage of subgrade excavation, the vibration displacement and vibration acceleration generated are the largest. The vertical vibration displacement amplitude reaches 1.9 mm, and the horizontal vibration displacement amplitude reaches 0.15 mm. The vibration frequency of the roadbed under the action of the train load is concentrated in the range of 0–50 Hz, and the vibration energy at the peak value is relatively large, which reflects the load action frequency of the train, and the peak value is mainly concentrated in the range of 20–40 Hz. These results show that the maximum vibration response peak appears in the subgrade excavation stage, that is, the most dangerous stage of the existing subgrade. The vibration acceleration and vibration displacement of each dynamic response parameter are important in that they reflect the dynamic performance of the subgrade and establish the index control standard, which can be used as a control index for roadbed dynamic stability monitoring. The dynamic test of the subgrade state provides data support for the reasonable opening of the construction skylight and the protection of the excavation slope. Taking into account the impact of piling vibration, technical measures such as static pressure, jumping construction, and setting up stress relief holes are adopted. The test results and engineering measures ensure the safe operation of the existing subgrade, and have important theoretical significance for guiding the construction of the new subgrade adjacent to the existing line.

Keywords: intercity high-speed railway construction; adjacent to existing rail lines; dynamic test; pipe–pile construction

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1. Introduction

The service life of large civil structures such as railways usually lasts for decades, or even hundreds of years. During the service period, structures are inevitably affected by external environmental erosion, material aging, etc., so that the engineering body becomes damaged to varying degrees. With the accumulation of these damages, the bearing capacity

of engineering structures is reduced, and their safety and normal function are affected. In recent years, China has built a large number of high-speed railways, and it is inevitable that the new lines will run parallel to the existing lines in normal operation. The new Shanghai–Nanjing Passenger-Dedicated Line was built for public transportation between Nanjing and Shanghai. The total length of the line is 300.209 km, and the construction technical standard includes: a railway passenger dedicated line, a speed target value of 300 km/h; and a ballastless track. Some sections of the newly built railway are parallel to the existing railways in Beijing and Shanghai. The parallel section of the new and old lines is approximately 170 km long, the closest distance is only 4–5 m, and a drainage ditch is shared. During the construction of the Shanghai–Nanjing Line, a foundation pit was excavated close to the existing line causing the existing line foundation to be up in the air on one side, which significantly changed the stress field and displacement field of the existing line foundation. Therefore, compared with ordinary railways, the Shanghai–Nanjing section of the Beijing–Shanghai railway has more obvious characteristics of external interference. The existing Beijing–Shanghai line, which was built in the 1960s, is one of the busiest railway lines in the country, and has been in a saturated state for a long time. With the long-term effects of the construction of the new line and the long-term dynamic load of the trains, the service status of the existing roadbed needs to be carefully evaluated.

Research on the deformation of adjacent structures caused by construction has attracted the attention of engineers and technicians; for example, research on the stability and safety of railways under tunnels, which are now common [1]. Through the real-time monitoring of parameters, such as settlement deformation and horizontal displacement of adjacent structures, timely engineering measures can be carried out according to the monitoring to ensure the stability of structures during construction [2–4]. System safety assessment can be performed through efficient probabilistic stability analysis of engineering slopes along new railway lines [5]. The author carried out deformation monitoring of the existing line foundation, mainly based on the horizontal displacement and lateral stress changes, on the Shanghai–Nanjing Intercity Railway Project [6]. The above studies are all analyzed from a static point of view; however, research on the stability of the roadbed on one side facing up under the action of the train load belongs to the category of dynamic analysis. The dynamic response of the subgrade under the action of a dynamic load has become the basis for evaluating the subgrade condition [7,8], especially under conditions where the excavation on the side of the existing line base causes the stress field of the existing line to change. The dynamic deformation is an important evaluation index reflecting the state of the subgrade [9,10]. Ye et al. [11] combined field measurements and finite element calculations of dynamic response to analyze the amplitude characteristics of dynamic stress, dynamic deformation and vibration acceleration of ballastless track road foundations, and revealed the distribution law of stress and strain in the foundation bed under train load. Han [12] studied the dynamic characteristics of the roadbed of the Beijing–Shanghai high-speed railway, and concluded that the acceleration, dynamic stress, and displacement decrease nonlinearly with the increase of depth, and the dynamic stress in the roadbed increases with the increase of train speed. Bi [13], through the combination of three-dimensional finite difference software and theory, carried out the numerical simulation of the heavy-haul railway roadbed, analyzed the dynamic response law of the multi-layer roadbed system, and obtained the lateral distribution law of the dynamic stress on the roadbed surface. In the field of subgrade dynamic test research, in the early stages of the research on the dynamic characteristics of track subgrades, the test method was mostly used, and the dynamic response law of the subgrade was obtained by statistical analysis of the test data. Okumura analyzed the field test data of eight ordinary lines in Japan, and believed that the distance from the track, the structure of the track, the type of train, the speed of the train, the length of the train, and the natural vibration characteristics of the ground are the six main factors that affect the vibration level of the roadbed. Madshus proposed a semi-empirical model of subgrade low-frequency vibration, which includes five independent influencing factors: train type, speed, distance, track

state, and the influence of structures. Roadbed dynamic tests have been conducted in China many times. Yang et al. [14] obtained the following from the actual measurement of subgrade dynamic stress and vibration: the irregularity of the line has a great influence on the subgrade dynamic stress, an increase in the driving speed can lead to an increase in the subgrade dynamic stress, and the subgrade dynamic stress and vibration acceleration show an exponential decay law in the depth direction. Mao [15] surveyed the ground vibration attenuation curves of 109 trains, and obtained empirical formulas for ground vertical and horizontal vibration attenuation consistent with the principles of vibration propagation energy diffusion and energy attenuation through composite regression. In research on the dynamic stability of roadbeds, Nie et al. [16] used the critical dynamic stress method (dynamic strength) and the dynamic shear strain method (dynamic deformation) to preliminarily evaluate the long-term dynamic stability of the red clay roadbed of the Wuhan–Guangzhou high-speed railway ballastless track. Wang [17] established a three-dimensional finite element numerical model of a ballastless track-roadbed system to simulate the running process of eight marshalled bullet trains. Combined with the measured data, the effects of track irregularity, train speed, axle load and depth on vertical dynamic stress were analyzed. Nie [18] developed the track-roadbed dynamic test system and applied it to the long-term dynamic performance tests of the dynamic response of many heavy-haul railway and high-speed railway subgrade projects.

In general, there are few studies on the dynamic performance of roadbeds. Restricted by objective conditions, most of the vehicle speeds in China's track subgrade dynamic response tests are not high, and cannot reflect well the subgrade dynamic characteristics of the trains at high speed. Due to the lack of high-speed test data, the dynamic response law of track subgrades cannot be summarized with high reliability. In addition, testing of the dynamic deformation of the subgrade requires the arrangement of measuring rods or embedded components in the subgrade drilling [19,20], which is costly and cumbersome, and is not suitable for promotion and application in the public works sector. Moreover, this type of subgrade dynamic response research is tested under the normal operating state of the subgrade body.

There are few relevant experimental studies on the dynamic testing of the existing line foundation under the influence of construction, and only numerical simulation methods can be used to analyze such problems. Bi et al. [21] used the finite element method to simulate a wheel-rail excitation load for the foundation pit construction of a nearby operating subway station in Shanghai, analyzed the dynamic response of the existing station structure during the excavation of the deep foundation pit, and used displacement and vibration acceleration to evaluate the subway's degree of impact. In addition, there are also research results on the use of numerical models to calculate the impact of foundation pit excavation on existing lines [22]; however, no experimental data were used to prove it.

In order to ensure the construction quality and construction period of the new line, and to ensure the safe operation of the existing line on the basis of static tests, this study carried out dynamic testing of the existing line foundation operation state close to the construction site. The test design takes the existing line operating trains as the vibration source, and the existing line base dynamic response as the goal to test the existing line base dynamic response; obtains the vibration displacement, frequency, acceleration and other subgrade dynamic performance indicators; and based on the test data, optimizes the construction technology and proposes construction safety measures adjacent to the existing subgrade. The test results and engineering measures provide a scientific basis for the safe operation and smooth construction of the existing line, and have important theoretical significance for guiding the construction of new subgrades adjacent to the existing line.

2. Project Overview

The nearest road section between the old and new lines was selected as the test section on the site to obtain data on the largest impact of new railway construction on the service status of the existing lines. The test section is located at DK95 + 256 of the Shanghai–Nanjing

Line, corresponding to K1274 + 128 of the existing Beijing–Shanghai Line, and is composed of silty clay with a thickness of 18–20 m underneath. The two lines are only 4–5 m away from each other and share a drainage ditch. The existing Beijing–Shanghai railway line was built in the 1970s using natural foundations. The post-construction settlement control of the newly built Shanghai–Nanjing Railway is strictly regulated, with the requirement being not greater than 15 mm. Therefore, the new pile-raft composite foundation adopted by the Ministry of Railways was introduced for soft soil foundation reinforcement. Taking into consideration that it was close to the existing line, the pile type adopted was that of static pressure construction, the pipe pile was 18 m long, the raft thickness was 0.5 m, and the gravel cushion was laid underneath. Temporary protection was provided before excavation and replacement, and a row of rail piles was installed outside the excavation range for protection. The distance between rail piles was 2.0 m, and the pile length was 5.0 m. The design section of the test section surface is shown in Figure 1.

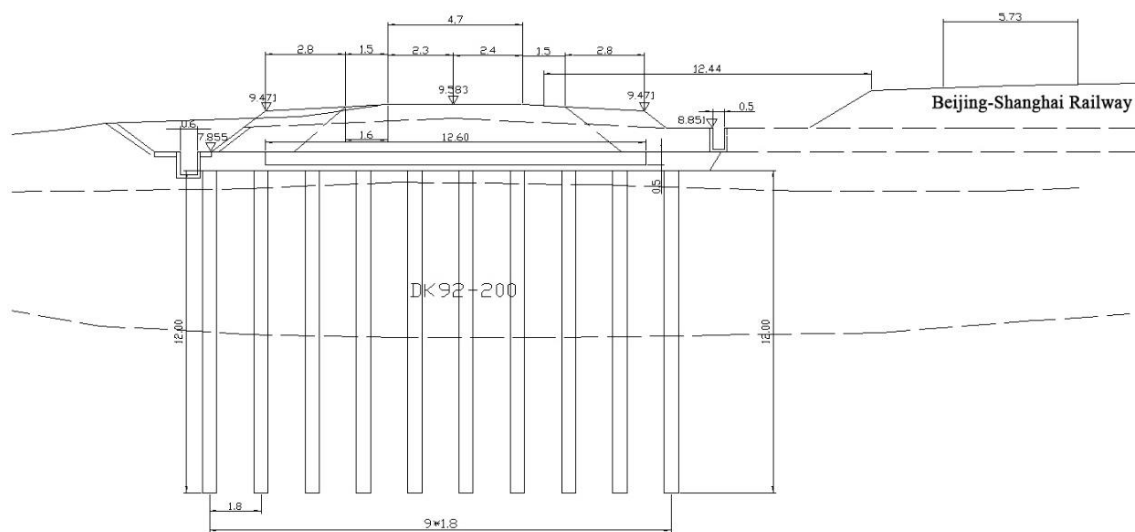


Figure 1. Schematic cross-section of two lines close to the test.

Subgrade dynamic analysis research is an important technical guarantee to reduce the damage of railway subgrades. The dynamic response of the subgrade includes the deformation, dynamic stress, acceleration, etc., of the subgrade. The magnitude and process of the dynamic response of the subgrade are directly related to the strength and fatigue characteristics of the subgrade, the cumulative deformation, and its dynamic stability [23,24]. Therefore, how to test and evaluate the stable state of the roadbed under the action of train power was the main problem to be solved.

3. On-Site Power Test

3.1. Test Content

The dynamic response of the subgrade under the action of a dynamic load (that is, a change in parameters such as vibration acceleration, vibration velocity, vibration flat rate, and vibration displacement of the subgrade) includes change over time, attenuation, and transmission and diffusion along the horizontal and vertical directions of space. The dynamic characteristics of the subgrade are exhibited by the changes in these dynamic parameters and directly affect the operation of the road. Field testing was undertaken to test the dynamic changes in these parameters over time and space under load. The dynamic test acquisition system is shown in Table 1.

Table 1. Dynamic testing of collecting systems.

Equipment Name	Quantity
Donghua DH-5935 dynamic signal test and analysis system	1
DH-5935 solid state acceleration shock pickup	4
imc16 channel dynamic signal tester	1
imc accelerometer	6
891-II type vibration sensor (horizontal, vertical)	6
RDP displacement sensor	5
Laptop	1
Desktop PC	1

The on-site in situ dynamic test mainly tested the following: (1) the distribution and changing law of soil dynamic parameters in the subgrade under a specific train operating speed; and (2) the distribution and variation law of dynamic displacement and overall displacement under different train operating speeds.

There was a large number of construction machines and personnel on the project site, and the test environment was very complex. During the test process, attention had to be paid to the safety of personnel and equipment, and various interferences in the process of power data acquisition had to be eliminated as much as possible. Combined with the actual situation on site, the overall test idea adopted an “embedded vibration measuring pile + sensor”. Between the new line raft and the existing line base guardrail, three steel pipes with different depths were buried along the line, with depths of 1, 2, and 3 m, respectively, as permanent monitoring piles. In addition, at each construction stage, a section of temporary observation piles was buried.

The tested and operated trains were mainly the “Harmony” series of power-dispersed EMUs, with two 12-car trains as the standard. During the test, a particular person was responsible for measuring the speed of the vehicle and recording the vehicle information at the beginning/continuing/end of each construction stage. In addition, the test included parameter changes in the case of natural environment excitation, construction machinery vibration, etc.

3.2. Test Methods

When the existing train passed by, the test software was started to collect the vibration data. The channel adopted 1–4 piezoelectric sensor channels, which could obtain vibration acceleration, dynamic displacement, and vibration speed. The on-site power monitoring is shown in Figure 2.

**Figure 2.** Field dynamic testing. (a) is data acquisition; (b) is data processing.

The data collection was designed according to the plan, and the sensor was fixed at the measuring point. In the cross-section direction, measuring points were arranged at different distances from the existing line to monitor the distribution law of the dynamic characteristics of the embankment soil in the cross-section direction. Acceleration sensors were arranged on the top of steel pipes with different depths to monitor the distribution of the dynamic characteristics of the embankment at different depths. Displacement sensors

were arranged at different heights of steel pipes and slopes to monitor the distribution law of dynamic displacement when the train passed by. The layout of the measuring points is shown in Figure 3.

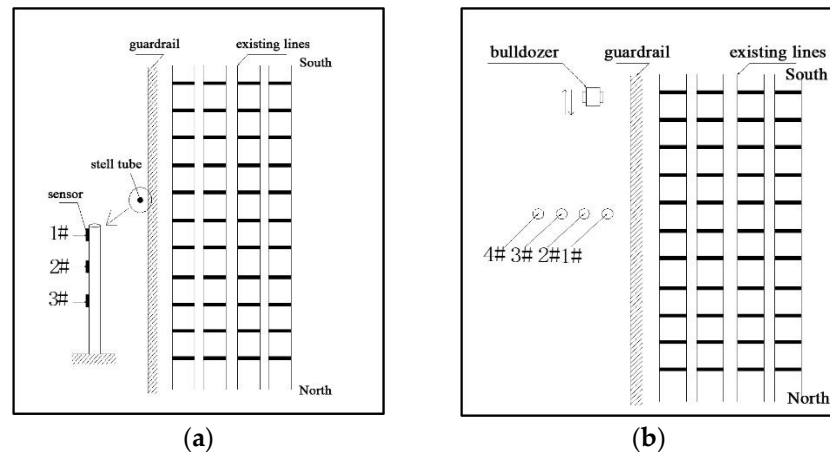


Figure 3. Monitoring of measuring points. (a) is schematic of sensor placement; (b) is schematic diagram of steel pipe laying.

4. Analysis of Test Results

According to the collation and analysis of the test results, the data were analyzed in terms of vibration frequency, acceleration, velocity, and displacement. In the current vibration research, the vertical vibration generally dominated. Therefore, the vertical vibration was used as the evaluation index for the analysis in this paper.

4.1. Vibration Frequency

4.1.1. Subgrade Natural Frequency

The natural frequency of the structure is the basis of dynamic calculations. Before the dynamic calculation, frequency extraction and analysis must be carried out to obtain the mode shape and natural frequency of the structure. The natural vibration frequency of the existing line base depends on the subgrade filler, the subgrade soil properties, and the surrounding soil conditions. The dynamic test studies of high-speed railway foundations, such as Wuhan–Guangzhou and Qinshen, show that the [17,18] high-speed railway subgrade is an engineering structure with a low natural vibration frequency. The subgrade natural vibration is mainly a low-frequency natural vibration, ranging from 0 to 40 Hz, and there are five large peaks in between, namely 11.8, 20.1, 22.6, 27.9, and 30.9 Hz, which all represent the subgrade natural frequency. In a field resonance test of an embankment conducted in Japan on the Tokai Shinkansen, the resonance frequency of the embankment was measured at 15–20 Hz.

The train wheel group forms a load series through the subgrade section, so that the subgrade is subjected to repeated loads to generate forced vibrations, and the excitation frequency changes with the train speed. When the excitation frequency is equal to or close to the natural vibration frequency of the roadbed, resonance will occur and the vibration of the roadbed will be aggravated. Therefore, during the test, attention was paid to the coincidence of the natural frequency of the roadbed and the external excitation frequency.

4.1.2. Train Excitation Frequency

The dynamic loads imposed by trains on the roadbed are complex, and mainly include: the vibration of the sprung and unsprung mass of the vehicles, the impact of the trains on the track caused by the uneven surface of the wheel and the rail, and the periodic load caused by the movement of the wheel axle load, etc. These excitations can be regarded as the superposition of harmonic components of different frequencies, in which the shaft load movement mainly produces a low frequency (0–40 Hz). The unevenness of the wheel and

rail mainly produces the middle frequency band (40–100 Hz). According to the measured data of the dynamic stress spectrum of the subgrade, the fundamental frequency (that is, the passing frequency of the vehicle) has the greatest impact on the subgrade. The self-power spectrum is obtained by filtering and eliminating the trend. The typical test spectrum is shown in Figure 4.

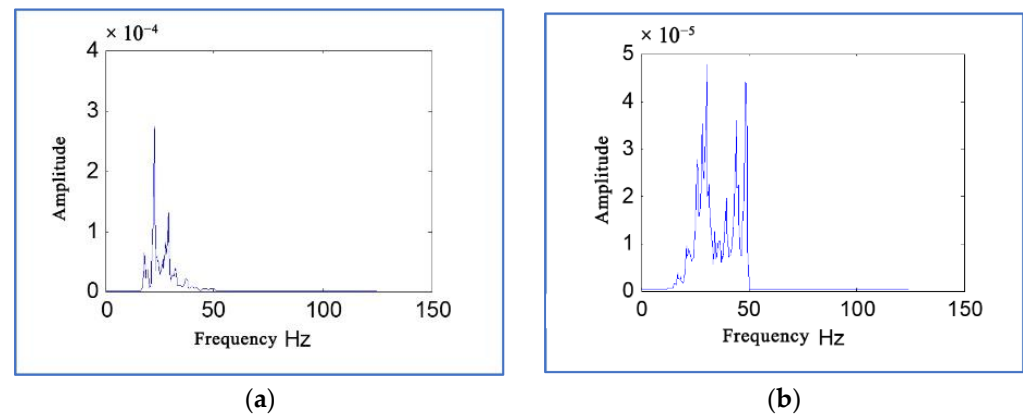


Figure 4. Testing spectrum under train vibration load. (a) is Speed 60 km/h; (b) is Speed 120 km/h.

As shown in Figure 4, the vibration frequency of the subgrade under the train load is concentrated in the 0–50 Hz range, the vibration energy at the peak value is relatively large, reflecting the load action frequency of the train, and the peak value is mainly concentrated in the 20–40 Hz range. At different speeds, the soil response frequency has two peaks, verifying the double-peak phenomenon of the dynamic response of the high-speed railway subgrade mentioned in the literature [5]. The excitation frequencies all move to the high frequency direction with the increase in speed, and the energy corresponding to the peak value increases. The in situ monitoring went through the process of pouring the bearing plate, embankment filling, and preloading in the test section. The vibration frequency of the foundation soil caused by the train load was kept at 20–40 Hz. Among these, the excitation frequency at the foundation excavation stage is relatively low, maintained at approximately 20 Hz, which is close to the natural frequency of the roadbed obtained by the Japanese test in the existing research.

The impact or vibration frequency of the axle of high-speed trains on the roadbed is determined by the speed of the vehicle and the structure of the roadbed. The test vibration frequency is close to the natural vibration frequency of the roadbed. When the natural vibration frequency of the roadbed is consistent with the action frequency of the train, resonance will occur, which will intensify the vibration of the train and the track, increase the sinking deformation of the track, and affect the stability of high-speed driving. From the perspective of vibration frequency control, the safety monitoring of the dynamic performance of the existing line base should be further strengthened, and necessary engineering safety protection measures should be taken.

4.2. Vibration Displacement

Vibration displacement is generated briefly when a train passes by. Generally speaking, train speed, axle load, line smoothness, and the properties of roadbed materials are the main factors affecting vibration displacement. However, in the case of excavation at the immediate position, the stress state of the subgrade changes greatly, and attention should also be paid to the change in vibration displacement of the subgrade. The line/vehicle vertical coupled vibration analysis model is used by Southwest Jiaotong University to calculate the subgrade dynamics. It is believed that, under the action of the train load, the elastic deformation of the subgrade is the result of the interaction of all the parts of the wheel/rail/track bed/subgrade system. It is proposed that the dynamic elastic deformation value corresponding to subgrades with differing stiffness should be controlled within

3.5 mm. According to the compaction standards and wheel load conditions proposed in the “Interim Regulations on the Design of Bridges, Tunnels and Stations on the Beijing-Shanghai High-speed Railway Line”, when the height of the embankment is 5–10 m, the elastic deformation of the subgrade surface is within the range of 1.32–2.25 mm.

As shown in Figures 5 and 6, during the construction excavation stage, the test vibration displacement was basically in the elastic stage, the vertical vibration amplitude reached 1.9 mm, and the horizontal vibration amplitude reached 0.15 mm. The maximum vibration displacement occurred in the surface soil of the slope during the excavation stage. After the embankment was filled in the later stage of the test, the vibration displacement of the foundation soil was basically in the 10–4 m range. Richart compiled data on the allowable vertical amplitude of vibration for a specific frequency, and found that when the operating frequency is 40 Hz (2400 cycles/min), the displacement amplitude limit is approximately 0.018 inches (0.457 mm). Based on this preliminary judgment, the dynamic characteristics of the foundation soil between the two lines tended to be stable.

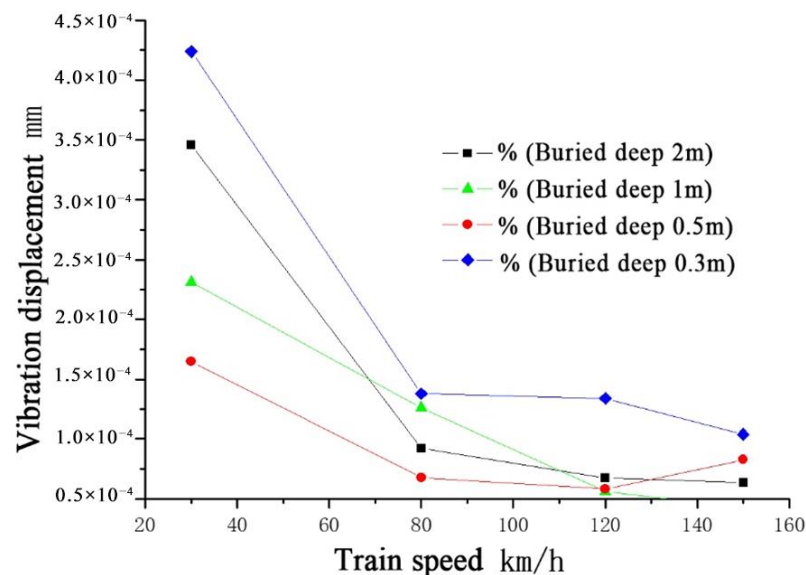


Figure 5. Foundation soil vibration displacement-train speed curve.

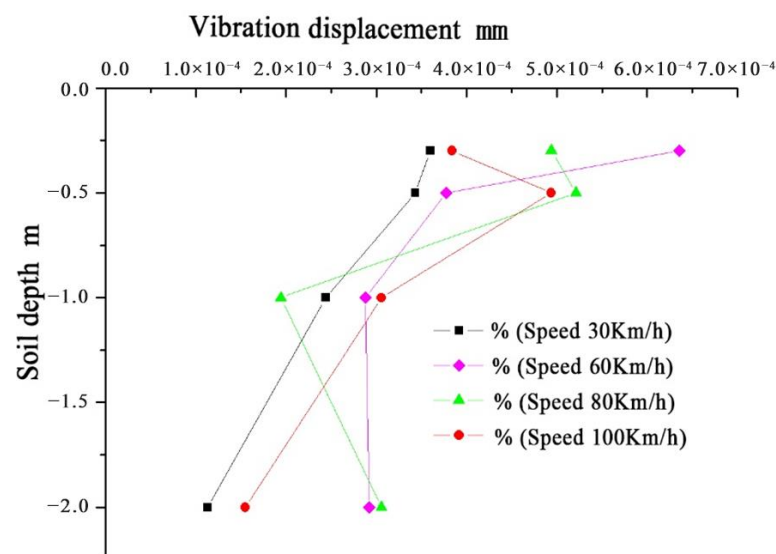


Figure 6. Foundation soil vibration displacement—depth distribution rule.

In addition, during the test period, the vertical vibration displacement of the ground had a certain attenuation along the depth direction. When the trains ran at low speed, the surface vibration mainly represented the local elastic deformation caused by the axle load, the surrounding surface fluctuation was very small, and the vibration was a quasi-static response. When the train speeds reached the foundation shear wave speed of 80 m/s, the vibration increased, the Mach effect was obvious, and the vibration influence range became larger.

The fast Fourier transform (FFT) was used to calculate the convolution integral, finding that when the excitation frequency is larger, the fluctuation of the surface displacement is larger. This can be explained as follows: after the vibration wave is transmitted to the underlying soil, it is reflected on the surface and interferes with the original wave. As the distance increases, the vertical amplitude will fluctuate to a certain extent, instead of being monotonically attenuated. This phenomenon increases with the excitation frequency, and the increase becomes more and more obvious.

As shown in Figure 7, taking the peak value of vibration displacement at the same measuring point in each construction stage, it is found that the vibration displacement generated is the largest in the stage of subgrade excavation, and the dynamic deformation gradually decreases with the completion of subgrade filling.

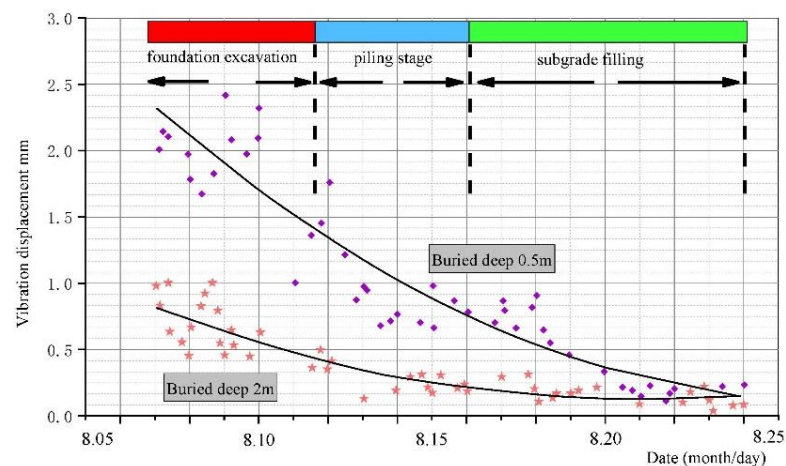


Figure 7. The vibration displacement peak trend of different construction stage vibrations.

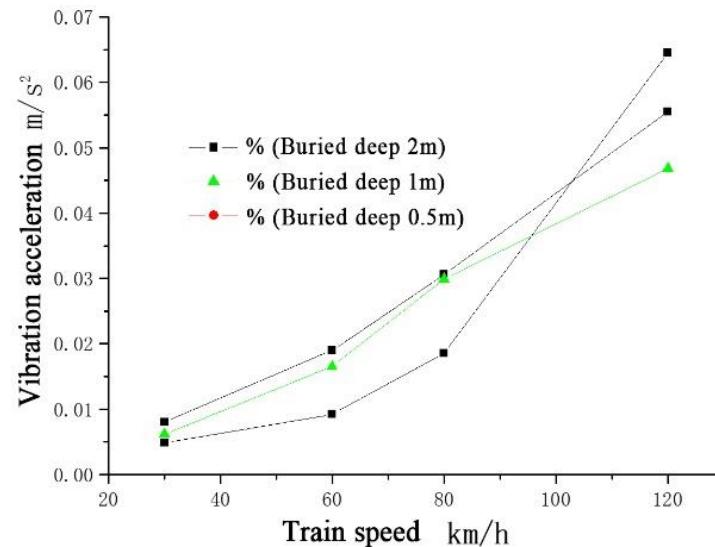
4.3. Vibration Acceleration

The magnitude of the vibration acceleration of the subgrade is the main parameter for judging the effect of vibration on track damage [16], and the amplitude of the vibration acceleration is closely related to the dynamic characteristics of the subgrade and the frequency of load action. In Japan, vibration tests have been conducted on the subgrade surface of Shinkansen high-speed trains, and the maximum acceleration is 5–20 m/s^2 . From the acceleration time–history curve, the peak periodicity of the acceleration waveform under the action of the dynamic load of the running train can be clearly distinguished.

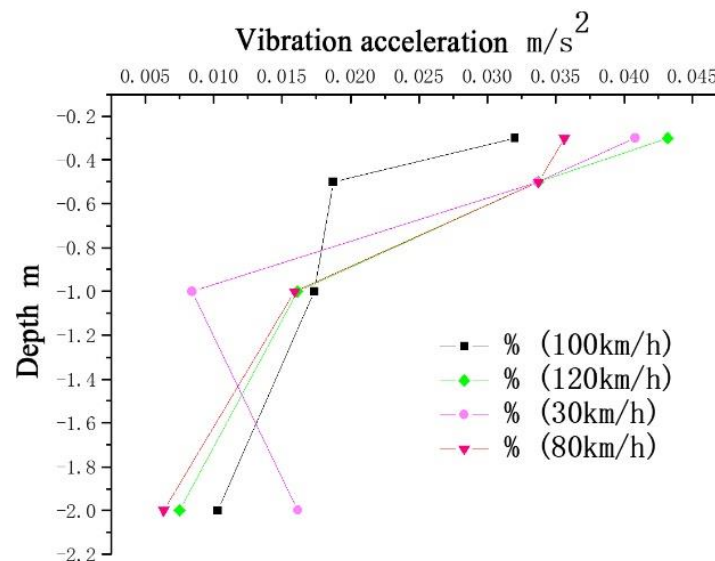
The data show that there is a certain correlation between the peak vibration acceleration, the speed of the train, and the depth of the soil layer, showing a law similar to that of dynamic stress. When the train speed was 30 km/h, the vibration acceleration peak depth range corresponding to buried depths of 2, 1, 0.5, and 0.3 m was attenuated by 67%. At a burial depth of 0.5 m, the train speed increased from 30 to 120 km/h, and the ground response vibration acceleration peak increased by 44%. The vibration acceleration distribution and vibration acceleration–train speed change curves are shown in Table 2 and Figure 8.

Table 2. Vibration acceleration distribution.

Train Speed km/h	Buried Deep (m)	Content Test	Numeric Value m/s^2
30	2	Vibration acceleration	0.01029
	1		0.01736
	0.5		0.01874
	0.3		0.03202
120	0.5		0.03375

**Figure 8.** Vibration acceleration–train speed changing curves.

The foundation soil is non-uniform and non-isotropic, and is an inelastic medium. Vibration waves propagate in the soil layer, and different types of vibration waves will be reflected and refracted when they encounter the soil layer interface. In the process of wave propagation, even a very small plastic deformation will cause energy dissipation due to the internal friction of the soil. With an increase in the distance of the train excitation vibration source, the wave energy is dissipated, and the acceleration vibration amplitude is also obviously attenuated. The vibration acceleration–depth distribution is shown in Figure 9.

**Figure 9.** Vibration acceleration–depth distribution rule.

According to the regression analysis of the measured data, the vertical and horizontal vibration acceleration attenuation formulas of the foundation soil of the Shanghai–Nanjing and Beijing–Shanghai projects can be summarized as follows. The measured vibration acceleration curve is shown in Figure 10.

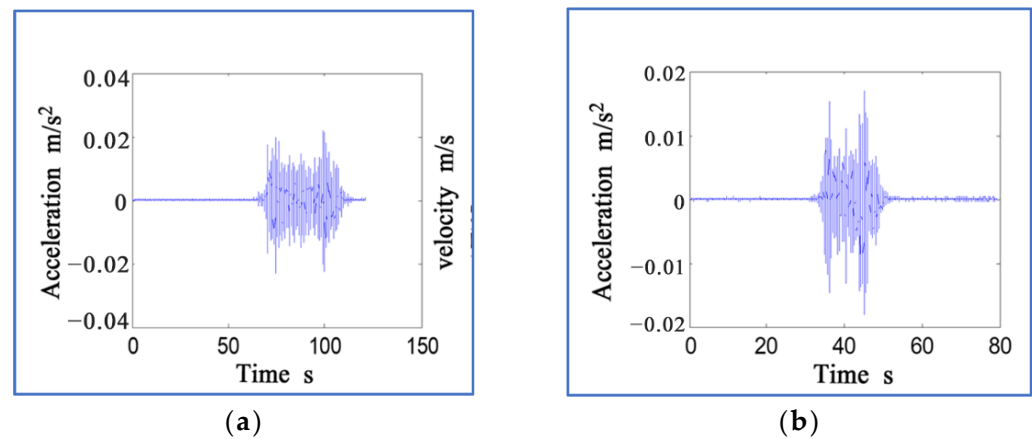


Figure 10. Testing vibration acceleration curve. (a) is Speed 60 km/h; (b) is Speed 120 km/h.

Horizontal vibration acceleration distance attenuation formula:

$$a = 0.145 \times e^{-0.20s}$$

Vertical vibration acceleration attenuation formula:

$$a = 0.031 \times e^{-0.17s}$$

The variation law of the vibration velocity distribution can be obtained by the convolution and integration of the vibration acceleration, as shown in Figure 11.

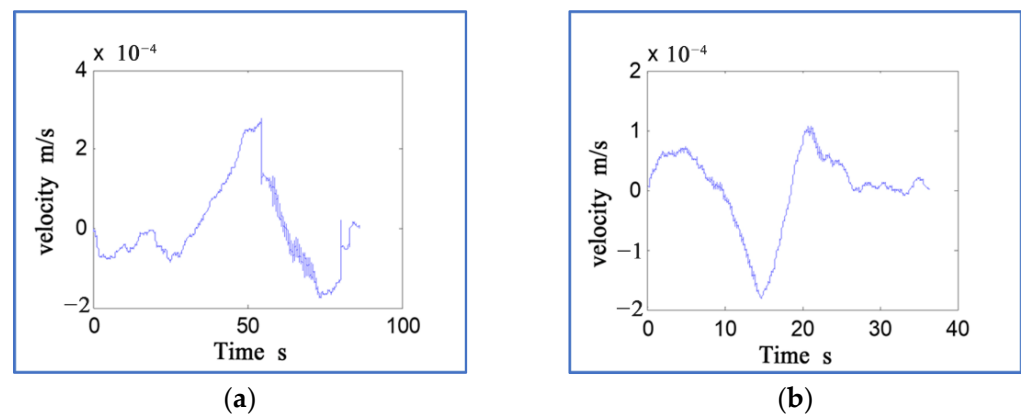


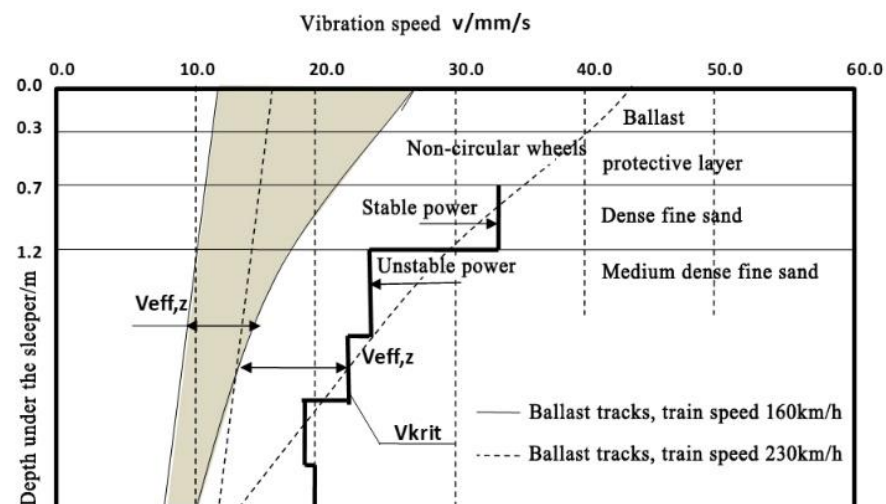
Figure 11. Testing vibration velocity curve. (a) is Speed 60 km/h; (b) is Speed 120 km/h.

Many domestic and foreign scholars or codes have given structural vibration limits based on the peak ground vibration velocity (PPV). The blasting vibration effect is shown in Table 3.

German researchers obtained the relationship between vibration speed and the stability of railway subgrade at different depths using tests, as shown in Figure 12. The critical vibration speed of coarse-grained soil is approximately $V_{krit} = 30 I_p$ (mm/s), while for mixed-grained clay soil, it is $V_{krit} = 25 I_p^{1.5}$ (mm/s).

Table 3. Blasting vibration effect.

Particle Velocity/mm·s ⁻¹	Vibration Effect
<1	Hard to feel
1	Humans can feel weak vibrations
5	Makes people feel uncomfortable, vibrate
10	Disturbing, with obvious vibration
33	Gives a strong sense of vibration
50	Safe vibration limit of general civil residential buildings
100	Safe vibration limit of reinforced concrete structure and tunnel support structure
140	Cracks in rock medium and expansion of old cracks
190	General civil buildings are severely cracked and damaged
300	Vibration and shedding of rock in unsupported tunnel
600	New cracks in rock

**Figure 12.** The dynamic stability and vibration velocity at different depths.

As peak velocity is less affected by frequency relative to peak acceleration and peak displacement, the dynamic test found that the attenuation of the vibration velocity in the horizontal amplitude is much faster than that in the vertical amplitude. The data show that the attenuation of soil vibration velocity along the depth direction is similar to the vibration acceleration, that the maximum vibration velocity of the surface is 5.04×10^{-3} m/s, and that the influence of vehicle speed on the vibration velocity is not obvious. Referring to the vibration velocity limit of the blasting vibration effect, there is an obvious vibration sense at the test site, but it is within the structural engineering safety limits.

Extracting the amplitudes of the dynamic response parameters in the three construction stages of roadbed excavation, piling, and roadbed filling, it can be seen that the vibration acceleration and vibration velocity show similar laws. That is, from the beginning of the excavation construction measurements, the vibration response tends to decrease, and the vibration response is the largest in the subgrade excavation stage. In the pile formation stage, due to the disturbance to the foundation soil, there is a slight rebound, and until the subgrade is filled, the vibration parameter gradually stabilizes. The trend of peak vibration acceleration in different construction stages is shown in Figure 13.

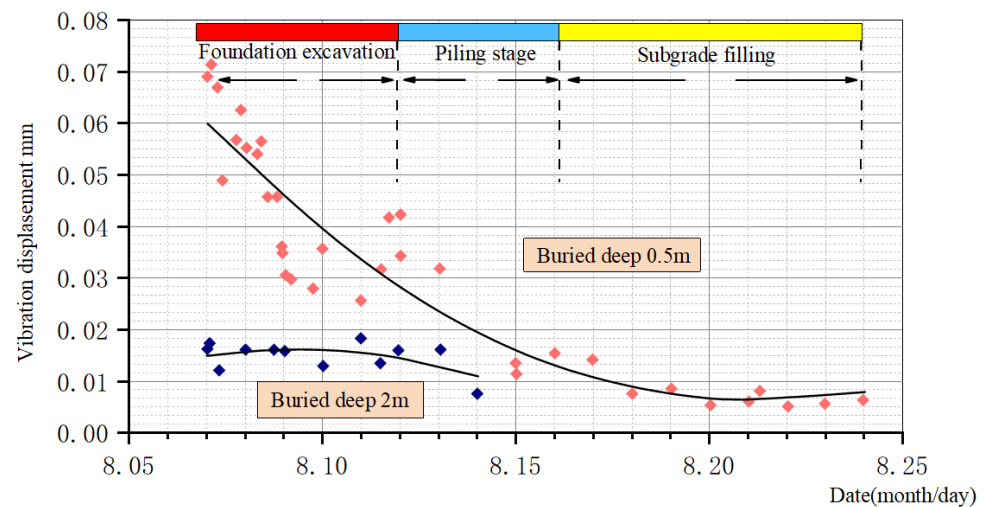


Figure 13. The vibration acceleration peak trend of different construction stage vibrations.

To sum up, the construction in the immediate position has a significant impact on the dynamic performance of the existing line foundation. Referring to the relevant regulations, the vibration displacement and vibration acceleration of the roadbed do not exceed the standard, but they specifically involve the dynamic stability of the existing line foundation. As there is no norm or experience in this area for reference, from the perspective of construction safety and existing line operation safety, close attention should be paid and construction safety measures should be adopted.

5. Test-Based Construction Safety Measures

5.1. Subgrade Excavation Stage

The stability of the roadbed was monitored through real-time dynamic tests during construction, and the test data were analyzed and fed back to the project site in real time. On one hand, protection measures for the subgrade slope were proposed, while at the same time, the road sections with abnormal stability and dangerous driving safety were found in the test to provide a basis for a reasonable opening time for the emergency construction skylight.

5.1.1. Excavation Roadbed Slope Protection

The test shows that the stability of the existing line foundation deteriorates after excavation. In addition, the exposed soil of the silty clay slope becomes looser under the influence of rain erosion and soaking, so the slope should be protected accordingly. On the project site, measures such as shotcrete plastering and hanging nets are mainly used to reinforce the excavation slope. Shotcrete plastering can effectively prevent the infiltration of rainwater and surface water, produce an embedded effect, improve cohesion, and improve the properties of the soil. Hanging nets can help strengthen the overall strength of the slope.

5.1.2. Reasonable Opening of Skylights and Determination of Construction Blocking Time

The “gap” required for maintenance and construction is reserved on the train operation map, and this gap is called the “skylight”. The skylight is the comprehensive embodiment of the coordination between construction organization and transportation organization. In order to minimize the interference of the construction blockade on the existing line transportation, the arrangement of the blockade time is a core issue.

In the construction operation in the parallel section of the operation line, the normal operation of the existing line should be maintained as much as possible; however, from the perspective of safe driving, once the test finds that the stability of the subgrade is abnormal, emergency repairs should be carried out. The dynamic test provides a data basis for determining a reasonable opening time for the skylight, the timely forecasting of

the road section with the subgrade stability exceeding the standard, and formulating the skylight opening time as soon as possible, so as to save valuable time for the transportation organization and the construction organization. The time actually occupied by the skylight consists of the following three parts: (1) the opening time of the fixed skylight; (2) the safety time reserved before and after the skylight; and (3) the additional time for the train to run after the skylight is opened.

5.2. Engineering Measures in the Piling Stage

In the Shanghai–Nanjing Intercity Railway DK92 + 700~DK93 + 515, DK95 + 404.3~DK95 + 619.54, and DK95 + 656.56~DK95 + 692, a total of 1065.62 m of subgrade foundation was reinforced with PC-A500 high-strength prestressed concrete pipe piles. The concrete strength grade was C60, the wall thickness of the pipe pile was 100 mm in a square layout, the pile spacing was 2.2~2.4 m, and the pile length was 15~37 m. The three sections of subgrade were all parallel to the existing Shanghai–Nanjing Railway, and the distance from the center of the outermost row of pipe piles to the center of the existing Shanghai–Nanjing Railway line was less than 20 m, with the nearest one being 6.8 m.

Engineering practice shows that prefabricated pipe piles are widely used because of their good bearing capacity, scale of production and construction, and easy detection of quality. The vibration, soil squeeze, and noise caused by the construction process have a negative impact on the surrounding environment; in particular, the stress waves generated by piling will cause a strong vibration of the surrounding soil, which will affect the normal use and safety of the surrounding buildings and underground facilities. A schematic diagram of the impact of piling is shown in Figure 14.

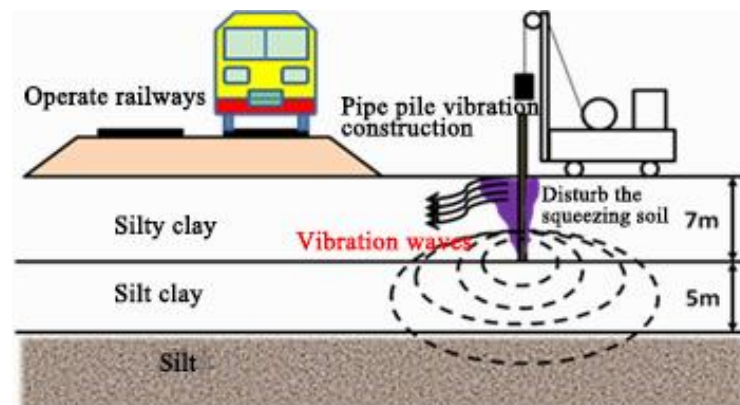


Figure 14. Piling influence diagram.

The vibration method of pile driving is similar to an earthquake or blasting. Energy is transmitted in the form of waves from the earthquake source through the rock and soil, thus causing the vibration of the foundation, existing bridges, and buildings. Although the duration of the single-click vibration is short, the amplitude and period of the impact energy load generated by the mechanical action are almost unchanged during the load action due to the small interval and the large number of strikes. According to the existing research results [25], the pile side soil can be divided into four areas according to the degree of disturbance from the pile driving process: Zone I—the hard layer area along the edge of the pile body, which is formed under the action of the extrusion force along the pile, with most of the pore water pressure in the soil being squeezed out; Zone II—known as the remodeling of the compacted zone, where the soil mass is disturbed and reshaped under the action of the compressive force, and the compaction is improved, while the shallow soil above the ground uplifts; Zone III—the disturbance zone, where the thickness is approximately 3.5 times the diameter of the pile, the average density of the soil is reduced, the water content is large, and the pore water pressure is increased; and Zone IV—the less affected

zone, where the additional stress and pore water pressure caused by the pile driving are relatively small, and the original structure of the soil remains basically unchanged.

In order to prevent the impact of piling construction on the vibration and extrusion of the existing line, it was proposed that the static pressure method be adopted to construct the prestressed pipe pile and that stress relief holes be set up on the side of the existing line with a spacing of 3 to 5 m and a diameter of 0.5 m, a depth not less than half the length of the pipe pile and not less than 10 m, and a general length of 10 to 19 m, which effectively minimizes the impact of the construction of adjacent prestressed pipe piles on the existing line. A schematic diagram of the optimization technology for pile construction is shown in Figure 15.

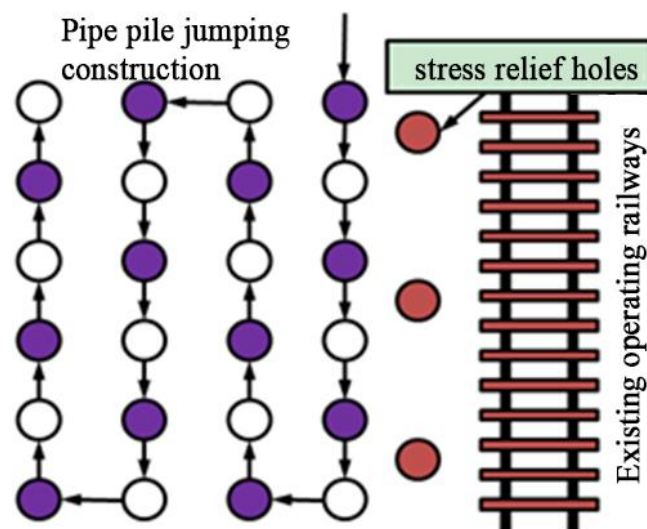


Figure 15. Pile construction optimization technology schemes.

The piling construction safety technology reduces the impact on the existing lines from three aspects:

- (1) By setting up stress release holes on the side adjacent to the existing line, the soil-squeezing effect of the pipe pile construction can be effectively released, and the continuous transfer of soil to the existing line side can be blocked to ensure the safety of the existing line.
- (2) Using static pressure construction instead of dynamic piling.
- (3) The jumping sequence is adopted to form the pile, and the purdah is formed as soon as possible.

The specific construction technology is as follows. Before constructing the pipe piles, construct stress-relief holes first, using a gravel grouting pile machine to extract the soil, with a diameter of 50 cm, laying them at 1–2 m outside the pipe pile closest to the existing line, parallel to the existing line 3–5 m, with a hole depth of 50 cm. At half the length of the adjacent piles, and not less than 10 m, use a short-screw earth-boring rig to borrow soil to form a hole, filling it with gravel over time after the drilling is completed. After the construction of the stress relief holes is completed, the pipe pile is then constructed. Its setting enables a drainage channel to dissipate the excess pore water pressure generated by the construction of the pipe pile. When constructing the pipe piles, the same row of jumping is used; that is, the No. 1, 3, 5, 7, etc., piles are constructed first, and the No. 2, 4, 6, 8, etc., piles are constructed after 24 h of no change from roadbed observation and rail surface observation. The second row of piles and the third row of piles are constructed according to this method, so that the constructed pipe piles on the side of the existing line form a pile group enclosure, suppress the effect of soil squeeze on the side of the existing line, and provide sufficient dissipation time for the excess pore water pressure.

After the construction was completed, the displacement test showed that the construction of the adjacent pipe piles did not cause obvious extrusion deformation of the existing line. Due to the use of the static pressure construction method, there was no adverse vibration effect.

6. Conclusions

In this study, combined with a passenger-dedicated line construction project, experimental research on the dynamic stability of an existing line foundation under the influence of a new line construction was carried out, and the construction technology was optimized according to the experimental analysis. The following conclusions were obtained:

- (1) A set of dynamic performance testing methods for railway subgrade in operation are proposed to obtain the dynamic performance indicators of the subgrade, such as vibration displacement, vibration frequency, and vibration acceleration of the existing subgrade under the influence of construction.
- (2) During testing, the peak value of the vibration response parameter in the subgrade excavation stage was the largest, which was the most dangerous stage of the subgrade dynamic stability. Subgrade vibration acceleration as a common index of structural dynamics has similar standards to follow, which can directly reflect the dynamic performance of the test body, and vibration displacement can be used as a dynamic deformation condition to evaluate the dynamic stability of the subgrade. Therefore, it is suggested that these two parameters be used as the dynamic stability control index of the existing roadbed under the influence of excavation.
- (3) During testing, the excitation frequency of the train to the subgrade is close to the natural frequency of the railway subgrade, and the possible resonance will lead to the aggravation of the subgrade vibration. This situation mainly occurs in the foundation excavation stage, and it was found that it is affected by the changes in the foundation stress field and displacement field. Based on this, it is suggested that the engineering construction department take preventative measures, such as strengthening the slope and speeding up the construction.
- (4) Dynamic testing provides a forecast and an early warning for the stability of the subgrade slope, and supplies data support for the protection of the subgrade slope and the reasonable opening of skylights. The piling construction technology is optimized from the perspective of subgrade dynamic stability, and the adverse effects, such as the extrusion and vibration of the adjacent existing line foundation, are eliminated by setting up stress relief holes, static pressure pipe piles, and jumping construction.

The test results provide data support for the safety evaluation of the existing subgrade. The construction safety measures proposed based on the test results ensure the safe excavation of the adjacent existing subgrade, which has important theoretical significance for guiding the construction of new subgrades adjacent to the existing subgrades in the future.

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Article

Experimental Study on Performance of Modified Cement-Based Building Materials under High-Water-Pressure Surrounding Rock Environment

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Abstract: Traditional cement-based grouting materials have good reinforcement and anti-seepage effects on the surrounding rock under normal conditions, but the grouting effect is not ideal due to problems such as a long setting time, a low stone ratio, and poor crack resistance under high water pressure and in a dynamic water environment. In this study, we aimed to improve the physical properties, chemical properties, and microstructure of a cement-based slurry by forming a hydrogel through its chemical crosslinking with polyvinyl alcohol and boric acid as modifiers for the purpose of improving the permeability resistance of the surrounding rock grouting under high-water-pressure conditions, which can expand the function of traditional building materials. The grouting effect of the modified cementitious material on the surrounding rock was analyzed through indoor tests, the SEM testing of the performance of the modified slurry, the numerical calculation of the seepage field, and the application of the modified slurry in combination with the actual project to verify the water-plugging effect. The research findings demonstrate that (1) the additives boric acid and PVA can significantly speed up the slurry gel time, and the gel time can be controlled within 2–20 min to meet the specification requirements. (2) At a velocity of moving water > 1 m/s, the retention of the solidified modified slurry stone body reaches more than 80%. According to the SEM analysis, the structure of the solidified modified slurry stone body is dense and has good impermeability. (3) According to the numerical calculation analysis, the modified slurry can effectively change the seepage field of the surrounding rock and improve its seepage resistance. The water pressure outside the lining is reduced by 47%, 31%, and 22%, respectively, compared with no slurry, the pure cement slurry, and cement–water–glass grouting, and the indoor test and numerical simulation conclusions are consistent.

Keywords: cement-based material; polyvinyl alcohol; boric acid; impermeability

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1. Introduction

With the rapid development of science and technology, building materials are increasingly widely used [1,2]. To increase permeability and protect the lining structure and the environment, the surrounding rock of the tunnel can be strengthened by grouting. Traditional grouting materials have good water-plugging and reinforcement effects in static environments, but the disadvantages of a slow setting time, dispersion resistance, and weak crack resistance of the traditional slurry will lead to a poor grouting effect in high-water-pressure and moving environments [3–6]. Therefore, an important research goal is to improve traditional cement-based grouting materials to meet the grouting requirements under high water pressure and in a dynamic water environment and to improve the application range of building materials.

The tunnel grouting material in the high-water-pressure surrounding rock environment needs to have good impermeability and injectability, and tunnel grouting materials have been studied by several researchers. Hua and Xie [7] conducted an indoor test to determine the fluidity, water barrier properties, and compressive strength of fly ash and a cement slurry and an engineering practice test to determine the grouting ratio. Zhang [8] analyzed the applicability and grouting effect of a single cement slurry, a double slurry composed of a cement slurry and water-glass slurry, and a fine cement slurry under different working conditions of karst tunnel curtain grouting in Gele Mountain, and the analysis showed that the double slurry grouting material composed of a cement slurry and water-glass slurry is more effective. Tian et al. [9] examined how fly ash, the water-reducing agent, and the water–cement ratio affected the grouting slurry’s flowability, setting time, and compressive strength. Through optimization, a grouting slurry with a high degree of compatibility was created. Li et al. [10] developed a new grouting material with a good grouting capacity, good water retention, and high early strength. The results show that the elastic modulus, grouting volume, and pressure of the material all have an impact on ground settlement, according to the application results of a cross-river shield tunnel grouting project. Through indoor tests, Peng et al. [11] examined the effects of the water-glass admixture, boehmite content, and powder ratio on the slurry’s performance. It was found that, compared with traditional materials, the double slurry has the advantages of a controlled setting time, high compressive strength, and environmental protection and is better used in river bottom tunnel grouting work. Zhang et al. [12] used bentonite and cement as base materials with a curing agent to obtain excellent flow, stability, and scouring resistance, which can be applied to meet engineering requirements in a moving water environment while being cost-effective and environmentally friendly. Zhang et al. [13] developed a grouting substance using cement, clay, meta-aluminate, and lignin. Indoor tests revealed that the substance has a quicker gel time, a lower exudation rate and volume shrinkage, a higher viscosity and anti-dispersion effect, and a better underwater grouting effect.

Through indoor tests, Zhu et al. [14] analyzed a slurry composed of cement, clay, and fly ash and tested its mechanical and rheological properties. The results show that the grouting material has good mechanical properties and impermeability. Liu et al. [15] prepared a kind of cement based on magnesium phosphate. The indoor test showed that GGBS significantly shortened the setting time, improved a certain amount of fluidity, produced a compact microstructure, and was environmentally friendly. Wan et al. [16] proposed water glass and bentonite as grouting materials and tested the mechanical properties and microscopic properties of the materials. The results indicate that the grouting materials have good viscosity. Wang et al. [17] studied the effect of PVA fibers on the mechanical properties and durability of rubber concrete through laboratory tests. The results show that PVA can significantly improve the impermeability and durability of rubber concrete. Zhang et al. [18] prepared a new type of double-fluid grouting material and analyzed the setting time, expansion rate, and mechanical strength characteristics of the grouting material through laboratory tests to meet the requirements of the grouting environment. Sun et al. [19] analyzed the influence of different proportions of organic materials such as fly ash, clay, waterborne polyurethane, and epoxy resin on the grouting performance of a cement slurry through a single-factor test, orthogonal test, and organic compound modification test. The results indicate that the addition ratio of clay and fly ash has a strong influence on the strength and stone ratio of the cement slurry.

Through indoor tests, Liu et al. [20] researched the effect of the PVA content on the performance of fly-ash-based cement grouting materials. It had a significant effect on the mechanical properties of the grouting material and could effectively improve the rock mass’ hydraulic coupling characteristics. The impermeability of PVA to the grouting material was markedly improved, and it increased with increasing amounts of PVA content. Lu et al. [21] proposed porous sand, PVA, cement, and rubber grouting materials. The density, consistency, fluidity, exudation rate, consolidation shrinkage rate, setting time, and

unconfined compressive strength of the slurry were tested in the laboratory. The results showed that the grouting material has good toughness and high strength.

In order to design grouting materials, many researchers have combined different grouting projects with their findings, obtained specific results based on the performance of cement bases, and added modified materials to the pure cement slurry to achieve the desired grouting anti-seepage and reinforcement effects under various working conditions.

However, there are still some issues within the current literature, such as the slurry's low stone solidification retention ratio, low compressive resistance, and insufficient crack resistance, which will result in a poor water-plugging effect when it is exposed to high water pressure or a moving water environment. In recent years, polyvinyl alcohol and boric acid have become increasingly applied modified materials, which are used to rapidly induce gelation via chemical crosslinking at certain concentrations. This mechanism improves the properties of cement-based materials so as to meet the grouting requirements under high water pressure and in moving water environments. In this study, an indoor test was performed to examine the effects of parameters such as the water–cement ratio, the concentration of polyvinyl alcohol, and the concentration of boric acid on the properties of the slurry setting time, stone solidification retention ratio, and compressibility in a moving water environment against the backdrop of the surrounding rock under high water pressure. Combined with scanning electron microscopy, the microstructural characteristics of the modified material's stone body were analyzed to verify whether the modified slurry achieved design expectations. Finally, the water-plugging effect of the cement–polyvinyl alcohol–boric acid slurry was examined by numerical calculations using a real-world engineering application as an example, and the consistency between the results of the indoor test and the numerical calculation was verified.

The main purpose of this work was to analyze the liquid properties of the grouting material, the properties of the stone body, and the water-plugging properties of the whole grouting ring after grouting through laboratory tests and numerical simulations. Further, tests were performed to verify whether the modified cement-based materials can meet the requirements of the surrounding rock grouting in a high-water-pressure environment from the characteristics of the liquid and solid material and the engineering application process in three stages. The results of this study can provide a comprehensive understanding of modified grouting materials in engineering practice.

2. Basic Performance Study of Single Cement Slurry

The primary component of composite grouting materials is the single cement slurry, which is enhanced by modified materials to meet the needs of various engineering environments, including requirements for grouting fluid impermeability and injectability [22]. As a result, the performance of the composite grouting material is directly influenced by the fundamental performance of the single cement slurry, and the following indoor test study was conducted to determine this fundamental performance.

2.1. Test Materials and Procedure

Since the water–cement ratio affects the primary characteristics of the common cement slurry, the pure cement slurry is primarily composed of silicate cement with different water–cement ratios (W/C). The pure cement slurry was prepared with five different water–cement ratios: 0.5:1 (50 g water, 100 g cement), 0.8:1 (80 g water, 100 g cement), 1:1 (100 g water, 100 g cement), 1.5:1 (150 g water, 100 g cement), and 2:1 (200 g water, 100 g cement). Each group of material tests was conducted three times, and the data in the figures in the manuscript are the average values of the three groups of tests.

Figure 1 shows the precise results from a rotary viscometer (Shanghai Jinglai Electronic Technology Development Co., Ltd., Shanghai, China) used to measure the viscosity of the cement slurry at various water–cement ratios. A viscometer (Shanghai Leiyun Test Instrument Manufacturing Co., Ltd., Shanghai, China) was used to determine the initial and final setting times of the cement slurry at various water–cement ratios, and the precise

results are displayed in Figure 2. For standard curing, the cement slurry was formed into cubic specimens. Uniaxial compressive strength tests were then carried out at 3 d, 7 d, 14 d, and 28 d [23]. The results are shown in Figure 3. To calculate the stone solidification ratio, 100 mL of each type of slurry with one of the five water–cement ratios was placed in a measuring cylinder and left for 3 h. The liquid level was then read and recorded.

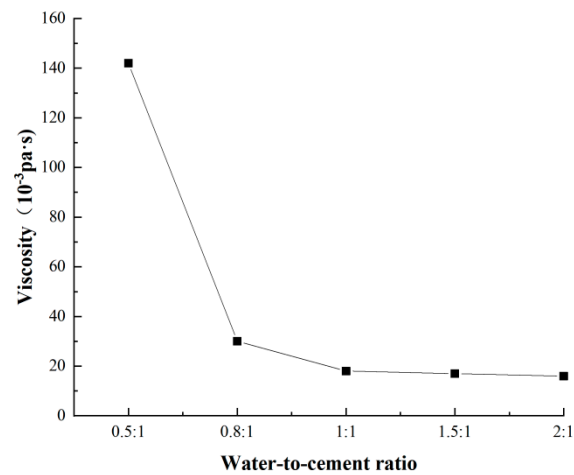


Figure 1. The influence of the water–cement ratio on the viscosity of the slurry.

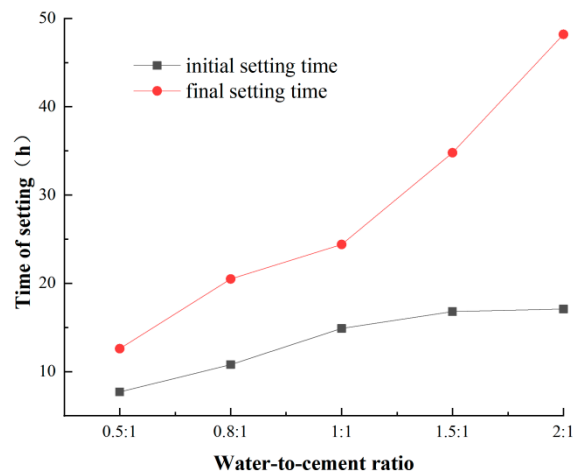


Figure 2. The influence of the water–cement ratio on the slurry setting time.

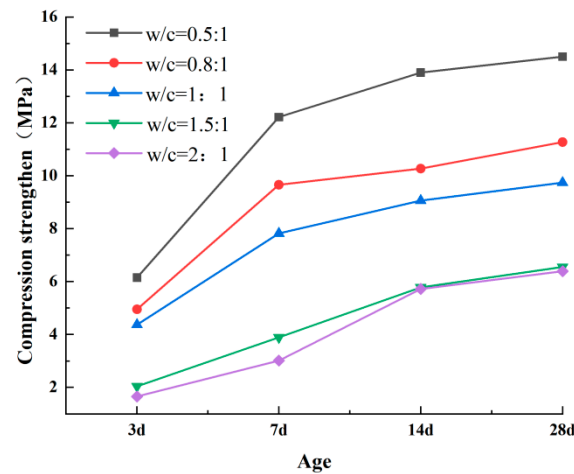


Figure 3. Compressive strength of the solidified slurry stone body.

Since the groundwater has a certain flow rate under the effect of high water pressure, the retention ratio of the stone body solidification of the slurry in a static water environment and a moving water environment after grouting must be taken into account in order to simulate the actual grouting environment. Cement slurries were prepared with water–cement ratios of 0.5:1, 1:1, and 2:1. After the initial setting of the material, it was placed on inorganic glass and in a homemade inorganic glass sink with a flow rate of 0.2, 0.4, 0.6, 0.8, 1.0, or 1.2 (m/s). The stone solidification retention ratio of the slurry was measured after 30 min in static water and moving water. The test primarily analyzed the water–cement ratio and water velocity on the slurry, combined with the anti-dispersion impact, and the test results are shown in Figures 4 and 5.

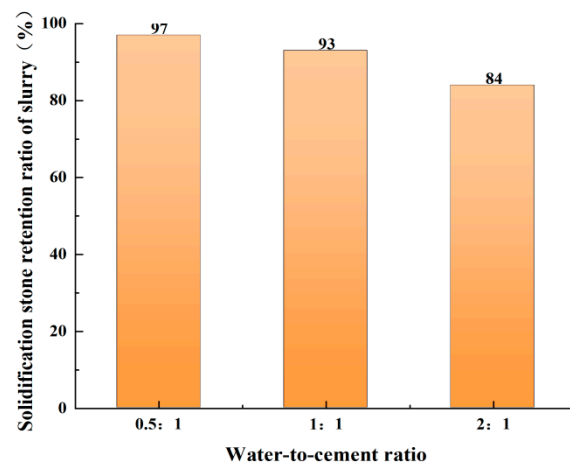


Figure 4. Stone solidification retention ratio of slurry in static environment.

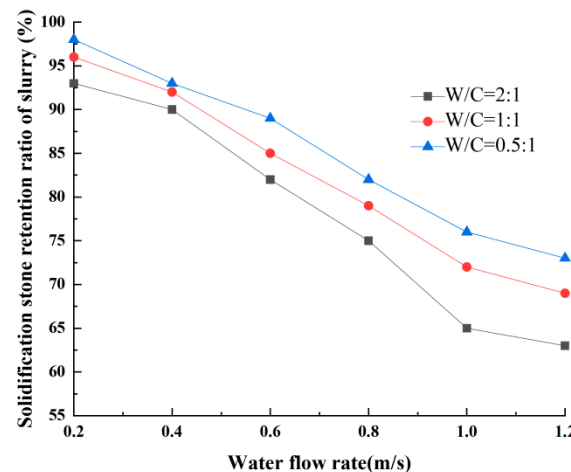


Figure 5. Stone solidification retention ratio of slurry in moving water environment.

2.2. Analysis of Experimental Results

According to Figure 1, as the water–cement ratio rises, the concentration of a single cement slurry decreases. The viscosity of the cement slurry decreases significantly from 140×10^{-3} to about 40×10^{-3} , or by about 70%, when the water–cement ratio is between 0.5 and 1.0. When the water–cement ratio is greater than 1, the viscosity of the slurry tends to reach a plateau and hardly change. The change is primarily caused by the relatively high water–cement ratio, which causes a complete reaction of cement hydration and prevents the viscosity from rising as the water ratio rises.

The water–cement ratio has a greater impact on the setting time of the pure cement paste, as shown in Figure 2. The initial setting time increases from 7 h to around 15 h as the water–cement ratio rises, and the final setting time rises as well. The initial to final setting

time interval of the pure cement slurry varies with the water–cement ratio; for example, for a water–cement ratio of 0.5, the interval is approximately 5 h, whereas for a water–cement ratio of 2.0, the interval is approximately 30 h. This difference is primarily due to the higher cement content, which causes the slurry to set faster.

The solidification strength of the pure cement slurry is also influenced by the water–cement ratio, as shown in Figure 3, and the greater the compressive strength at the same age, the smaller the water–cement ratio. The strength growth is faster for the four water–cement ratios from 3 d to 7 d, and the solidification strength increases gradually after 7 d. The solidification strength of the cement slurry varies significantly depending on the water–cement ratio; for example, the solidification strength of a water–cement ratio of 0.5 is approximately 15 MPa, while that of a water–cement ratio of 2 is only approximately 6 MPa, a difference of approximately 2.5 times [24]. The solidification ratio of the pure cement slurry, which can reach about 99% with a high solidification ratio for all five water–cement ratios, is essentially unaffected by the water–cement ratio.

Figure 4 shows that the slurry with a water–cement ratio of 0.5 had the highest combined retention ratio of 97%, and the slurry with a water–cement ratio of 2 had the lowest combined retention ratio of 84% after being placed in still water for 30 min after the initial setting. This shows that the slurry with a high cement content had a better cementing effect in still water. Following the initial slurry setting with various water–cement ratios, Figure 5 illustrates the retention ratio of slurry stone solidification in moving water at various water velocities. As can be seen in the figure, the retention ratio of stone solidification decreases with the increasing moving water flow velocity and increases with the decreasing water–cement ratio at the same flow velocity. The stone solidification retention ratio for each water–cement ratio at a 1.2 m/s flow velocity is only 60%–70% of the stone solidification retention ratio at a 0.2 m/s flow velocity according to this law, which states that the slurry needs to have a certain carrier dispersion to ensure the grouting effect in the moving water environment. The grouting fluid should have better anti-dispersion properties under high-water-pressure conditions because the slurry is easily dispersed in the moving water environment compared to the static water environment.

3. Basic Performance Study of Cement–Polyvinyl Alcohol–Boric Acid Slurry

The pure cement slurry has good viscosity and a high ratio of solidified stone formation, but it also has a long setting time, numerous brittle material pores in the formed stone body, a low ratio of stone retention under high water pressure, and insufficient durability. Under high water pressure and with enhanced permeability, the pores of the solidified slurry stone body will further open up and form cracks, which will greatly reduce the water-plugging effect and prevent the expected grouting effect from occurring [25–27]. The use of the polyvinyl alcohol and cement slurry mixture, owing to the polyvinyl alcohol hydrogel's anti-cracking properties, enhances the performance of the grouting solution to achieve the expected effect of grouting. This improves the cement slurry setting time, improves the crack resistance of the grouted stone body, and enhances the durability of the grouting material [28,29].

3.1. Effect of Polyvinyl Alcohol Concentration on Gelation Time

Polyvinyl alcohol, or PVA, is a type of polymer with good water solubility that deforms well and has good toughness once it has gelled into a film. The PVA solution and cement liquid mix well, creating a gel film with good plasticity that also protects the cement from corrosion and increases the durability and crack resistance of the stone body [30–32].

Polyvinyl alcohol is insoluble in water at room temperature. Therefore, in this test, in order to dissolve it in water, it was heated and stirred with a magnetic stirrer. The temperature was set at 80 °C, and the solution was stirred for twenty hours. Different concentrations of the PVA solution and cement slurry volumes were mixed to configure PVA aqueous solutions with concentrations of 1%, 2%, 3%, 4%, and 5%, and the volume ratio of cement to PVA solution was 4:1. The inverted cup method was used to measure the

water–cement ratio of the PVA solution with various concentrations and the time it took for the double-slurry gel to form. The specific results are shown in Table 1.

Table 1. Gel times of double slurries with different concentrations and water–cement ratios.

Concentration	W/C				
	0.5:1	0.75:1	1:1	1.5:1	
1%	189	654	1108	1598	
2%	174	612	1097	1574	
3%	154	563	1006	1528	
4%	121	487	987	1503	
5%	97	425	945	1458	

Table 1 shows that the gel time of the cement–polyvinyl alcohol slurry ranges from 97 to 1458 s at various water–cement ratios and polyvinyl alcohol concentrations, all within a half-hour, which essentially satisfies the requirements for the gel time of the grouting material in grouting construction. The gel time decreased by up to 50% with an increase in the polyvinyl alcohol concentration at the same water–cement ratio, indicating that the polyvinyl alcohol concentration has a greater influence on the gel time. At the same polyvinyl alcohol concentration, the gel time increases with a higher water–cement ratio, in line with the law of change of the pure cement slurry.

3.2. Effect of Boric Acid on Cement–Polyvinyl Alcohol Slurry Properties

The cement–polyvinyl alcohol slurry was mixed with a boric acid solution to speed up the gelation process. As a type of chemical crosslinking agent for polyvinyl alcohol, boric acid is very sensitive to a PVA aqueous solution and can shorten the time it takes for the solution to gel and turn into a hydrogel. This meets the requirement for a slurry's controllable setting time under high water pressure.

In order to easily analyze the influence law of boric acid on the gel time of the cement–polyvinyl alcohol slurry, a 5% PVA solution concentration was used, with the ratio of the cement solution volume to the PVA solution volume = 4:1, and the boric acid–polyvinyl alcohol ratios were taken as 0.05, 0.1, 0.15, and 0.2. The gel times of different slurries were measured by the inverted cup method. At the same time, compressive tests were conducted on the solidified stone bodies at the age of 3 d for different slurries. The results obtained are shown in Figures 6 and 7.

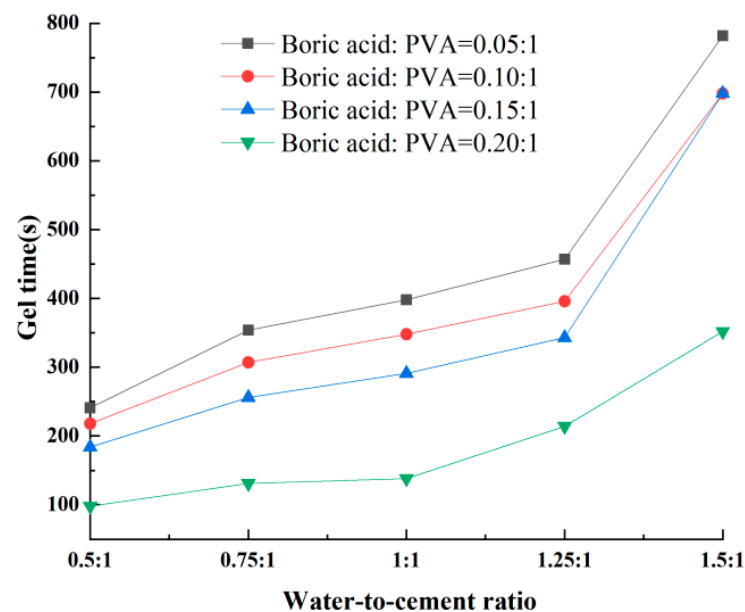


Figure 6. The influence of the water–cement ratio and boric acid on the slurry setting time.

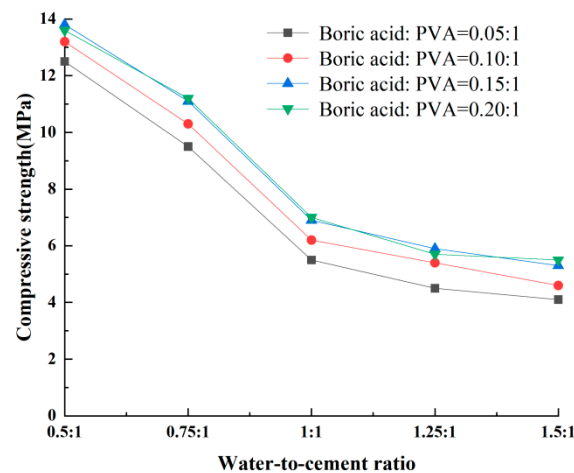


Figure 7. The influence of the water–cement ratio and boric acid on compressive strength.

As can be seen from Figure 6, when the ratio of boric acid to polyvinyl alcohol is less than 0.15, with an increase in the ratio, the time needed for the cement–polyvinyl alcohol slurry to gel is significantly shortened. PVA is very sensitive to boric acid, and a gel is formed between them through chemical crosslinking to accelerate the setting of the cement slurry. It can be seen from Figure 7 that both the water–cement ratio and boric acid ratio affect the compressive strength of the stone body. At the same water–cement ratio, the compressive strength increases with the increase in the boric acid ratio. When the boric acid ratio is higher than 0.15, the compressive strength increases slowly. BA ionizes $B(OH)_4$ in water, and $B(OH)_4$ crosslinks with the hydroxyl group on the PVA molecular chain to form a stable six-membered ring structure. Due to chemical crosslinking between boric acid and PVA, the compressive strength increases. The reaction process is shown in Figure 8.

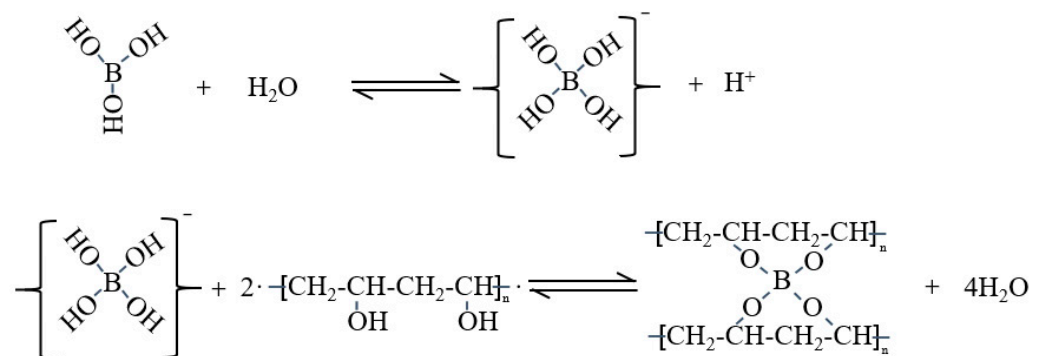


Figure 8. Reaction process of BA and PVA.

3.3. Effect of Polyvinyl Alcohol Concentration on Stone Formation Ratio

A cement solution with a water–cement ratio of 1:1 was mixed with 1%, 2%, 3%, 4%, and 5% polyvinyl alcohol solutions to form five cement–polyvinyl alcohol solutions with a volume ratio of cement solution to PVA solution of 4:1. In order to calculate the stone solidification ratio, 100 mL of each of the five water–cement ratio slurry mixtures was placed in a measuring cylinder. After three hours' rest, the liquid height was read and recorded. The overall stone solidification ratio is high, almost 100%. Figure 9 displays the results of measuring five solutions on a rotational viscometer to determine the viscosity of various slurries. From this figure, it is illustrated that the slurry viscosity generally increases with the concentration, but at concentrations greater than 3%, this growth is constrained. This is primarily due to the spatial grid structure that the solubilizer has created in the slurry, which no longer increases with the concentration.

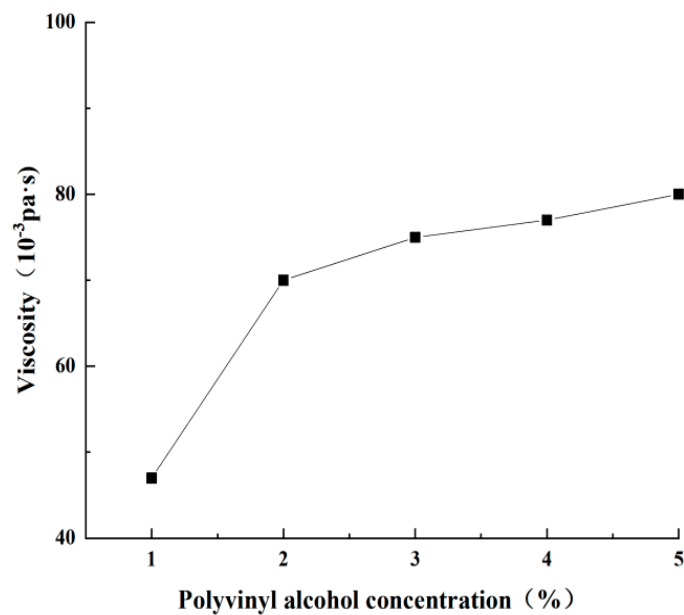


Figure 9. Effect of polyvinyl alcohol concentration on the slurry viscosity.

3.4. Cement–Polyvinyl Alcohol Slurry’s Moving Water Retention Ratio

A cement solution with a water–cement ratio of 1:1 was mixed with polyvinyl alcohol solutions with concentrations of 1%, 2%, 3%, 4%, and 5%. The ratio of the cement liquid volume to polyvinyl alcohol solution volume was 4:1, creating five different cement–polyvinyl alcohol solutions. In order to measure the retention ratio of solidified slurry stones after 30 min and analyze the slurry’s binding resistance to dispersion, the materials were first placed on inorganic glass and then placed in hydrostatic water and a homemade water tank with a specific flow rate. The results are shown in Figure 10.

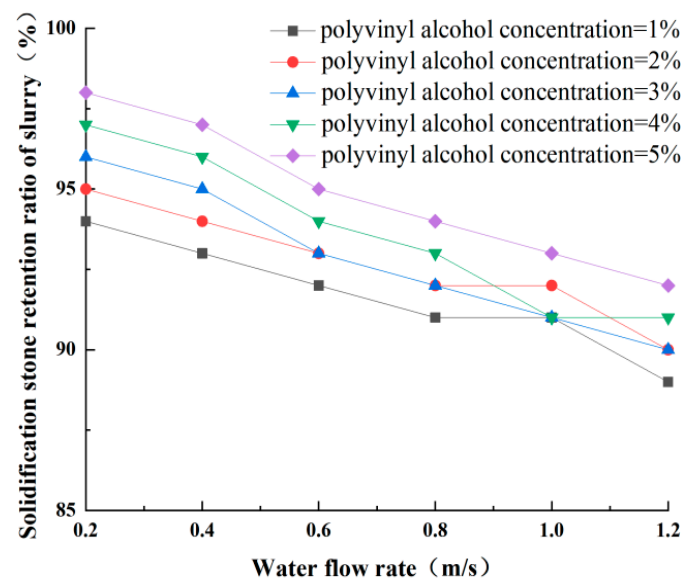


Figure 10. Effect of water flow rate on slurry solidification ratio.

Figure 10 demonstrates that the influence of different polyvinyl alcohol concentrations on the slurry’s moving water retention rate is less, but the higher polyvinyl alcohol concentration results in a higher stone solidification retention ratio. In addition, the retention of the cement–polyvinyl alcohol slurry solidification ratio is higher in a moving water environment. This is primarily because the higher the polyvinyl alcohol concentration, the

greater the gel formation and the higher viscosity of the slurry, which results in a higher retention ratio in the moving water environment.

3.5. Scanning Electron Microscope Analysis

Information on the solidified cement slurry and cement–polyvinyl alcohol slurry stones was obtained, as shown in Table 2, and a scanning electron microscopy test was carried out to observe the crack characteristics of both solidified slurry stones from a microscopic perspective. This was carried out to confirm that the crack resistance of cement–polyvinyl alcohol slurry nodules was improved [33]. A cold field-emission scanning electron microscope, model SU8010 from Hitachi, Japan, was used for the test. In order to conduct the test, samples from two solidified slurry stone bodies were first taken and pasted on conductive tape. The specimens were then placed in the apparatus for vacuum treatment and gold spraying. Finally, the specimens were scanned under a microscope to see the microscopic cracks.

Table 2. Preparation of slurry for electron microscope scanning.

No.	Slurry Type	Water–Cement Ratio	Additive Volume Ratio	External Additive Concentration	Age
A01	Cement slurry	1:1	/	/	3 d
A02	Cement–polyvinyl alcohol	1:1	4:1	5%	3 d

As seen in Figure 11a,b, the solidified cement slurry stones have more pores, which are randomly distributed throughout the stones. The porous and brittle nature of cement-based materials causes them to further develop when exposed to high water pressure, resulting in larger pores and the formation of fissures, which reduce the slurry stone body's bearing capacity and durability and interferes with the grouting effect.

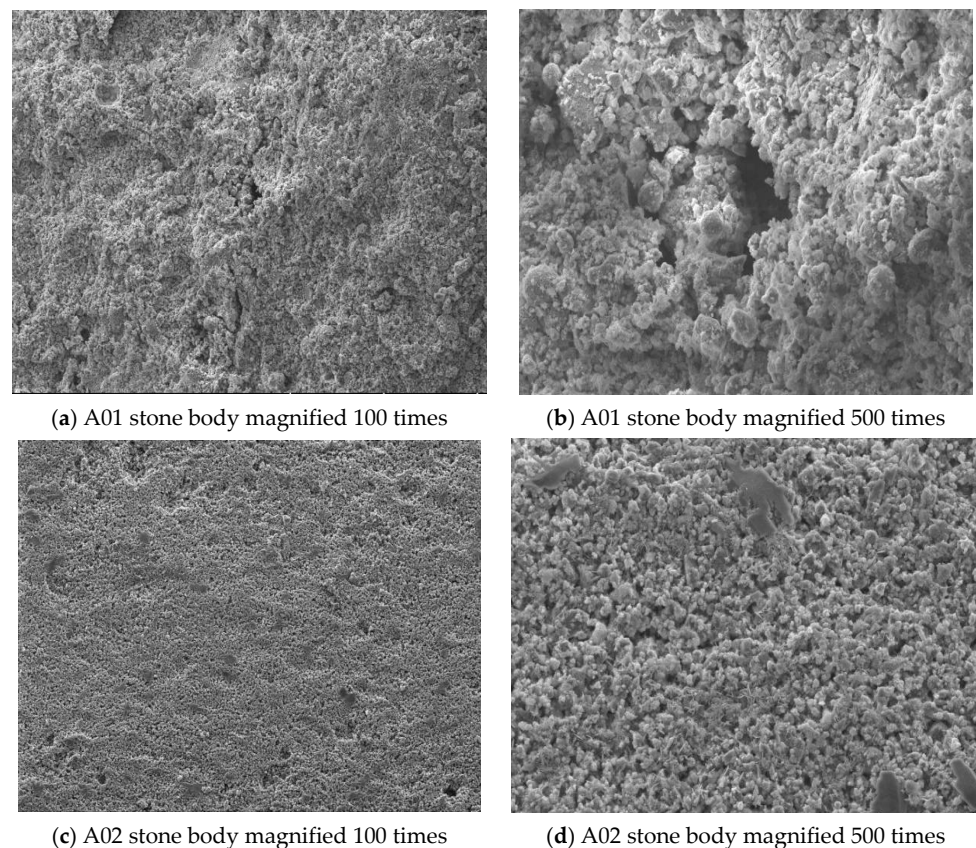


Figure 11. Scanning electron microscopy images of stone body.

The dense structure of the cement–polyvinyl alcohol slurry stone body in Figure 11c,d suggests that polyvinyl alcohol has some effect on the slurry stone body’s ability to resist cracking. The solidified cement–polyvinyl alcohol slurry stone body has weak permeability and high compressive strength due to the microscopic characteristics of the slurry stone body combined with compressive strength characteristics, which meets the requirement for the water plugging of the surrounding rock grouting under high water pressure. Compared to the conventional cement slurry and cement–water–glass slurry, it is more appropriate for grouting requirements under high water pressure.

4. Project Application

The study area is located in +900 m of Qiandao Lake water diversion tunnel K22 in Hangzhou. This site has an upper rock layer of 100 m and IV~V surrounding rock, and its permeability coefficient indicates strong permeability. The groundwater level is at ground level, and the tunnel’s central position head is 100 m. In order to prevent the excessive discharge of groundwater during tunnel construction, slurry plugging is applied to the surrounding rock [34–36]. The slurry ring thickness and slurry ring permeability coefficient are variables. The inner chamber of the cave is simplified as a circle, the equivalent inner diameter of the lining is 6.7 m, the equivalent outer diameter is 8.4 m, the lining permeability coefficient is 1.5×10^{-6} m/s, and the equivalent lining thickness is 0.85 m.

The tunnel excavation and grouting process was simulated using a two-dimensional seepage–stress coupling model created with the aid of the ABAQUS software. The model takes into account a hole influence range of 10 times its own diameter, and the model is designed to be 84 m wide and 130 m high. Horizontal displacement constraints are set on the model’s left and right sides, while vertical displacement constraints are set at the bottom. Position heads are set at the left and right boundaries, while boundary heads are set at the top and bottom, and the top of the model is impermeable. The inner surface of the liner is a permeable surface, and the water pressure is set to 0. Four calculation conditions were established, as shown in Table 3, in order to analyze the impact of cement–polyvinyl alcohol–boric acid slurry injection, and Figure 11 depicts the seepage field’s post-injection pore pressure cloud.

Table 3. Information of four working conditions and water pressure outside the liner.

Working Condition	Slurry Type	Slurry Ring Permeability Coefficient (m/s)	Water Pressure Outside the Liner (MPa)
Case 1	No grouting	1.0×10^{-5}	0.78
Case 2	Cement slurry	6.0×10^{-7} [37,38]	0.61
Case 3	Cement + water glass	1.8×10^{-7} [39,40]	0.54
Case 4	Cement + polyvinyl alcohol + boric acid	1.0×10^{-7} [41]	0.42

As seen in Figure 12a, when a tunnel is surrounded by rock without grout, the pore pressure cloud has the shape of an inverted funnel. The groundwater enters the tunnel after excavation, its level drops significantly, and the water pressure outside the lining is great. Excavation has a great influence on the seepage field. Figure 12d shows that after the injection of the cement–polyvinyl alcohol–boric acid slurry, the inverted funnel range of the pore pressure cloud decreases, and the drop in the groundwater level is smaller, which shows that tunnel excavation after grouting has little influence on the seepage field. This means that grouting has had a good effect on water plugging and achieved the expected plugging effect. When compared to Figure 12a, the pore pressure cloud diagram’s inverted funnel shape is improved in Figure 12c,d, suggesting that grouting has a stronger anti-seepage effect. However, the anti-seepage effect varies because the grouting materials have different anti-seepage effects, and cement-based slurry stones are brittle materials that are vulnerable to cracking and have a minimal anti-seepage effect when subjected to external water pressure. Due to the function of the modifiers, the anti-seepage effect of the cement–polyvinyl alcohol slurry is strengthened.

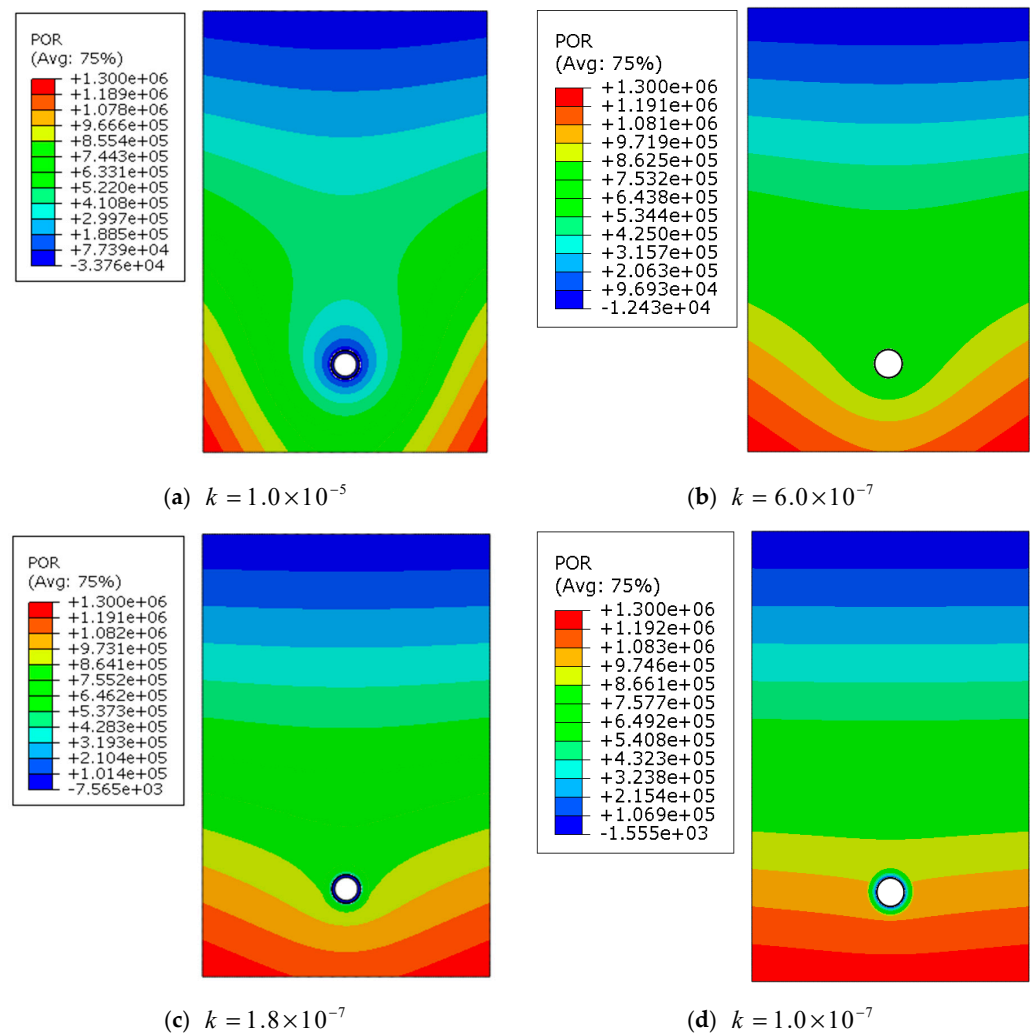


Figure 12. Pore pressure distribution clouds of the surrounding rock for the four working conditions.

According to Table 3, which displays the external water pressure of the lining under the four grouting conditions, the external water pressure of the lining starts to decrease once the permeability of the grouting ring is reduced, and it can withstand the external water pressure. In comparison to the first three working conditions, the external water pressure after the injection of cement–polyvinyl alcohol–boric acid grouting was reduced by 47%, 31%, and 22% after grouting with cement, polyvinyl alcohol, and boric acid, which perfectly demonstrates the improvement effect of the polyvinyl alcohol and boric acid modifiers. After grouting, a good water-plugging effect is achieved, which can reduce the water pressure outside the lining and reduce the discharge of groundwater.

5. Discussion

This study's findings indicate that when conventional grouting materials are used to seal off high water pressure around rocks, the grout's poor grouting performance and low retention ratio in a moving water environment endanger the tunnel's structural safety. As shown in Figures 6, 7, 10 and 11, the polyvinyl alcohol–boric acid–cement slurry controls the gel time through a modified concentration and volume ratio, a high retention ratio in moving water environments, and the good anti-seepage effect of the dense structure of the rock mass, and it had better grouting and water-plugging effects in an actual project of a water diversion tunnel, as depicted in Figure 12.

In this study, the laboratory test was mainly conducted to study the physical and mechanical properties of modified cement-based materials. To persuasively assess the

grouting effect of the grouting fluid on the surrounding rock, differences in the strength and permeability of the surrounding rock before and after grouting were tested by on-site sampling, and the reinforcement and water-blocking effects of the modified grouting fluid were analyzed. In the numerical calculation, for simplicity, only the influence of the grouting ring's permeability coefficient on the seepage field was considered, and the external water pressure of the lining was calculated. In situ rock sampling was not carried out for the test, nor were numerical calculation results compared. These optimal steps should be carried out in the future.

In different engineering contexts, the requirements for the physical properties and mechanical properties of the grouting fluid vary greatly [42–44]. Therefore, the grouting solution should be designed with modified materials according to the project needs, and if possible, intelligent machine technology should be combined with grouting technology to improve the grouting effect. In the near future, grouting technology may have broader applications, such as the landslide management of slopes and other projects, giving full play to the role of grouting material reinforcement and seepage resistance [45].

6. Conclusions

In this study, traditional building materials were improved to meet the requirements of the surrounding rock grouting under high water pressure and in a dynamic water environment. Through indoor tests, SEM tests, and numerical simulation methods, the following main conclusions are obtained.

The physical and mechanical characteristics of a single cement slurry exhibit a certain degree of injectability, a high stone ratio, high compressive strength, and high viscosity. The water–cement ratio has a greater effect on a single slurry; the smaller the ratio, the quicker the slurry initially and finally sets, and the higher the annual retention ratio of stones in a moving water environment.

The performance of cement-based materials for grouting is enhanced by polyvinyl alcohol and boric acid. By chemically crosslinking polyvinyl alcohol and boric acid to create a polymer–cement glue, the gel time of the cement-based slurry is shortened, indicating that the gel time can be adjusted by modifying the polyvinyl alcohol concentration and boric acid content. The formation of the cement gel increases the density of the slurry stone structure, and SEM verification suggests that the stone body will be more effectively sealed in the surrounding rock and will perform better in increasing seepage resistance.

The seepage flow field's numerical calculation demonstrates that the cement-glue-modified material has a better water-plugging effect. The low permeability of the grouting ring protects the tunnel's idyllic surroundings by reducing groundwater seepage, lowering water pressure outside the lining, and preventing groundwater levels from dropping.

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Blast Resistance of a Masonry Wall Coated with a Polyurea Elastomer

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Abstract: The coating of polyurea elastomers on walls is a hotspot in the protection field. This work combines a numerical simulation with experimental validation to examine the blast resistance after coating a polyurea elastomer on a 370 mm wall under a contact explosion. Firstly, the failure limit, failure mode, and failure mechanism of the 370 mm unreinforced wall under different strength loads are studied. In the case of the contact explosion, the increase in size of the 370 mm wall blasting pit gradually stops after the dose is increased to more than 1 kg. Thereafter, the energy of the explosive load was released by splashing wall fragments as well as by the deflection and movement of the wall. Spraying double-sided polyurea reinforcement on the wall can effectively improve the resistance to damages caused by exposure to explosive loads and can inhibit the damage to the surrounding personnel and equipment caused by flying structural debris. When the polyurea thickness on the front surface is 6 mm, the optimal thickness of the back surface should be 2 mm.

Keywords: masonry wall; polyurea elastomer; blast resistance; explosive loads

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1. Introduction

Terrorist acts on buildings have never disappeared in the modern world, where they routinely take place. Clay bricks are still a major component of modern buildings. Under detonation loading, the walls will suffer extensive damage. The lives of those residing in the buildings will be gravely affected when a large avalanche develops on the back of a wall. Therefore, it is vital to research ways to improve the anti-explosion performance of the walls without altering the building's fundamental design. According to Proter et al. [1], polyurea possesses excellent impact and rear resistance, as well as great application potential for anti-terrorism and explosive protection scenarios. Moreover, polyurea elastomers can strengthen a wall via spraying, and polyurea can cure and harden quickly, which is convenient during construction [2]. According to Bahei-El-Din et al. [3] and Tekalur et al. [4], spraying polyurea on a wall helps decrease the secondary damage produced by the wall's collapse by absorbing the shock energy. The impact of the polyurea and glass fiber-reinforced polymer (GFRP)-strengthened walls on the blast resistance was examined by Hrynyk and Myers [5]. The outcomes demonstrated that the wall reinforced with polyurea was the most efficient in lowering wall deflection and enhancing energy dissipation.

Researchers from around the world have studied wall blast resistance performance in an effort to reduce the damage caused by explosions to walls. Aghdamy et al. [6] studied the characteristics of the failure and collapse of concrete masonry walls reinforced by a nanoparticle polymer and aluminum foam under dynamic and impact conditions by combining experiments and simulations. Davidson et al. [2,7] analyzed the blast-resistant mechanism of a sprayed polyurea elastomer in strengthening masonry walls through a detonation test. Gattesco et al. [8] conducted experiments and numerical studies on the non-planar behavior of reinforced masonry walls. Fan et al. [9] simulated the response of and damage to brick-filled walls under explosive loads and reported multiple failure modes of masonry walls under different proportional distances. Zheng [10] studied the

responses of non-porous masonry walls under explosive loads. Xu and Lu [11] simulated the explosive shock behavior of reinforced concrete walls and found that wall caving may be considerably reduced in concrete walls by strengthening the back surface. By simulating the enhancement effect of polymers on the masonry wall performance, Aghdamy et al. [5] drew the conclusion that polymers can significantly increase the resistance of masonry walls to explosive loads. Dinan et al. [12] determined the damage mechanism and conducted experimental research on coated polymer masonry walls. The results showed that the polymer coatings can significantly enhance the explosion-proof performance of the masonry walls. Zhu et al. [13] developed a detailed micro-numerical model that can predict the non-linear dynamic responses of both unreinforced clay brick masonry walls and polyurea-reinforced clay brick masonry walls under close-in blasting. Davidson et al. [2,7] carried out a series of field explosion experiments to investigate the blast resistance performance of a wall enhanced with polyurea. Iqbal et al. [14] conducted an experimental investigation on a coated polymer masonry wall to identify the damage mechanism. The outcomes demonstrated that the masonry wall's performance as an explosion-proof barrier was greatly improved by the coated polymer layer.

In our previous study [15], we found that the blast resistance can be strengthened by coating polyurea elastomer on a 240 mm wall through an experiment and numerical simulation, and showed that the reinforced wall will not collapse even if the wall is penetrated. In addition, the influence of the thickness of the coated polyurea on the blast resistance ability was analyzed, and the optimal structure was found. In this paper, the same research method was used to study a 370 mm wall. It was found that the coated polyurea elastomer protected the 370 mm wall, as the main load-bearing structure of the buildings, from penetrating damage and large-sizes cracks, and we also found the best structure. A 370 mm wall and 240 mm wall will be different in a building, and the destruction forms in the building will also be different. The purpose of the study was to prevent a 240 mm wall coated with a polyurea elastomer from collapsing during explosion and a 370 mm wall coated with a polyurea elastomer from penetrating damage. In summary, the combination of the research results from the two articles can provide a basis for how to enhance the responses of existing building targets to terrorist explosion damage.

2. Numerical Simulation

2.1. Blast-Resistant Performance of Clay Masonry Wall under Contact Explosion

In this section, the damage to the masonry wall under contact explosions with different quality charges is analyzed via numerical simulation.

2.1.1. Simulation Model

The masonry wall was described using a different model. The bricks and mortar were created from several material units and were viewed as separate parts in the solution. The simulation was conducted in accordance with the masonry regulations for masonry walls in the relevant standard GB 50003-2011, Code for the Design of Masonry Structures [16]. In the model, the masonry method is "one straight, one horizontal, and one cement mortar joint". The LS-DYNA finite element dynamic analysis software was used to simulate the damage effect of the entire TNT explosion on the wall. For the bricks and mortar we used the Lagrange algorithm, and for both explosives and air we used the ALE algorithm. The size of the single masonry wall was 1990 mm × 1260 mm × 370 mm. Given that the single wall was axisymmetric, a 1/2 model of the masonry wall was used for calculation. The 1/2 model of the masonry wall is shown in Figure 1.

In actual brick wall buildings, the foundation is generally excavated. The walls are built from about 0.5 m below the ground, and the masonry is firmly connected to the ground. Therefore, the part of the wall bottom close to the ground can be regarded as fully constrained during the numerical simulation modeling. There are tie bars on both sides or columns connected with the wall on the side, which can effectively limit the displacement and deformation of the wall. In order to make the wall flat and beautiful

during construction, a thin layer of low-strength mortar (M5 and below) is often used to level the outside. In order to correspond to the actual situation, a full-constraint longitudinal strip is applied to the back burst surface of the 1/2 model to simulate the supporting effect of the adjacent walls on the masonry walls, so that the constraints on both sides of the wall are stronger than those on the upper and lower sides. The constraints in the X-axis direction are imposed on the symmetry plane, and non-reflective boundaries are imposed around the air domain (except for the symmetry plane). Due to the low strength and thin thickness of the plastering mortar outside the wall, its anti-explosion effect in the contact explosion is ignored.

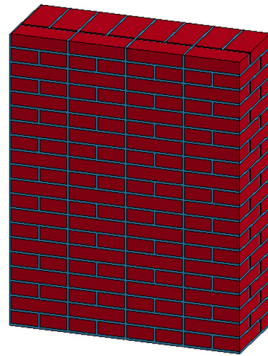


Figure 1. The 1/2 model and mesh generation of the masonry wall.

Since the duration of the explosion is very short, it is assumed that the clay brick block and mortar are well connected; that is, they are modeled in joint mode. In order to simulate the bonding, separation, and sliding between the bricks and mortar and to simulate the tensile failure and shear failure between the contact surfaces, TNTS is used to express the above relationship. The contact keyword is: * CONTACT_TIEBREAK_SURFACE_TO_SURFACE. The failure conditions of this keyword are:

$$\left(\frac{|\sigma_n|}{[\sigma_N]} \right)^2 + \left(\frac{|\sigma_s|}{[\sigma_S]} \right)^2 \geq 1 \quad (1)$$

where σ_n and σ_s are the calculated normal stress and calculated shear stress on the contact surface, respectively; $[\sigma_N]$ and $[\sigma_S]$ are the allowable normal stress and allowable shear stress on the contact surface, respectively, which can be taken as the elastic tensile strength and shear strength of the mortar material. As the tensile and shear strength between the masonry block and mortar of the brick masonry wall are relatively small, the allowable normal stress and allowable shear stress on the contact surface are 0.12 MPa and 0.14 MPa, respectively. After the failure of the contact surface, there will be dislocation and slippage between the block and mortar. At this time, the friction will prevent both from sliding. When the contact surface is dry, the friction coefficient μ is 0.7.

(1) Material parameters of Explosive and air.

The structural simulation model used the LS-DYNA ALE method to simulate the impact of the shock wave and the structure.

The JWL [17] equation of state for the explosives was used in the numerical simulation calculation;

$$P = A \left(1 - \frac{\omega}{R_1 V} \right) e^{-R_1 V} + B \left(1 - \frac{\omega}{R_2 V} \right) e^{-R_2 V} + \frac{\omega E}{V}, \quad (2)$$

where P is the pressure of the explosive detonation products; E is the internal energy per unit mass of the explosive; V is the relative volume; and A , B , R_1 , R_2 , and ω are the material parameters of the explosive. The material parameters and state equation parameters are provided in Tables 1 and 2.

Table 1. Material parameters of the explosives (g-cm-μs).

MID	RO	D	P _{CJ}	BETA	K	G	SIGY
—	1.63	0.693	0.27	0.00	0.00	0.00	0.00

Table 2. Parameters of the explosive (EOS_JWL) (g-cm-μs).

EOSID	A	B	R ₁	R ₂	OMEG	E ₀	V ₀
—	3.71	0.0743	4.15	0.95	0.3	0.07	1.00

(2) Material parameters of Masonry wall

The block bricks and mortar were described by the *MAT 096(MAT_-BRITTLE_DAMAGE) model in LS-DYNA in ANSYS 17.1. This model is primarily an anisotropic brittle damage model for concrete design and can be used for various brittle materials, including bricks and mortar. The restriction on the normal tractions is given by Equation (2) [17]:

$$\phi_t = (n \otimes n) : \sigma - f_n + (1 - \varepsilon)f_n(1 - \exp[-H\alpha]) \leq 0 \quad (3)$$

where n is the smeared crack normal, ε is the small constant, f_n is the initial principal tensile strength of the material, H is the softening modulus, and α is an internal variable. Here, H is set automatically by the program; α measures the crack field intensity and is output in the equivalent plastic strain field.

The primary material parameters of block bricks and mortar were listed in Table 3.

Table 3. Material parameters of blocks and mortar.

Material	Density (kg/m ³)	Elastic Modulus/MPa	Poisson's Ratio	Tensile Strength/MPa	Shear Strength/Mpa	Compressive Strength/Mpa	Fracture Toughness (N/m)	Shear Transfer Coefficient
Block	1150	380	0.15	1.00	0.50	17.6	120	0.03
Mortar	2100	4644	0.25	1.76	0.90	9.0	140	0.03

2.1.2. Numerical Simulation Results

Figure 2 depicts the damage to the wall over time when the TNT was 1 kg. The left and middle figures show that the explosion load first caused a central blasting pit and longitudinal cracks on the blast front surface of the masonry wall and gradually formed divergent cracks along the mortar joint around the blasting pit at about 400 μs. The longitudinal cracks expanded and thickened. The figure on the right suggests that the blast back surface first formed a longitudinal crack, and the central mortar joint of the wall began to crack at 400 μs. As shown in Figure 2c–e, the cracks on the blast back surface show a divergent shape as a whole (except the rib plate).

Figure 3 shows the damage to the wall at $t = 1000 \mu s$ under different explosive contact weights. The longitudinal cross-sectional view on the left shows that as the amount of charge gradually increased, the depth of the blasting pit increased and cracks were formed. The cracks in the vertical direction of the wall center, i.e., the direction of the axis of symmetry, were particularly deep. The vertical cross-sectional area S of the blasting pit of the brick wall under the contact explosions with different charges showed a gradual increase with increasing charge, and the volume of the blasting pit also showed an increasing trend. The destruction of the middle front surface showed that the strain gradually increased as the amount of charge increased. The area affected by the contact explosion also increased from spherical (Figure 3a) to petaloid (Figure 3b) and to square (Figure 3c,d). The deformation around the blasting pit diverged along the mortar joint in a radial shape (see Figure 3d–h). Sporadic avalanches appeared at the edges of the wall (see Figure 3a–h). The destruction of the back surface on the right side indicated that with the increasing charge, the deformation

range of the back surface gradually increased, and the mortar joint was a weak link. Collapse and spalling also developed along the mortar joint but in the shape of a ring (except the rib plate). The central mortar joint was severely injured, forming a through crack.

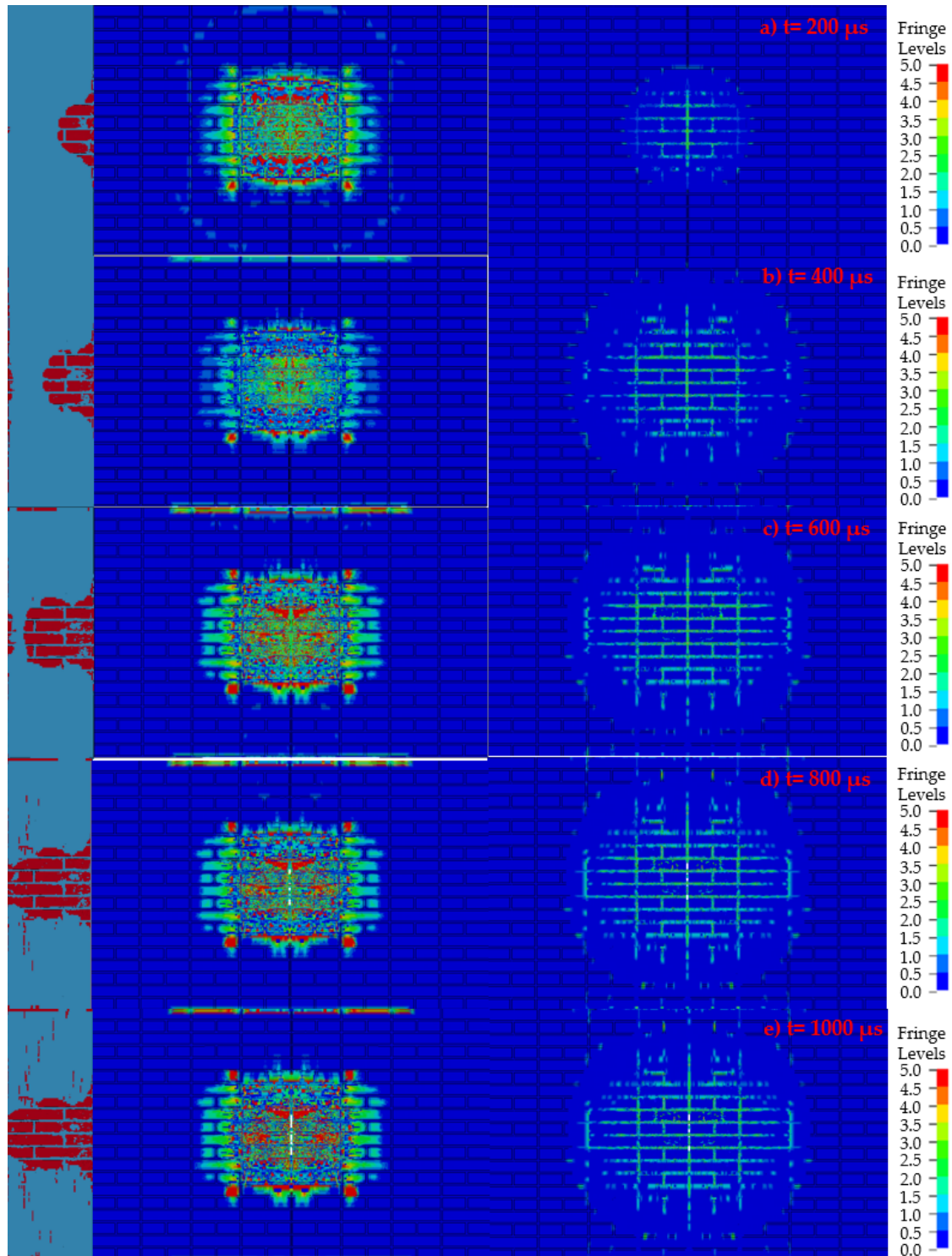


Figure 2. Damage to the 370 mm wall at different times. (a) 200 μ s; (b) 400 μ s; (c) 600 μ s; (d) 800 μ s; (e) 1000 μ s.

Figure 4a–c illustrate the linear increases in the area, depth, and radius of the blast pit when the explosive weights increase from 0.25 kg to 1 kg. However, with the increase in

mass of the charge, when the charge is greater than 1 kg, the area, depth, and radius of the explosive pit increase slowly and tend to be horizontal. This means that the charge mass of 1 kg is a critical state, and the damage form changes when it exceeds 1 kg.

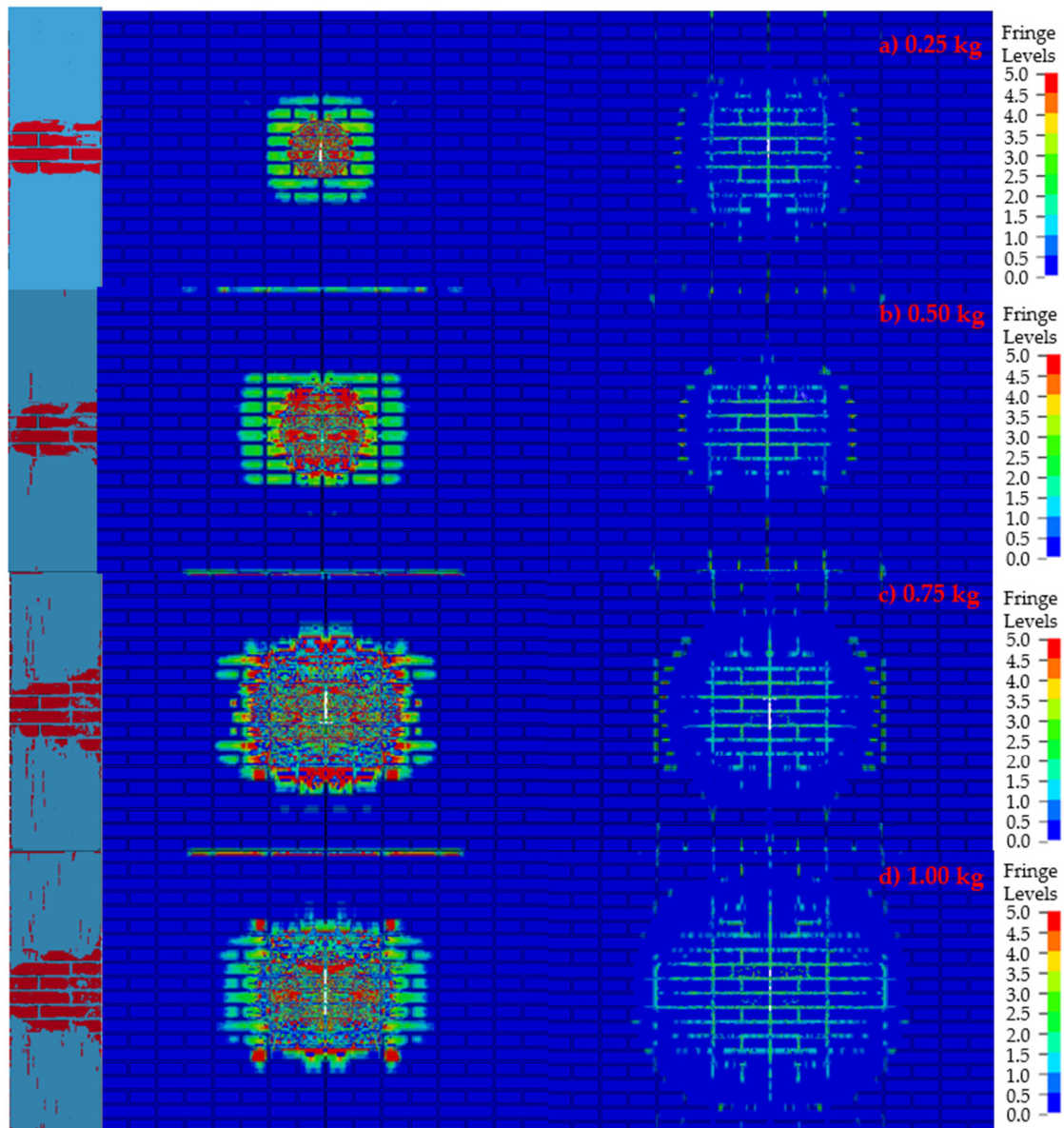


Figure 3. Cont.

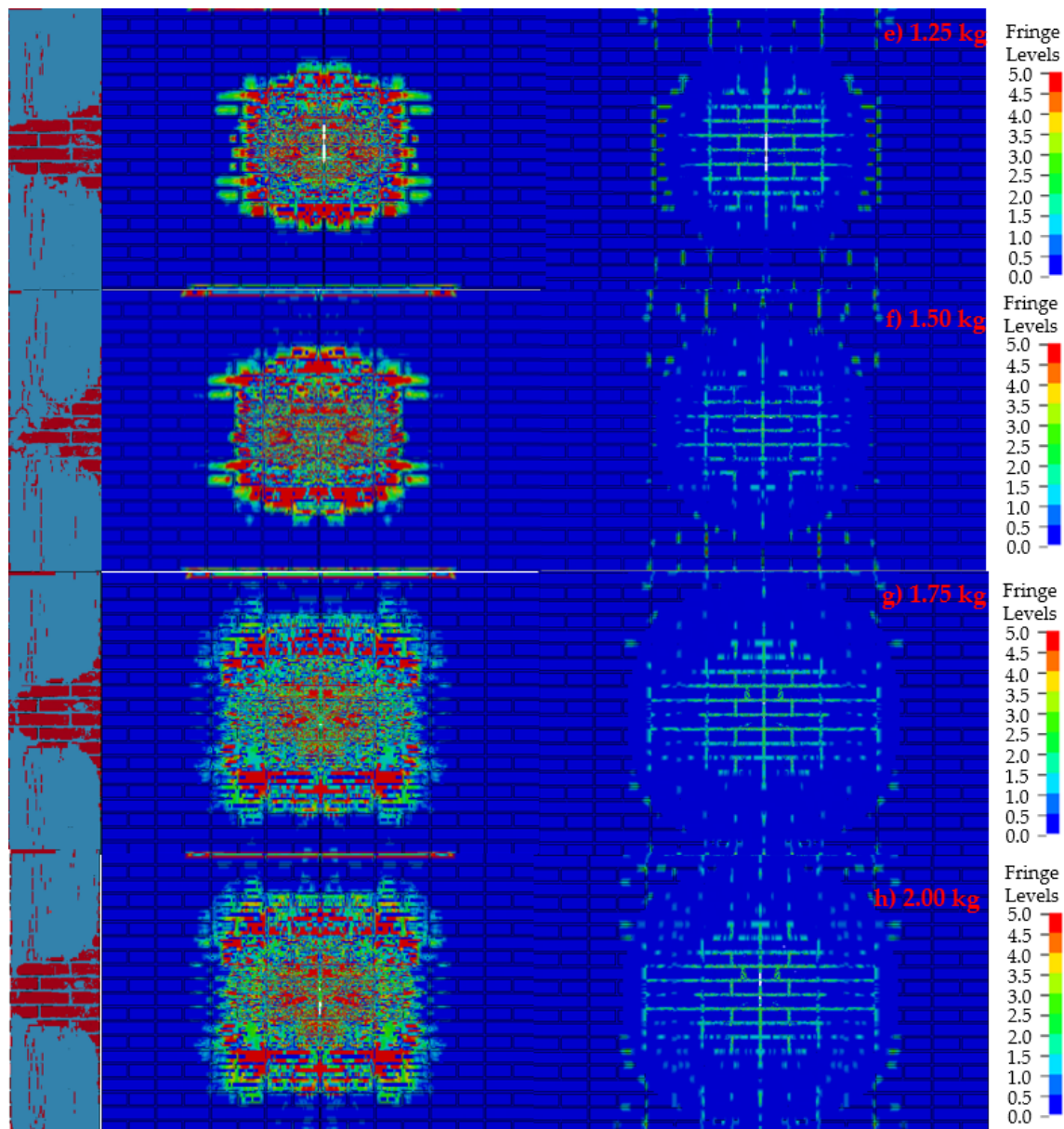


Figure 3. Damage to the 370 mm wall under different weights of TNT. (a) 0.25 kg; (b) 0.50 kg; (c) 0.75 kg; (d) 1.00 kg; (e) 1.25 kg; (f) 1.50 kg; (g) 1.75 kg; (h) 2.00 kg.

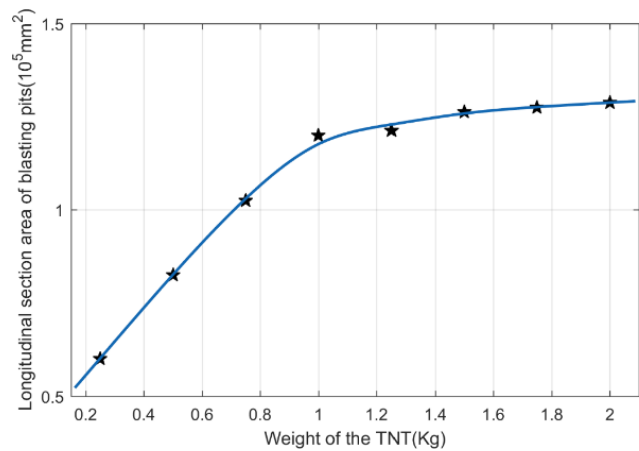
2.2. Blast-Resistant Characteristics of Masonry Wall Sprayed with Polyurea Elastomer

In this section, we detail the numerical simulations of the blast-resistant characteristics of masonry walls sprayed with different thicknesses of a polyurea elastomer.

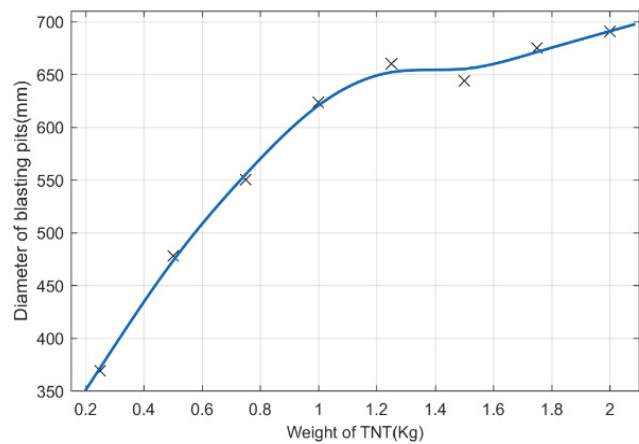
2.2.1. Numerical Model

In the numerical simulation, the thickness of the polyurea reinforcement layer on the front surface remained unchanged at 6 mm, while the thickness of the back surface reinforcement layer was changed to study the influence of the thickness changes in the polyurea reinforcement layer on the failure of the masonry wall under contact explosion. The thickness of the polyurea reinforcement layer of the 370 mm wall model is shown in Table 4. The 1/2 model of the sprayed polyurea-reinforced 370 mm wall and the meshing of the large-scale surface of the polyurea coating are shown in Figure 5. The bonding force between the polyurea elastomer and the brick wall was obviously greater than the peel strength of the mortar, so the polyurea elastomer sprayed on the surface of

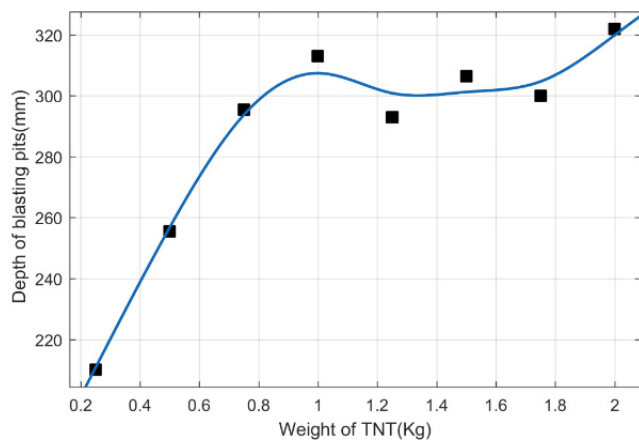
the clay masonry wall and the wall were coupled by a common node to simulate their deformation coordination.



(a)



(b)



(c)

Figure 4. Statistics for the longitudinal section size of the 370 mm wall under different weights of TNT: (a) longitudinal section area of the blasting pits; (b) diameter of the blasting pits; (c) depth of the blasting pits.

Table 4. Thickness dimensions of the polyurea reinforcement layer of a 370 mm wall model.

Serial number	1	2	3	4
Polyurea thickness on back surface/mm	2	4	6	8

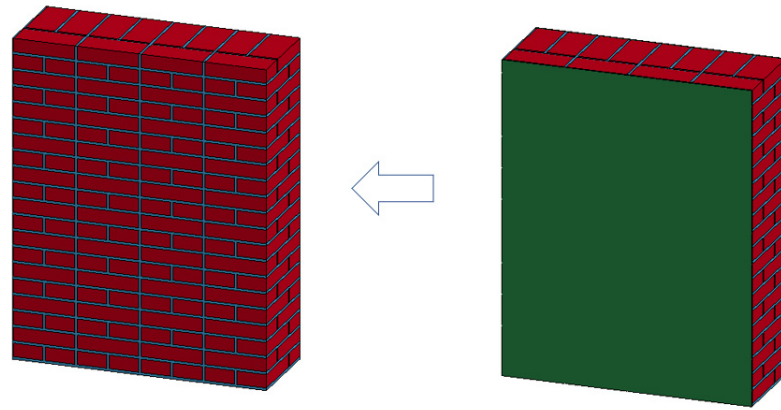
**Figure 5.** The 1/2 finite element model of the sprayed polyurea-strengthened 370 mm wall and the meshing of the polyurea layer.

Table 5 shows the mechanical properties of the polyurea elastomer [18].

Table 5. Parameters of the polyurea elastomer.

E/GPa	$\rho/(\text{g}/\text{cm}^3)$	ν	σ_Y/GPa	E_Y/GPa	σ_0/GPa
230	1.02	0.4	1.38	3.50	13.8
Load curve 1			Load curve 2		
1.0×10^{-1}	2.3×10^{-4}		1.6×10^{-9}	5.0×10^{-5}	
3.0×10^{-1}	2.7×10^{-4}		1.6×10^{-7}	6.0×10^{-5}	
5.0×10^{-1}	3.5×10^{-4}		1.4×10^{-5}	8.0×10^{-5}	
8.0×10^{-1}	4.2×10^{-4}		8.0×10^{-5}	8.5×10^{-5}	
8.5×10^{-1}	4.9×10^{-4}		8.0×10^{-4}	1.1×10^{-4}	
9.0×10^{-1}	5.8×10^{-4}		2.25×10^{-3}	1.2×10^{-4}	
			16.5×10^{-3}	1.6×10^{-4}	
			9.0×10^{-3}	2.2×10^{-4}	

2.2.2. Numerical Simulation of the Polyurea-Reinforced 370 mm Wall

When the amount of TNT was 1 kg under working condition 2 (the thickness of the reinforcement layer on the front surface was 6 mm and the thickness of the reinforcement layer on the back surface was 4 mm), Figure 6 illustrates the 370 mm wall's damage over time. The figure on the left shows the front side of the reinforced wall before the explosion, which reflects the damage to the surface of the polyurea reinforcement layer before the explosion. The middle figure is only the stress cloud diagram of the wall, reflecting the damage of the inner wall of the masonry. The figure on the right shows the rear side of the reinforced wall after blasting, reflecting the damage to the polyurea reinforcement layer on the blasted surface.

From the front view on the left, after the contact explosion, the polyurea layer on the front surface stretched and dented quickly under the action of the explosion load and was quickly damaged to expose the masonry wall structure of the lower layer. The polyurea layer at the central symmetry plane showed wavy distortion (see Figure 6a). The breach of the polyurea layer on the blast front face expanded rapidly with time, and the polyurea layer around the breach was stacked and folded after extrusion and stretching (see Figure 6a–d). After 800 μs , when the central breach of the polyurea layer expanded to

a certain extent, the size growth slowed down and some near-circular bulges appeared on the periphery of the stacked fold layer of the breach, leading to the uneven periphery of the folds (see Figure 6d,e).

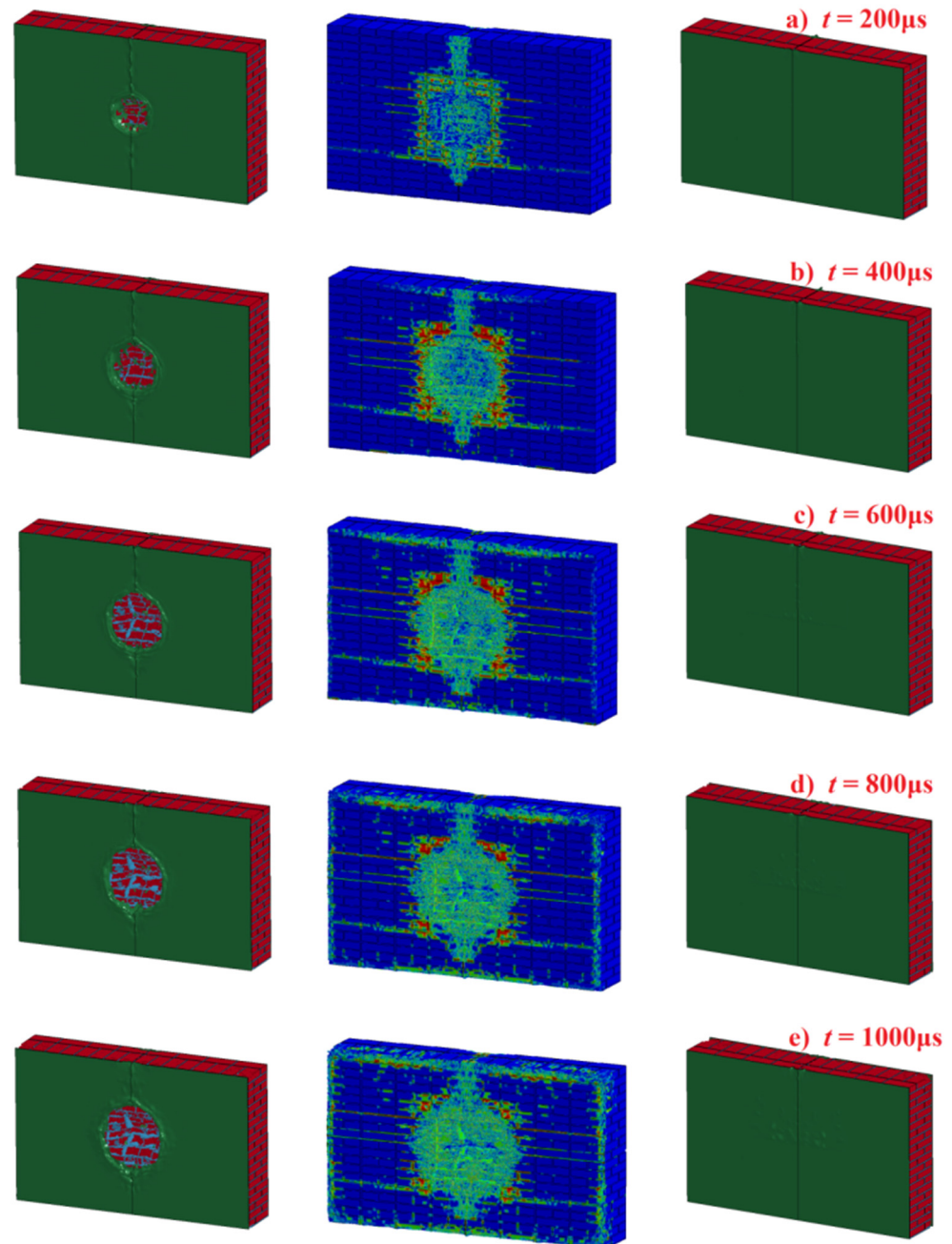


Figure 6. Destruction of the polyurea-reinforced 370 mm wall at different times. (a) 200 μ s; (b) 400 μ s; (c) 600 μ s; (d) 800 μ s; (e) 1000 μ s.

Based on the stress diagram of the brick masonry in the middle, the explosive load first caused a rapidly expanding central blasting pit and large longitudinal cracks on the front surface of the masonry wall, with some divergent cracks formed along the mortar joint around the blasting pit (see Figure 6a). After 400 μ s, the area and volume of the central blasting pit increased, radial cracks around it developed and were densely distributed, and the upper and lower sides of the wall were damaged (see Figure 6b,c). After 800 μ s, the increase in size of the central blasting pit was no longer obvious, and a little peeling damage also appeared on the left and right sides of the wall (see Figure 6d,e). Based on a

comparison of the figures on the left and in the middle, the area of the polyurea elastomer coating breach was always smaller than the area of the central blasting pit sandwiched by the masonry wall at any time.

The back surface on the right showed that the explosion load caused large longitudinal cracks in the center of the wall on the back surface, resulting in slight distortion in the top and middle of the back polyurea reinforcement layer (see Figure 6a,b). From 600 μ s, the middle gray seam of the polyurea reinforcement layer on the back surface gradually showed lumpy protrusions under the action of impact waves, and the raised lumps merged with each other and expanded (see Figure 6c–e).

Figure 7 shows the damage to the polyurea reinforcement layer on both sides under contact explosion at $t = 1000 \mu$ s. The back surface reinforcement layer has thickness k values of 2, 4, 6, and 8 mm, which correspond to the damage of working conditions 1–4, respectively, as shown by labels a through d. The figure on the left is the front surface of the reinforced wall, reflecting the damage of the polyurea reinforcement layer on the front surface; the figure on the right is the back side of the wall, reflecting the damage of the polyurea reinforcement layer on the back surface. Based on the front surface on the left under the same contact explosion load, the levels of deformation of the reinforcement layers from the explosions on the surfaces of the same thickness were very similar. The wavy distortions of the central symmetry plane were similar, and the sizes of the openings were almost the same. The polyurea layers around the break showed similar stacking, folding, and bulging after extrusion and stretching.

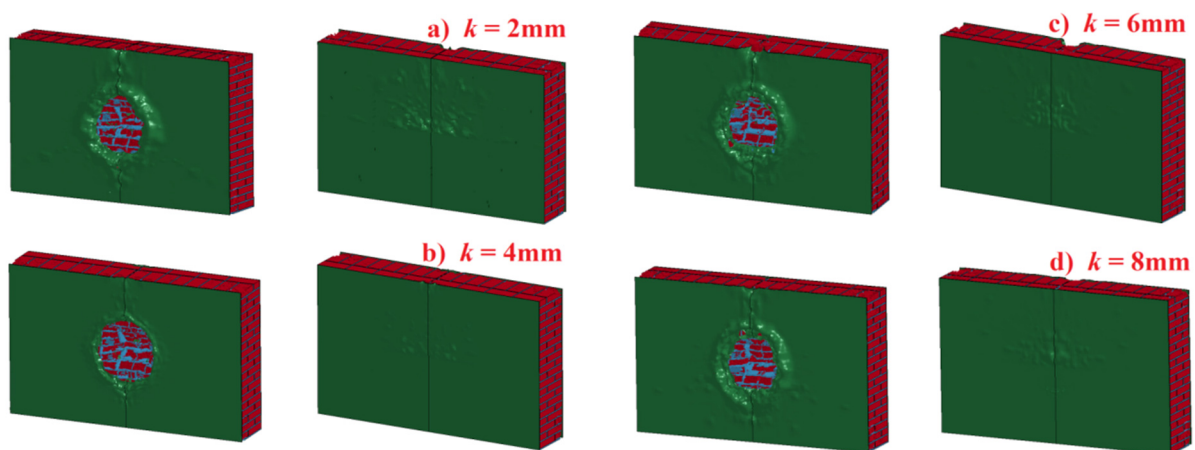


Figure 7. Damage of polyurea reinforcement layer under contact explosion at 1000 μ s with different k values (thickness of the back face): (a) $k = 2$ mm; (b) $k = 4$ mm; (c) $k = 6$ mm; (d) $k = 8$ mm.

Based on the back surface on the right, when the polyurea reinforcement layer on the back surface was thin (i.e., in working conditions 1 and 2, as shown in Figure 7a,b), the distortion of the top and middle parts of the reinforcement layer was obvious, and the uplift of the reinforcement layer was small and dense, often distributed along the mortar joint. When the polyurea reinforcement layer was thick (i.e., in working conditions 3 and 4, as shown in Figure 7c,d), the distortion of the top and middle parts of the reinforcement layer was not obvious, and the uplifts of the reinforcement layer were larger but smaller in number, distributed in the corresponding central part of the blasting pit.

To more clearly see the damage to the reinforced masonry wall under contact explosions, Figure 8 was created without the polyurea reinforcement layers on each side. Only the damage to the bricks and mortar under the four working conditions in Table 4 is depicted in the image. The reinforced masonry wall's longitudinal section or symmetrical plane, blast front surface, and blast back surface are all destroyed on the figure's left, middle, and right sides, respectively.

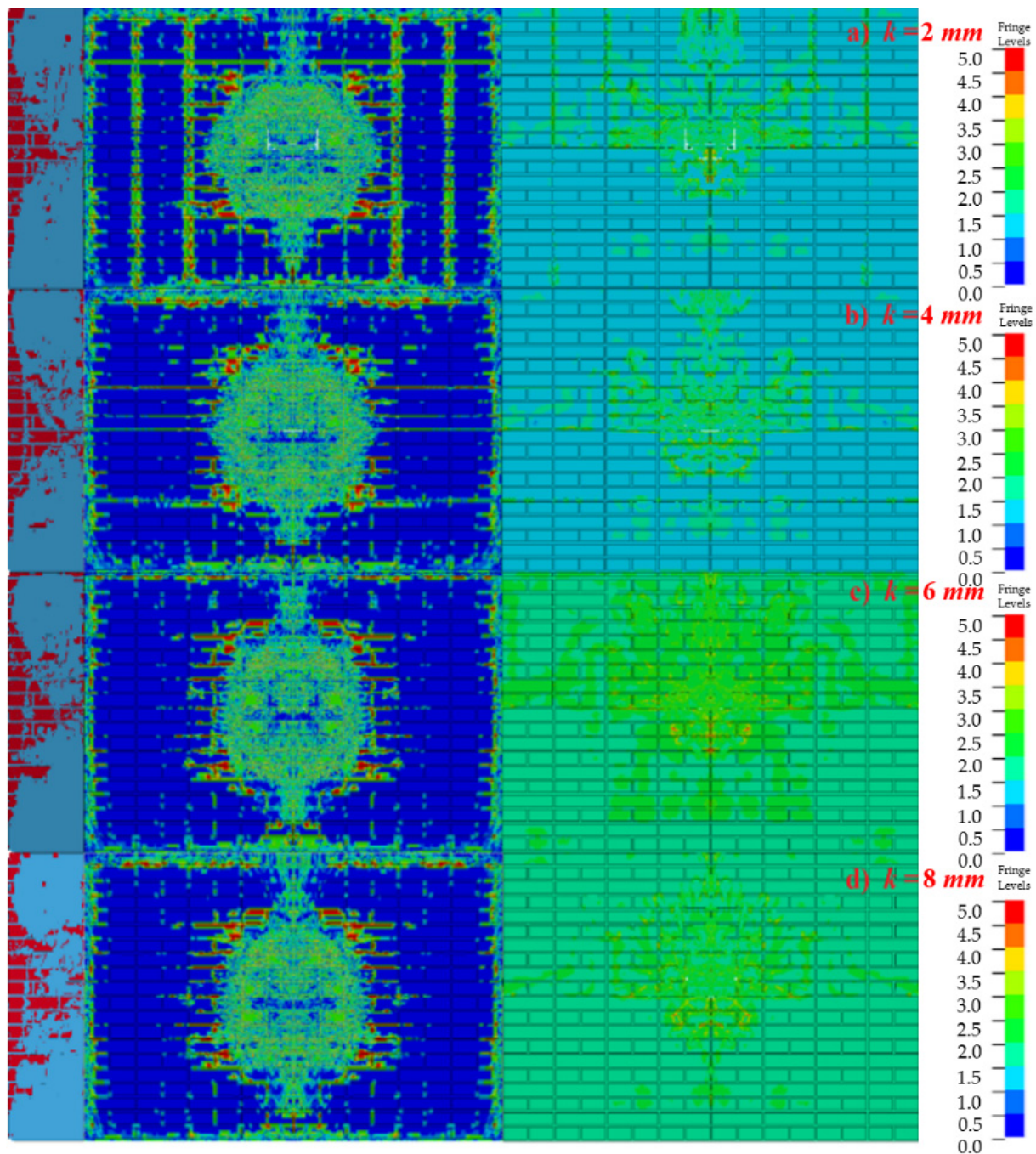


Figure 8. Damage to the reinforced 370 mm wall under contact explosion at 1000 μ s with different k values (thickness of back face: (a) $k = 2$ mm; (b) $k = 4$ mm; (c) $k = 6$ mm; (d) $k = 8$ mm).

As shown in the longitudinal section on the left, when the thickness of the polyurea reinforcement layer on the front surface remained unchanged, the depth of the central blasting pit of the masonry wall did not change much with the increasing thickness of the back surface reinforcement layer, and the cracks were densely distributed inside the masonry wall. The damage to the front surface in the middle shows that the areas of the central blasting pit (circled in the figure) under the four working conditions were different. When the back surface was thin (i.e., in conditions 1 and 2, as shown in Figure 8a,b), the area of the blasting pit was larger. When the polyurea reinforcement layer was thicker (i.e., in conditions 3 and 4, as shown in Figure 8c,d), the area of the blasting pit was smaller; the spalling and cracking of the explosion-proof surface of the wall under the four working

conditions were also different. Compared with conditions 3 and 4, the damage phenomena, such as the spalling and cracks on the blast front surface of the masonry wall concentrated in the mortar joint in conditions 1 and 2, were more obvious. The damage to the back surface on the right shows that under the four working conditions, the mortar joints were all weak links, and the collapse and spalling developed along the joints, with an overall radial trend (except for the rib plate). Compared with working conditions 3 and 4, where the polyurea reinforcement layer on the back surface was thicker, the strain on the back surface of the masonry wall in working conditions 1 and 2 was smaller.

By selecting the damaged areas along the bounding box, the size statistics of the same wall with different reinforcement methods under the same contact explosion load are shown in Table 6. As the thickness of the polyurea elastomer reinforcement layer on the blast back surface increased, the area of the central blasting pit decreased slightly; the values of working conditions 1 and 2 were close to those of working conditions 3 and 4. With the increasing thickness of the polyurea reinforcement layer on the blast back surface, the overall depth of the blasting pit did not change much, whereby the depths for working conditions 1, 2, and 4 were quite close, at about half of the thickness of the masonry wall.

Table 6. Size statistics for the destruction of the reinforced 370 mm wall under contact explosion.

Serial Number	Thickness of Front Surface/mm	Thickness of Back Surface/mm	Burst Area/mm ²	Burst Depth/mm
1	6	2	388,654	201
2	6	4	380,232	199
3	6	6	353,282	236
4	6	8	339,903	203

3. Experiment

3.1. Experimental Verification of 370 mm Wall under Contact Explosions

3.1.1. Test Plan and Arrangement

The test wall size was 2000 mm × 1200 mm × 370 mm, and a pair of ribs were symmetrically arranged on both sides of the wall to simulate the supporting effect of the surrounding walls on the actual building. The wall surface was smoothed with about 2 mm of mortar. The explosive used in the test was a TNT cylindrical compressed explosive with a charge density of 1.63 g/cm³. Two charge specifications were used, i.e., 0.50 kg (test 1) and 1.00 kg (test 2). The basic dimensions were Φ100 mm × 39 mm and Φ100 mm × 78 mm. The wall was larger than the explosive to ensure that the explosive would not destroy the entire wall and to avoid the influence of the masonry wall and the size effect of the charge. Tests 1 and 2 correspond to the b and d working conditions in the numerical simulation, respectively. The test layout is shown in Figure 9.



Figure 9. Test layout.

3.1.2. Experimental Results and Analysis

The experimental results and measured parameters are shown in Figure 10. In Figure 10a, a blasting pit is shown in the center of the blast front surface of the masonry wall. Most of collapse damage occurred in the mortar position, resulting in a small amount of block fragments, and the fragments scattered outwards. The fragments fell below the blasting pit, and the fragments were all over the area in front of the test wall. Large cracks appeared on the left and right sides and the upper side of the blasting pit on the front surface of the wall in the horizontal direction. The cracks penetrated to the back surface of the masonry wall. Small spidery radioactive spalls appeared around the blasting pit on the front surface. In test 2, a large blasting pit appeared in the center of the front surface of the masonry wall (see Figure 10b–d). The collapse damage also mostly appeared in the mortar, resulting in large amounts of block fragments and flying debris. Large penetrating cracks appeared in the horizontal direction and upper side of the blasting pit on the front surface of the wall. The upper left part of the masonry wall collapsed backward along the large cracks, and the upper right part was staggered along the large cracks and deflected at a small angle of about 2.3° .

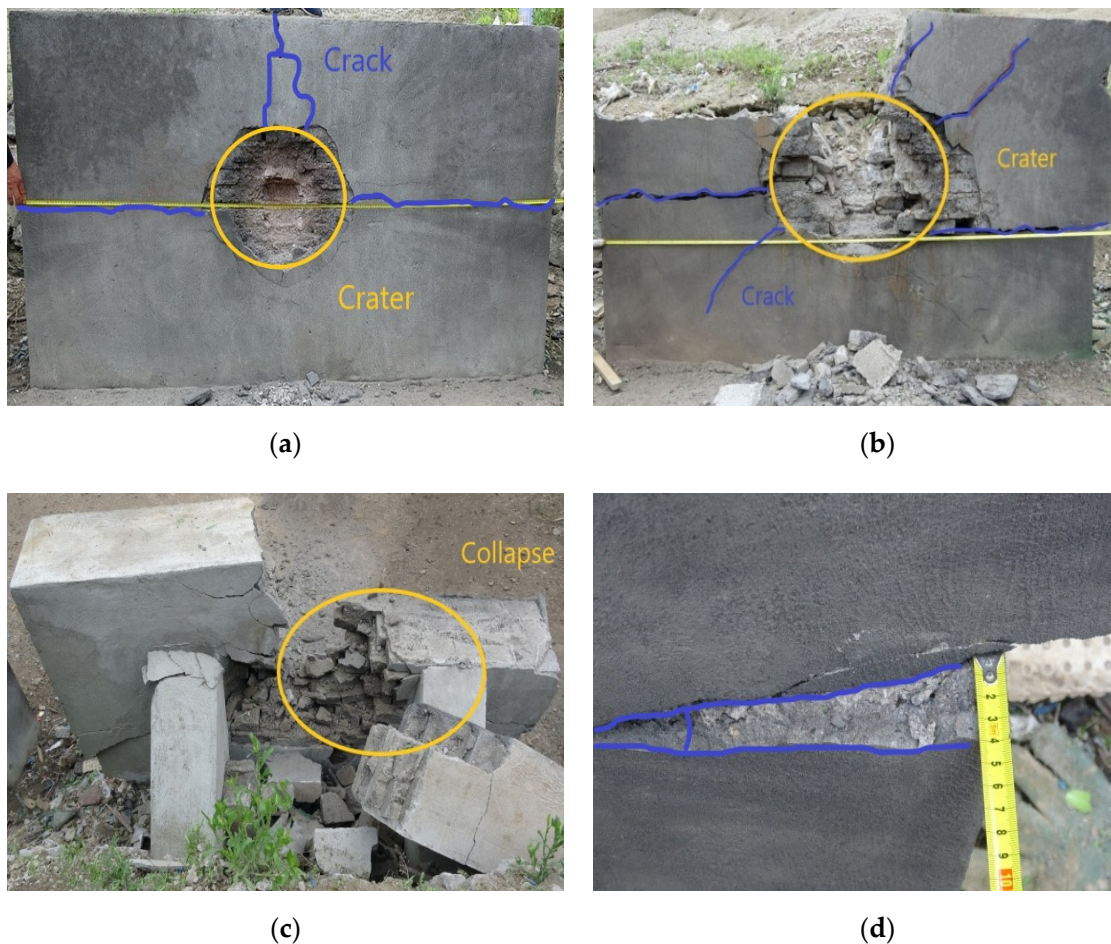


Figure 10. Damage of the wall in the test. (a) Test 1 blast front surface; (b) Test 2 blast front surface; (c) Test 2 blast back surface; (d) Test 2 Wall deflection.

3.2. Experimental Verification of Sprayed Polyurea to Reinforce 370 mm Wall under Contact Explosion

3.2.1. Test Plan and Arrangement

The masonry wall was the same as in the prior part, and a polyurea spray was utilized as the reinforcement. The TNT explosive with a density of 1.63g/cm^3 , a mass of 1 kg, and

dimensions of 100 mm × 78 mm was employed in the test. The reinforced 370 mm wall was sprayed with a 6-mm-thick polyurea reinforcement coating on the blast front surface and 2-mm- and 4-mm-thick polyurea reinforcement layers on the rear surface, respectively. Tests 1 and 2 correspond to working conditions 1 and 2 in the simulation, respectively, and the layout of the test site is shown in Figure 11.



Figure 11. Test site layout.

3.2.2. Experimental Results and Analysis

Figure 12 displays the damage to the reinforced wall during the test. The polyurea elastomer reinforcement layer coated on the surface was removed while maintaining the damaged state of the wall as much as possible. The right side of the figure demonstrates the damage to the wall when the reinforcing layer was removed. In tests 1 and 2, the center of the front surface in Figure 12a,b produced a hole with a small diameter, and on both sides of the hole there were several short tears. The area between the polyurea reinforcement layer and the brick wall “caught” some of the wall fragments, trash, and dust, which was visible through the hole in the reinforcement layer. Large blasting pits with shallow edges were formed in the center of the internal masonry wall, but the depth of the pits became larger in the part corresponding to explosive position. A large number of large radial cracks were distributed around the blast pit, with horizontal and vertical cracks penetrating the brick wall. In test 2, around the center blasting hole of the interior masonry wall, numerous little spidery fractures and spalls appeared along the mortar joints. Based on Figure 12c, the large cracks in the wall center of test 2 extended to the back surface, and the back surface of the internal masonry wall had cracks diverging from the center to the surroundings, where the horizontal and vertical cracks were the coarsest. The polyurea elastomer reinforcing layer on the blast back surface was not torn by massive fractures though. The reinforcing layer, which experienced a minor uplift under the displacement of the internal brick wall, blocked the huge fractures on top, as seen in the left half of Figure 12c.



(a)

Figure 12. Cont.

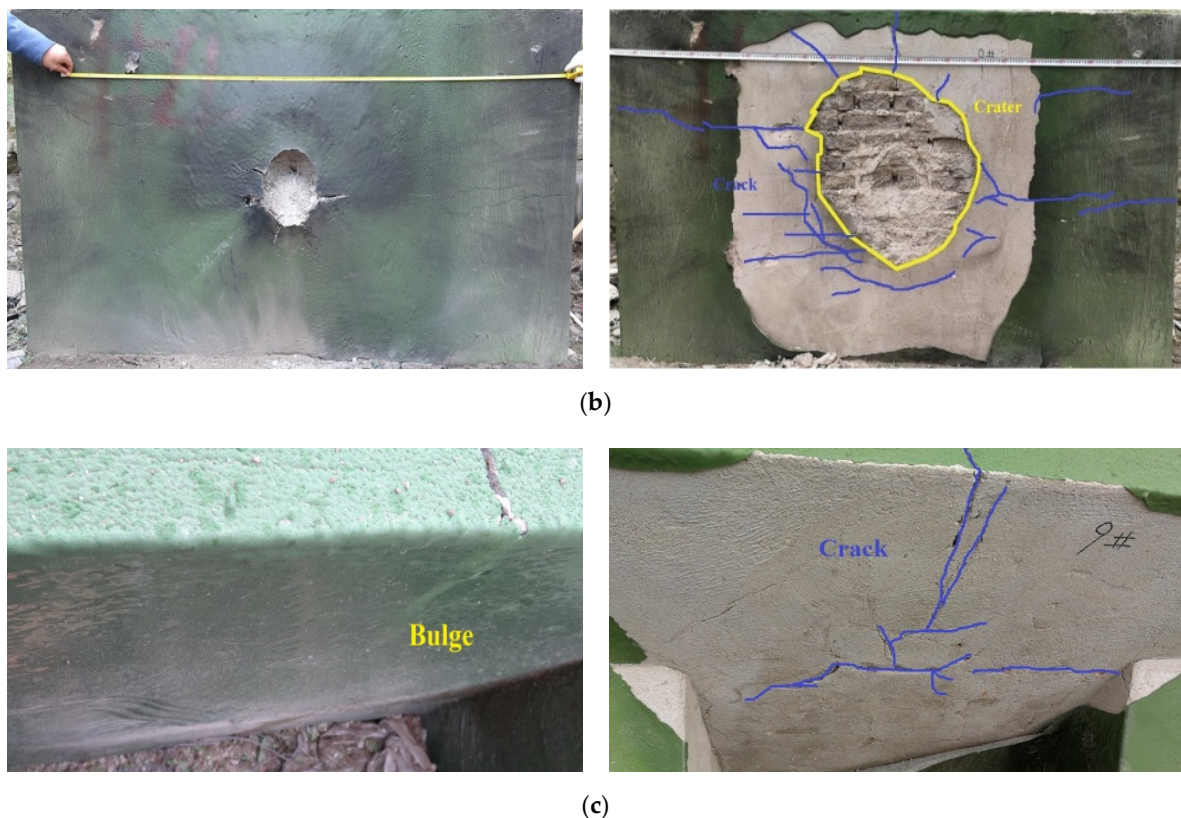


Figure 12. Damage to the reinforced 370 mm wall in the test. (a) Test 1 blast front surface; (b) Test 2 blast front surface; (c) Test 2 blast back surface.

4. Comparative Analysis

4.1. Comparative Analysis of Numerical Simulation and Experimental Results for an Unreinforced Wall

The numerical simulation showed that when the charge was less than 1 kg, blasting pits and longitudinal cracks predominated; when the charge was greater than 1 kg, the wall's front surface was covered in cracks and its back surface was covered in annular cracks along the mortar joint. In the test, the small charge in test 1 only formed blasting pits and a few cracks. However, not only did the increased charge in test 2 form larger blasting pits and thicker cracks, the complete penetration of the cracks also led to the brittle fracture of the wall. As such, dislocation, deflection, and collapse occurred under the action of the impact wave.

When the contact explosion occurred, the expansion wave and impact wave of the detonation products immediately reached the masonry wall, and the compressive stress wave caused serious damage to the front face of the wall. The central blasting pit and its surrounding cracks developed to different thicknesses along the mortar joint. The pressure surge propagated to the blast back surface to form a strong tensile wave, which caused the collapse and spalling of the back surface. The collapse and spalling caused by the strong tensile wave also developed along the mortar joint in a circular trend, forming penetrating cracks at the central mortar joint.

As shown in Figure 3, after the charge quantity exceeded 1 kg, the expansion of the blasting pit size gradually came to an end. According to the dislocation and collapse of the masonry wall in test 2, when the charge was large, the wall was penetrated and the energy of blast was converted into the displacement part of the wall. The dominant mode of energy release for the masonry wall changed as the charge increased; that is, when the charge was less than 1 kg, the damage to the masonry wall by explosives was mainly reflected in the cross-shaped cracks in the central blasting pits and in horizontal and vertical directions, and then when the charge exceeded 1 kg, the wall was penetrated and the damage to

the masonry wall gradually spread, especially the damage to the surrounding mortar joints, until part of the mortar joints penetrated the wall and caused the wall's dislocation, deflection, and collapse.

4.2. Comparative Analysis of Numerical Simulation and Experimental Results for a Reinforced Wall

When the reinforced masonry wall was subjected to the contact explosion load, the numerical simulation depicted the process of large blasting pits forming, with small cracks forming at the edge of the pits and growing along the mortar joints, and coarse cracks gradually extending and even penetrating along the mortar joints in the horizontal and vertical directions. The general consistency of the damage state with the test results shows that the wall model with the polyurea reinforcement layer and the polyurea material model can accurately represent the dynamic response of a reinforced masonry wall under contact explosion load.

In the numerical simulation of the reinforced 370 mm wall, the depths of the blasting pits in the four working conditions were close. When the polyurea reinforcement layer on the back surface was thin, the deformation mainly appeared as the collapse of the masonry around the blasting pits and large-area cracks that developed along the mortar joints, except for in the central blasting pit. When the thickness of the polyurea reinforcement layer on the blast back surface was thick, the crack damages on the front surface of the wall converged.

Compared with the unreinforced 370 mm wall under a 1 kg TNT contact explosion (see Figure 3b–d), the damage to the masonry wall reinforced by the polyurea elastomer under the two working conditions was improved under the same conditions. After reinforcement, the cracks on the surface and other minor damages to the masonry wall increased. The blasting area of test 2 was significantly larger than that of the unreinforced wall, but the number of thick cracks penetrating the 370 mm wall was significantly reduced, with only the longitudinal cracks in the middle remaining. More importantly, the unreinforced masonry, which suffered severe penetrating damage under the contact explosion in the upper part of the wall, deflected and even collapsed, and large amounts of wall fragments, debris, and dust were scattered around the wall after splashing. After spraying the polyurea elastomer for reinforcement, the depth of the central blasting pit was suppressed effectively, the masonry wall remained complete under the contact explosion load, and the polyurea reinforcement layer on the blast back surface of the wall wrapped the debris generated by the wall damage effectively and prevented the splashing of the debris and other destructive elements.

Table 7 shows a simulation and test parameter comparison of the damages to the unreinforced 370 mm wall and the 370 mm wall reinforced by spraying the polyurea elastomer under contact explosion. The areas of the central blasting pit after reinforcement were all larger than those of the unreinforced 370 mm wall. With the increasing thickness of the polyurea reinforcement layer on the blast back surface, the area of the blasting pit slightly reduced; the blasting area in test 2 was close to the numerical simulation value (the error was about 10.3%), but the blasting area in test 1 was quite different from the numerical simulation value. The manually sprayed polyurea layer on the brick wall in the test was not uniform, leading to a certain degree of variation in the test results. Before the explosion test, the wall in the explosion test site of test 1 was improperly stored, and the polyurea reinforcement layer at the lower left corner was damaged and partially peeled off (see Figure 12a), causing a break in the lower left corner. Although the break was at the edge of the wall and far from the center of the contact explosive load, the break acted as a “vent.” The right figure in Figure 12a shows that from the surroundings of the central blasting pit to the lower left corner, black detonation product residue was left on the masonry wall, which reduced the surface damage to the masonry wall in test 1 and decreased the area of the blasting pit.

Table 7. Comparison of the parameters for the destruction of the reinforced 370 mm wall under contact explosion.

Serial Number	Polyurea Thickness on Front Surface/mm	Polyurea Thickness on Back Surface/mm	Blasting Area/mm ²		Blasting Depth/mm	
			Numerical Simulation	Experimental Validation	Numerical Simulation	Experimental Validation
Unreinforced	0	0	292,247	—	315	Penetrated
Working condition 1/test 1	6	2	388,654	271,847	201	191
Working condition 2/test 2	6	4	380,232	344,577	199	196
Working condition 3	6	6	353,282	—	236	—
Working condition 4	6	8	339,903	—	203	—

Table 7 shows that the depth of the central blasting pit after reinforcement was less than the corresponding value of the unreinforced 370 mm wall. The simulation values for working conditions 1, 2, and 4 were close. Compared with the unreinforced 370 mm wall, the depth of blasting pit reduced significantly to about 36.4%. The numerical simulation values of blasting pit depths for working conditions 1 and 2 were in good agreement with the experimental values, with errors of 5.1% and 1.4%, respectively.

In summary, for working condition 1, the depth of the central blasting pit of the 370 mm wall was suppressed effectively, and the scattering of fragments, debris, and other destructive elements was prevented, thereby greatly improving the support and integrity of the wall. For working conditions 2 to 4, although a certain blast-resistant effect was achieved, the doubling of the thickness of polyurea layer on the back surface did not significantly increase the corresponding blast-resistant effect. Therefore, when the polyurea layer on the front explosion surface of the 370 mm wall was fixed at 6 mm, a coating of 2 mm of polyurea elastomer on the blast back surface can achieve a good reinforcement effect.

5. Conclusions

The following conclusions were reached after an analysis of the failure characteristics and damage processes of unreinforced and reinforced masonry walls under contact explosions:

(1) The destructional forms of the masonry wall under the contact explosions can be grouped into two kinds. When the explosive charge was below the critical value, the destruction of the masonry wall occurred near the explosive, forming a central blasting pit, a penetrating hole, and cross-cracks oriented in both horizontal and vertical directions. The other form of destruction slowly spread to the surrounding area, particularly the mortar joints; the spalling and collapse of the blast back surface also increased noticeably, meaning that eventually the wall would fall;

(2) The critical value of the TNT explosive for the 370 mm masonry wall under contact explosion was 1 kg. When the TNT explosive was over 1 kg, the destructional form of the masonry wall involved the splashing of wall fragments, deflection, and movement of the wall after the explosion;

(3) The polyurea elastomer sprayed on the front surface of the masonry wall can attenuate the shock waves generated by the TNT explosions effectively, and the polyurea elastomer sprayed on the back surface of the masonry wall can effectively restrain the wall breakage after the explosion. When the masonry wall was sprayed with the elastomer on the front and back surfaces, the wall's resistance to the damage caused by the explosive increased;

(4) When the thickness of the front polyurea layer was 6mm and the thickness of the back polyurea layer was 2 mm, the masonry wall had the best blast-resistant effect.

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Article

Effect of Planting Rebars on the Shear Strength of Interface between Full Lightweight Ceramsite Concrete and Ordinary Concrete

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Abstract: The use of full lightweight ceramsite concrete (FLWCC) for the repair of ordinary concrete (OC) structures has a good application prospect in the field of engineering structural strengthening. However, the interface here is less studied at present. For this purpose, 10 sets of FLWCC (new concrete)–OC (old concrete) specimens were produced for the shear test to test the bonding properties of the interface. The results showed that the damage form was changed from brittle damage to ductile damage after strengthening. It was proven that planting rebars in the interface could improve the shear performance. The interface shear strength increased with the number of rebars and it had a better effect after the number was more than 2. The strength was related to the rebar diameter and the maximum was obtained when the diameter was 8 mm. The most suitable spacing of the bars was 80 mm. The one-way analysis of variance (ANOVA) showed that the number of rebars had the greatest effect on shear strength followed by rebar diameter, while the effect of the spacing of the bars was less pronounced. Moreover, Fib's (2010) specification of the interface shear strength formula could be used for the calculation of FLWCC–OC.

Keywords: full lightweight ceramsite concrete; planting rebars; strengthening; new and old concrete; bonding interface; shear performance

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1. Introduction

Concrete materials are widely used in construction and transportation. However, in the course of daily use, concrete buildings can fail to reach their expected lifespan because they suffer from problems such as creeping and freeze-thaw cycles [1]. Strengthening is usually used in engineering to extend its life. Therefore, the choice of repair materials has an important influence on the strength of the reinforced concrete structure [1]. With the innovation of construction materials, lightweight [2,3], energy-saving and environmentally friendly materials have been widely used in building structures [4]. FLWCC, for example, has been extensively researched and commonly used in practical engineering and repair fields [5–7]. FLWCC is a kind of lightweight aggregate concrete with shale ceramic granules and shale ceramic sand instead of crushed stone and river sand. Shale ceramic granules are made from shale as raw material and industrial waste is added during the roasting process. This reflects its environmental advantages. Ceramic granules have a large number of pores inside, which gives them the advantage of being lightweight and having thermal

insulation. Therefore, when FLWCC is used in construction projects, it not only reduces the dead weight of the structure and saves the use of cement and steel but also reduces the burden of indoor HVAC (heating, ventilation and air conditioning). Because of the water retention properties of ceramic granules, when FLWCC is used to strengthen OC, the water inside the ceramic granules can further promote the hydration reaction after the hardening of the interface between the new and old concrete, thus making the interface stronger. At present, there is a lot of research on the composition of raw materials [8–10], force characteristics [11–13] and durability properties [14–16]. The application of FLWCC in the field of repair and strengthening can give full play to its potential to replace the shortcomings of traditional concrete, which is in line with the environmental protection concept of “carbon neutrality”. And at the same time, it also brings a new direction for the topic of concrete structural strengthening.

The bonding interface is the weak part of the structure due to the discontinuity of aggregate particles in the transition region between the new and old concrete and the more complex load transfer [17]. The bonding strength of the interface is mainly influenced by the roughness of the interface, the Van der Waals force and the shrinkage and rebar difference between new and old concrete [18]. In recent years, scholars have conducted relevant studies mainly on roughness treatment [19], interface agent performance enhancement [20], high-tech repair material application [21] and rebar implantation [22]. Among them, the research direction of roughness, interface agent and high-tech repair materials is to improve the adhesion and friction of the bonding interface to improve the strength, which is to improve the bonding strength by mechanical occlusion force and its own strength and stiffness. Planting rebars in the interface can increase the ductility of the repaired structure relative to other methods, thus greatly improving the safety of the overall structure. Therefore, this method is widely used in strengthening works. In the process of rebar implantation, the influencing factors of bonding interface implantation are rebar diameter, rebar planting rates, spacing of bars, anchorage depth, properties and coating on rebars, etc. [23].

Even after controlling the influencing factors mentioned above, the effect of rebar implantation may show different results under different conditions. The five-zone three-layer model is a widely recognized microtheoretical basis for reinforced concrete [24]. Based on this microscopic model, many scholars have conducted research on the effect on bonding performance. The interfacial shear strength between the new and old concrete is the most important factor affecting the strengthening effect [25]. There are two dominant views on whether planting rebars strengthens the bonding ability: one is that scholars believe that rebars enhance the bonding effect. For example, Du [21] concluded from double-sided shear tests of OC strengthened with ultra-high performance RPC reinforcing mesh that the shear strength of the reinforced concrete was 1.37–3.11 times higher than before strengthening. Tian [26] carried out finite element simulations of the shear performance of bonding interfaces strengthened with rebars. They concluded that rebar implantation can inhibit the development of cracks. The rebar implantation makes the damage process of the new and old concrete structures ductile and it can increase the cracking load and damage load of the specimens to a certain extent. Yang [27] investigated the effects of different rebar planting rates, anchorage depths and different spacing of bars on the shear properties of the UHPC–stone interface. The results showed that the interfacial shear strength and ductility of the combined specimens increased step by step with the increase in the rebar planting rate when it was less than 0.269%.

Another view is that the presence of rebar breaks the continuity of the concrete near the bonding interface and affects the force transfer. Jiang [28] summarized from rebar pull-out tests that the average bonding strength of prefabricated connected members with notches is reduced by approximately 26%. The reduction in strength is explained by the discontinuity in the mechanical properties of the concrete around the rebars due to the presence of the bonding interface. Costa [29] evaluated the effect of normal stress and rebar planting rates on the shear resistance of the bonding interface of light aggregate concrete

strengthened plain concrete. The tests illustrated that the bonding effect of the rebars comes into play only after the concrete near the bonding interface has cracked and bonding slip has occurred. The rebar planting rates do not contribute significantly to the maximum shear strength of the bonding interface. Goldyn [30] launched tests by strengthening plain concrete members with light aggregate concrete poured at different ages with bonding interfaces. Rebar implantation on the bonding interface causes a change in the damage mode of the member from brittle to ductile. The rebar planting rate was subjected to about 18%–23% of the stress at yield when the bonding interface was just cracked. This suggests that rebar can increase the post-peak residual stresses without significantly affecting the load-carrying capacity of the bonding interface.

The shear transfer mechanism of two layers of concrete in direct shear is complex, so it is necessary to evaluate the shear properties of the interface [31]. In response to the above research status, this paper investigated the shear performance of the bonding interface between FLWCC and OC. The rebar planting rate is varied by the number of rebars and diameter, so it is necessary to investigate which of the two ways enhances the interface shear best. Also, the spacing of the bars determines the rebar-concrete force transfer mode, so these three influencing factors are investigated in this experiment. The following work was performed using straight shear tests on the bonding interface. (1) The shear damage pattern and maximum shear strength under the influence of different factors were recorded and compared. (2) The most influential factors and factor levels were calculated and analyzed using one-way ANOVA combined with the analysis of the shear strength mechanism. (3) The predicted shear strength was predicted using the Fib (2010), the AASHTO (2010) and the ACI (2008) to derive the most appropriate expression for the specification expressing the present test situation. This paper expanded application areas for lightweight aggregate concrete. The properties of lightweight and thermal insulation give the ceramic granules a great research market with the potential to replace traditional concrete. This paper can provide macroscopic data reference for the study of the interface between old and new concrete and lay the foundation for the study of the microscopic interface of FLWCC–OC. Combined with the already available ceramsite concrete members' stress characteristics, durability, etc., it fills part of the research field of ceramsite.

2. Experimental Materials and Specimen Production

2.1. Experimental Materials

(1) Coarse aggregates

Ordinary crushed stone was used for OC in this study. The coarse aggregate of FLWCC used 900-grade crushed shale ceramsite pellets (as shown in Figure 1). Before concrete mixing, it was necessary to soak the ceramsite particles for 4 h and then dry for 12 h for prewetting treatment. The main physical properties of the coarse aggregates are shown in Table 1.

Table 1. Physical property parameters of coarse and fine aggregates.

Type of Aggregate	Particle Size (mm)	Volume Density (kg/m ³)	Apparent Density (kg/m ³)	Compressive Strength of Concrete Cylinder (MPa)	Water Absorption in 1 h (%)	Mud Content	Fineness Modulus
Ordinary crushed stone	5~20	814	2688	6.8	2.42	/	/
Ceramsite pellets	5~20	896	1553	9.7	6.3	/	/
Ordinary river sand	/	890	2641	/	/	≤2	3.5
Ceramsite sand	/	1570	1479	/	/	≤2	2.7



Figure 1. Coarse aggregates. (a) Ordinary crushed stone. (b) Ceramsite pellets.

(2) Fine aggregate

The ceramsite granules were made of 800-grade shale ceramsite sand produced by Yichang Guangda Ceramsite Granule Products Limited company. Its relevant physical property parameters and the ordinary river sand are also shown in Table 1.

(3) Cement

All of the cement's physical property indexes meet the test specification requirements, as shown in Table 2.

Table 2. Physical property parameters of cement.

Type of Cement	Code Number	Density (g/cm ³)	Mineral Composition of Clinker				Fineness (%)
			C ₃ S	C ₂ S	C ₃ A	C ₃ AF	Margin of 80 μ m Square Sieve
Ordinary silicate cement	P.O. 42.5	3.15	45	25	12	8	6.5

(4) Water-reducing agent

The water-reducing agent was HsC polycarboxylate-type superplasticizers, as shown in Table 3.

Table 3. Physical property parameters of water-reducing agent.

Appearance Color	PH Value	Specific Gravity	Solid Content (%)	Water Reducing Ratio (%)
Pale yellow	6~8	1.08 $\pm$ 0.02	40	25~35

(5) Rebar

The tested rebar type was HRB400 and the rebar diameters were 6 mm, 8 mm and 10 mm. The basic physical properties tested according to GB/T28.1-2010 [32] are shown in Table 4.

Table 4. Physical property parameters of rebar.

Category	Yield Strength (MPa)	Ultimate Strength (MPa)	Modulus of Elasticity (MPa)
HRB 400	435.5	671	2.03 $\times$ 10 ⁵

2.2. Mix Proportion

The old concrete was configured with strength grade C45. According to GB 50367-2013 [33], all new concrete should be one level higher than the strength grade of the original structural concrete. In order to meet the practical requirements of the project, new concrete with a strength class of LC50 was used. The proportions of OC and FLWCC are shown in Table 5.

Table 5. Mixed proportion of FLWCC (Unit: kg/m³).

Cement	Coarse Aggregates	Fine Aggregate	Water-Reducing Agent	Water
605	1050	565	/	230
550	620	600	5.1	154

2.3. Physical and Mechanical Properties

Some of the necessary indicators were measured according to GB/T50081-2019 [34]. The test results are shown in Table 6.

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

Types of Concrete	Cube Compressive Strength (MPa)	Axial Compressive Strength (MPa)	Modulus of Elasticity (GPa)
LC50	53.66	46.53	3.067
C45	35.31	27.28	3.043

2.4. Specimen Design

The rebar connection of the interface is shown in Figure 2. The mode of arrangement was divided into the forms of 1, 2, 3 and 4 bars. The bonding performance under the action of number of rebars, rebar diameter (double rebars) and spacing of the bars (double rebars) were studied. The roughness of interface was processed using the manual chiseling method and the roughness was determined using the sand cone method. Three specimens were made in each group, which was for accuracy. The final results of the test should be taken as the average. Shear test groups are shown in Table 7.

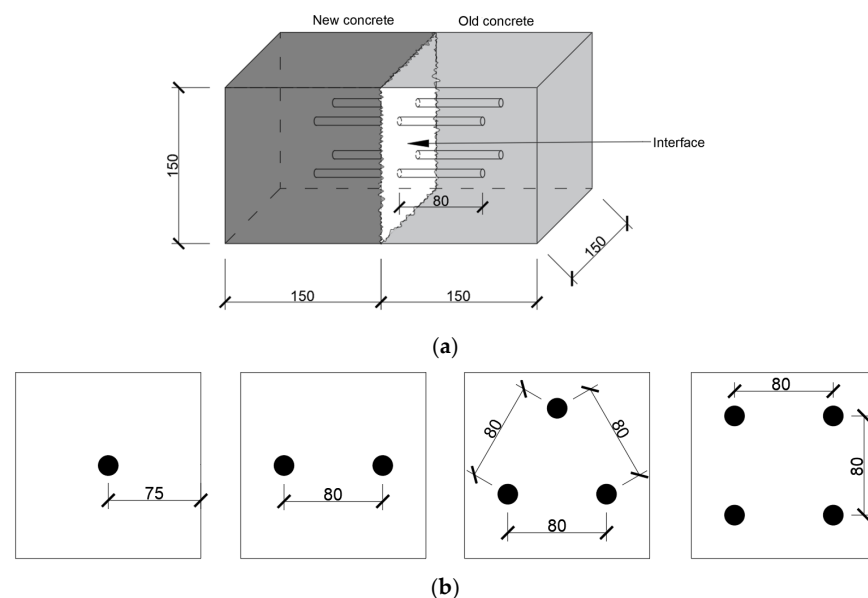


Figure 2. Design solutions for new and old concrete specimens (unit: mm). (a) Bonding specimens. (b) Rebar arrangement.

Table 7. Groups of shear tests.

Specimen Number	Number of Rebars	Rebar Planting Rate (%)	Spacing of Bars (mm)	Rebar Diameter (mm)	Anchorage Depth (mm)	Interface Roughness (mm)	Interface Agents	Number of Specimens
CZJ	/	/	/	/	/	/	/	3
SJ	/	/	/	/	/	2–3	cement paste	3
B1G1	1	0.223	C8	80	10 d	2–3	cement paste	3
B1G2	2	0.447	C8	80	10 d	2–3	cement paste	3
B1G3	3	0.670	C8	80	10 d	2–3	cement paste	3
B1G4	4	0.894	C8	80	10 d	2–3	cement paste	3
B2G1	2	0.251	C6	80	10 d	2–3	cement paste	3
B2G2	2	0.447	C8	80	10 d	2–3	cement paste	3
B2G3	2	0.698	C10	80	10 d	2–3	cement paste	3
B3G1	2	0.447	C8	60	10 d	2–3	cement paste	3
B3G2	2	0.447	C8	80	10 d	2–3	cement paste	3
B3G3	2	0.447	C8	100	10 d	2–3	cement paste	3

Note: The number of specimens was a shear test of each group of three specimens; “/” means that the group did not have this content. B1G1–B1G4 were the rebar planting rate groups. B2G1–B2G3 were the rebar diameter groups. B3G1–B3G3 were the spacing of bars groups. SJ was the cement net slurry treatment group. CZJ was the old concrete whole cast group. “d” was the diameter of the rebars.

2.5. Fabrication of Specimens

The poured interface of the old concrete was selected as the interface. After treatment with the manual chiseling method, rebars were then implanted on the interface. The method was scribing—drilling—clearing—filling with adhesives—implanting rebars—curing and maintenance. Before placing the new concrete, a cement paste was applied to the interface. The finished specimens are shown in Figure 3.



Figure 3. New and old concrete specimens. (a) After pouring. (b) After demolding.

2.6. Test Device and Loading Method

Four-point loading of distribution beams was used as the test loading method. And the main instruments of the test were a microcomputer-controlled electro-hydraulic servo universal testing machine, a loading sensing cell, etc.

The conceptual and field diagrams of the test setup are shown in Figure 4. It can be seen that the relationship between the distance d_2 between the far loading point and the bonding interface and the distance d_1 between the near loading point and the interface was $d_1 = \frac{2}{5}d_2$. The compact four-point shear loading was $p_1 = \frac{5}{2}p_2$ and the load at the interface could be calculated as $P_3 = \frac{3}{7}F$. That is to say, the shear force at the bonding interface was 3/7 times of the load loaded by the testing machine.

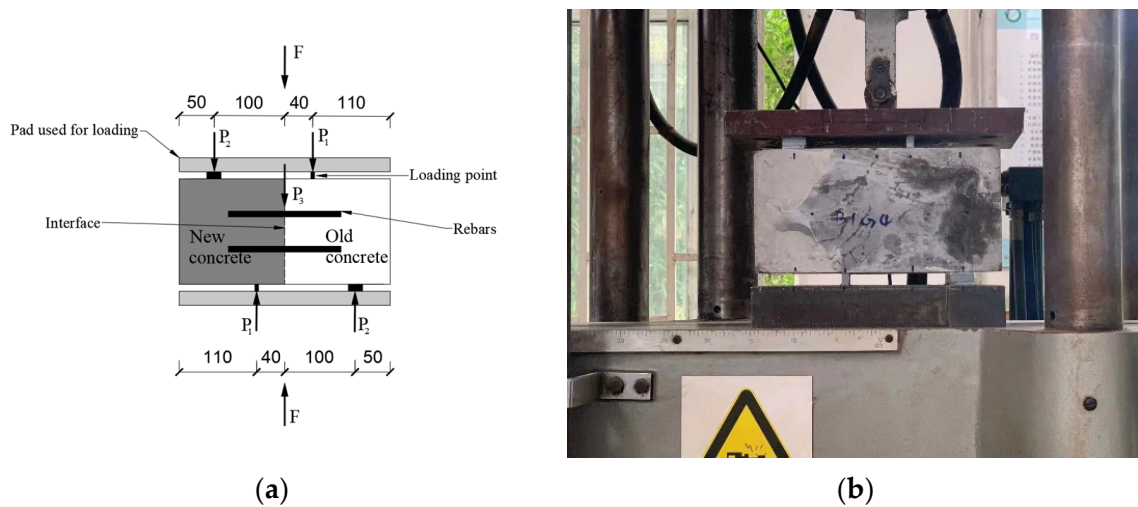


Figure 4. Test configuration of concrete samples. (a) Test diagram (unit: mm), (b) Laboratory test setup.

3. Experimental Results

3.1. Destruction Form

The damage pattern of 12 groups of specimens in a direct shear test is shown in Figures 5–8. When the shear force of the interface without rebars group (SJ) reached its limit, the specimens cracked along the interface instantly. There was no obvious crack from a macroscopic point of view before fracture of the interface and its damage form was brittle damage. The damage pattern of the interface showed that the damage interface was in the same direction as the shear force action. The damage interface on the old concrete side was similar to the interface morphology after the old concrete was chiseled at the beginning. This fully proved that the interface was the weak part of shear damage resistance.

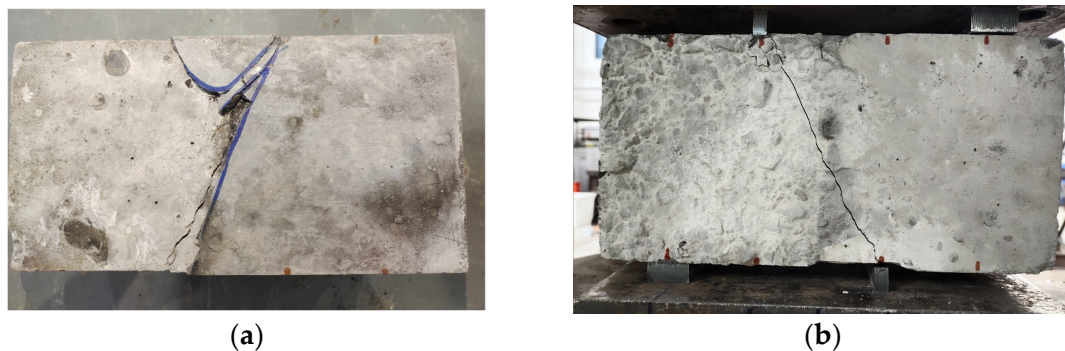


Figure 5. Unstrengthened group shear damage. (a) CZJ group. (b) SJ group.

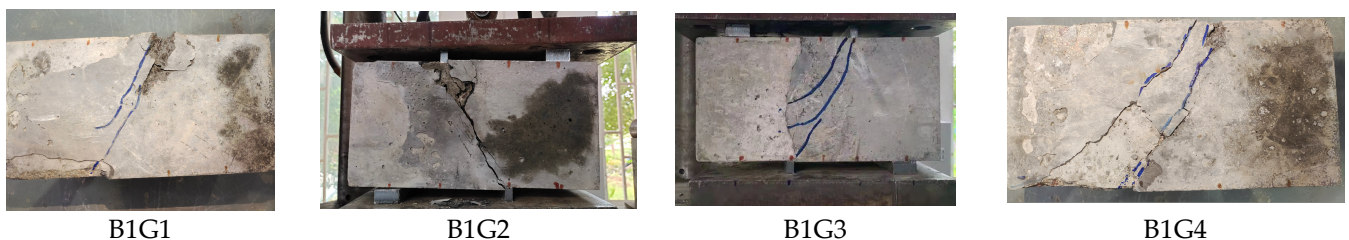


Figure 6. Shear damage with different number of rebars.



Figure 7. Shear damage with different rebar diameters.



Figure 8. Shear damage with different spacing of bars.

The damage of the bonding interface group (B1G1–B1G4, B2G1–B2G3, B3G1–B3G3) is shown in Figures 6–8. The whole damage process could be divided into four stages: At the beginning of loading, the specimens had no obvious changes. With the increase in load, small cracks appeared at the edge of interface. Upon continued loading, the cracks gradually penetrated, with their width widened, and a small amount of sheared aggregate was dislodged from the surface. When the ultimate load was finally reached, the rebars were not pulled out or cut off and the concrete was not partially pulled out.

Observation of the damage pattern showed that the damage pattern of the interface group was that cracks extended from the interface to the concrete on both sides caused by the shearing force. The destruction process of the specimens with different variables was approximately the same. The main cracks were the oblique cracks distributed between the shear supports, and some specimens also had a small number of micro cracks. This phenomenon was due to the axial pulling force of the rebars after cracks appeared in the specimens, which caused tensile stresses in the concrete around the rebars, therefore causing micro cracks.

What could also be seen was that aggregate shedding from the outer surface was a little more common in the groups with low rebar planting rates, such as B1G1, B1G2 and B2G1. It was clear to see that this phenomenon diminished as the rates were increased. This was due to the fact that the rebar bond to the concrete in the group with lower rebar planting rates had a smaller area of influence.

3.2. Shear Strength Analysis

The results of the shear strength tests at the interface are shown in Table 8. In summary, the shear strength was improved overall after 2 to 3 mm of manual chiseling method as well as rebar planting. The gain in shear strength rose from 0.48% to 50.52%. This indicated that the rebar planting played a good role in improving the shear resistance of the new and old concrete interface. A reasonable number of rebars, rebar diameter and spacing of the bars could significantly improve the shear strength so that the bonding performance could be enhanced.

Table 8. Results of shear test.

Group Number	Shear Load/kN	Shear Strength τ /MPa	Average Shear Strength/kN	$\frac{\tau}{\tau_{SJ}}$	Strength Gain Rate/%
CZJ	252.35	11.22	11.22	/	/
	256.90	11.42			
	247.80	11.01			
SJ	139.80	6.21	6.29	1	0
	143.25	6.37			
	141.53	6.29			
B1G1	142.30	6.32	6.32	1.005	0.5
	143.60	6.38			
	141.00	6.27			
B1G2	188.30	8.37	8.46	1.345	34.5
	179.35	7.97			
	203.60	9.05			
B1G3	181.30	8.06	9.02	1.434	43.4
	201.45	8.95			
	225.90	10.04			
B1G4	196.65	8.74	9.47	1.505	50.5
	218.75	9.72			
	223.65	9.94			
B2G1	142.23	6.32	6.32	1.005	0.5
	141.01	6.27			
	143.45	6.38			
B2G2	188.30	8.37	8.46	1.345	34.5
	179.35	7.97			
	203.60	9.05			
B2G3	147.40	6.55	7.46	1.185	18.5
	168.50	7.49			
	187.40	8.33			
B3G1	155.50	6.91	7.56	1.202	20.2
	165.35	7.35			
	189.30	8.41			
B3G2	188.30	8.37	8.46	1.345	34.5
	179.35	7.97			
	203.60	9.05			
B3G3	154.65	6.87	7.44	1.184	18.4
	177.75	7.90			
	170.10	7.56			

Note: τ_{SJ} was the shear strength of SJ group, $\frac{\tau}{\tau_{SJ}}$ was the ratio of shear strength of each group to SJ group.

1. Number of rebars

Figure 9 illustrates the influence pattern of the number of rebars on the shear strength. In conjunction with Table 8, the increase in shear strength was 0.5%, 34.5%, 43.4% and 50.5% when the number w increased from 0 to 4 (SJ, B1G1–B1G4). This indicated that the method of growing the rebar planting rate by increasing the number of rebars was effective in improving the shear strength of the interface. This finding was also in agreement with Yang [27]. This was because when the number of rebars increased, the tensile force generated by the rebars at the interface also increased. This led to an improvement in friction and pinning action at the interface.

Individually, B1G1 only increased by 0.5% compared to SJ, while B2G2 had a large increase of 34.5%. This implied that the degree of improvement in shear strength was not significant with a low rebar planting rate. When analyzed in conjunction with Yang's [27] study, this was because at a low rebar planting rate, the shear strength was mainly supported by the concrete and the rebars were unable to give full play to its ductile role. When the rate was high, the role of rebars in the middle and late stages of loading was more significant, thus the interface was strengthened by the rebars and the ductility was more obvious.

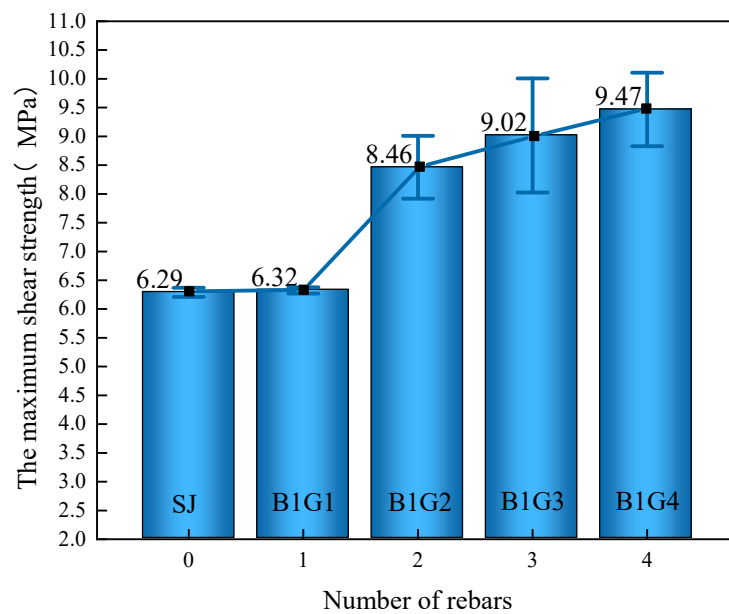


Figure 9. The maximum shear strength of the interface under the influence of the number of rebars.

2. Rebar diameter

As can be seen from Figure 10, the shear strength increased and then decreased with the growth of the rebar diameter. The shear strength reached its maximum at a diameter of 8 mm (B2G2), which was 34.5% higher than that of the SJ group. When the diameter was 10 mm (B2G3), the shear strength tended to decrease to 7.46 MPa, which was still larger than that of 6.32 MPa for 6 mm (B2G1). It could be seen that too large a rebar diameter did not improve the shear strength well. On the one hand, the increase in the diameter reduced the bonding performance with the concrete. On the other hand, it reduced the area of the concrete that was effectively loaded [35]. The selection of the appropriate rebar diameter in the actual project could better improve the interface bearing capacity.

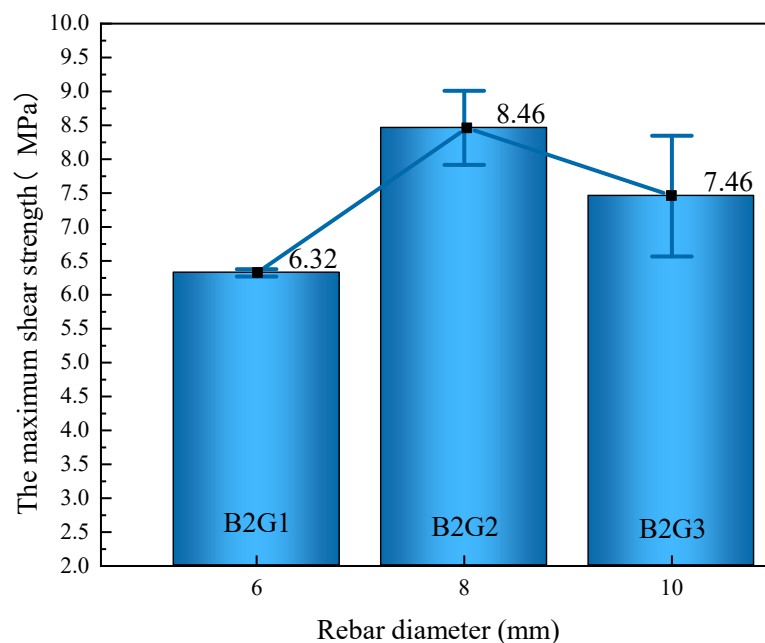


Figure 10. The maximum shear strength of the interface under the influence of the rebar diameter.

Combining B1G3 and B2G3, although the rebar planting rates were almost the same, the former had more than twice the rate of gain in shear strength than the latter. Figure 9 illustrates that the shear strength was positively correlated with the number of rebars within a certain range, while Figure 10 does not have this pattern. If the shear strength was to be increased by increasing the rebar planting rate, the number of rebars had a better gain than the rebar diameter. What was being suggested was that increasing the number of rebars in a strengthening project would be better than using larger diameters. Similarly, in practice, uniform sizes of smaller diameter rebar are also better constructed than different sizes of larger diameter.

3. Spacing of bars

Figure 11 depicts that the shear strength increased and then decreased with a rising spacing of the bars. This suggested that unsuitable spacing weakened the shear resistance of the rebars. The interface shear strength of the B3G2 group reached a maximum of 8.46 MPa. The shear strength of B3G1 was greater than B3G3. The reason for this phenomenon was the anchor group effect [36]. The internationally accepted method for calculating concrete post anchorage was the CCD method (concrete capacity design). It is shown in Figure 12, where h_{ef} is the anchorage depth. At a spacing of 60 mm (B3G1), the bearing areas of the two bars overlapped too much. As a result, the bars could not make good use of the concrete taper to transfer the load. When the spacing was 100 mm (B3G3), although the two bars could make better use of the concrete taper than with a spacing of 60 mm (B3G1), the edge distance of the rebar anchorage of 25 mm was too small for the cross-section of 150 mm × 150 mm. Hence, the cracks near the maximum stress of the bars developed toward the outer edge of the specimens, eventually leading to concrete splitting damage. Therefore, the spacing of 80 mm (B3G2) was the most appropriate in this test to achieve a balance in terms of edge distance and spacing compared to the other two groups. This conclusion was similar to that of Abbas [37]. The group anchor effect had to be taken into account in the actual project so that the connection between the rebar and concrete could be utilized as much as possible.

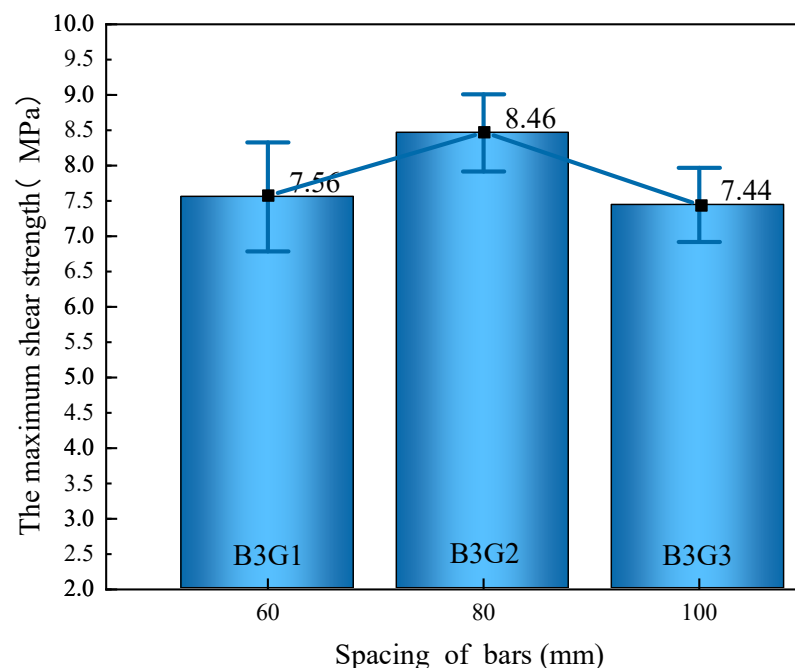


Figure 11. The maximum shear strength of the interface under the influence of the spacing of bars.

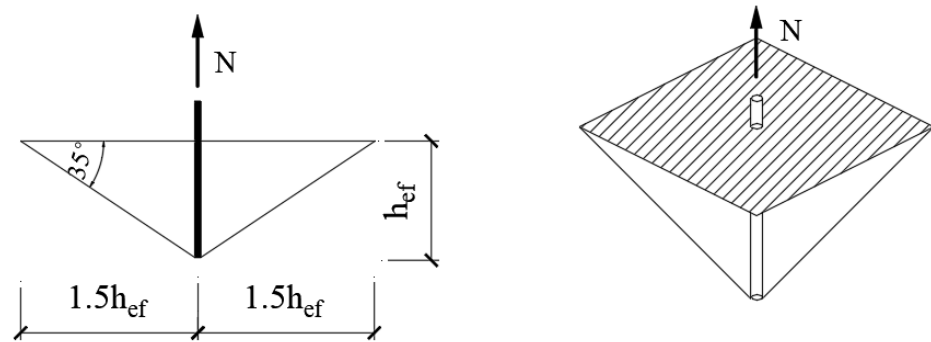


Figure 12. CCD algorithm calculation model.

3.3. One-Way ANOVA

The variation in the maximum shear strength reflected the effect of three influencing factors on the bonding performance. However, the strength of the influence of these factors was difficult to visualize. Therefore, in this section, one-way ANOVA was used to determine the strength and to further analyze which level of shear strength was greatest for a given factor.

The number of rebars (Factor A), rebar diameter (Factor B) and spacing of the bars (Factor C) were selected for significance analysis. The four levels of Factor A: A_1 , A_2 , A_3 , and A_4 meaning the number of rebars 1 (B1G1), 2 (B1G2), 3 (B1G3) and 4 (B1G4), respectively. As above, the three levels of Factor B: B_1 , B_2 , B_3 meaning the diameter of rebar 6 mm (B2G1), 8 mm (B2G2) and 10 mm (B2G3). The three levels of Factor C: C_1 , C_2 , C_3 meaning the spacing of bars 60 mm (B3G1), 80 mm (B3G2) and 100 mm (B3G3). The formula and results are shown in Table 9.

Table 9. One-way analysis of variance table.

Source of Variance	Calculated Formula	Number of Rebars (Factor A)	Value Rebar Diameter (Factor B)	Spacing of Bars (Factor C)
Correction C	$\frac{T^2}{n}$	830.17	494.77	550.53
Total sum of square S_T	$\sum \sum x_{ij}^2 - C$	20.81	9.07	4.21
Sum of effect squares S_A	$\frac{1}{l} \sum T_i^2 - C$	17.42	6.88	1.88
Error sum of squares S_e	$S_T - S_A$	3.39	2.19	2.34
Total degree of freedom df_T	$rl - 1$	11	8	8
Effect square and degree of freedom df_A	$r - 1$	3	2	2
Error square and degree of freedom df_e	$df_T - df_A$	8	6	6
Mean squared sum of effects $\bar{S}_A$	$\frac{S_A}{df_A}$	5.81	3.44	0.94
Mean square sum of errors $\bar{S}_e$	$\frac{S_e}{df_e}$	0.42	0.36	0.39
Statistic F	$\frac{\bar{S}_A}{\bar{S}_e}$	13.72 **	9.43 *	2.41
$F_{0.01}(df_A, df_e)$	Checklist	7.59	10.90	10.90
$F_{0.05}(df_A, df_e)$	Checklist	4.07	5.14	5.14

Note: “***” means that the effect of this level on the test results was highly significant; “**” means that this level had a significant effect on the results of the test.

In a further F-test, if $F < F_{0.05}$, it means that this level had a nonsignificant effect on the results of the test. If $F_{0.05} < F < F_{0.01}$, it means that this level had a significant effect on

the results of the test (indicated by “*” in the following tables). If $F > F_{0.01}$, it means that the effect of this level on the test results was highly significant (indicated by “***” in the following tables).

The F -test can only test that the variation in the results stems mainly from the variation between levels. It cannot detect whether the difference between the level means is significant. To address this issue, multiple comparisons were further used to determine the significance of the difference between two horizontal means. As shown in Tables 10 and 11, Factor C was not subjected to multiple comparisons because its effect was not significant.

Table 10. Multiple comparison table of average shear strength under number of rebars.

Group Number	Number of Rebars	Level	Average $\bar{x}_i$	$ \bar{x}_i - 9.47 $	$ \bar{x}_i - 9.02 $	$ \bar{x}_i - 8.46 $
B1G1	1	A_1	6.32	3.14 *	2.69 *	2.14 **
B1G2	2	A_2	8.46	1.00	0.55	
B1G3	3	A_3	9.02	0.45		
B1G4	4	A_4	9.47			

Note: The minimum significant differences at the significant level of 0.05 and 0.01, respectively, were $LSD_{0.05} = t_{0.025}(df_e)S_{\bar{x}_i - \bar{x}_j} = 1.225$, $LSD_{0.005} = t_{0.005}(df_e)S_{\bar{x}_i - \bar{x}_j} = 1.7822$. “***” means that the effect of this level on the test results was highly significant; “**” means that this level had a significant effect on the results of the test.

Table 11. Multiple comparison table of average shear strength under rebar diameter.

Group Number	Rebar Diameter	Level	Average $\bar{x}_i$	$ \bar{x}_i - 6.32 $	$ \bar{x}_i - 7.46 $
B2G1	C8	B_2	8.46	2.14 **	1.00
B2G2	C10	B_3	7.46	1.14	
B2G3	C6	B_1	6.32		

Note: The minimum significant differences at the significant level of 0.05 and 0.01, respectively, were $LSD_{0.05} = t_{0.025}(df_e)S_{\bar{x}_i - \bar{x}_j} = 1.207$, $LSD_{0.005} = t_{0.005}(df_e)S_{\bar{x}_i - \bar{x}_j} = 1.828$. “***” means that the effect of this level on the test results was highly significant.

Table 9 demonstrates the formula and results of the calculations. The quantitative results of the completed analysis were used to accurately rank the strong and weak effects of the three factors and ten levels. These data tested the conclusions of the previous section.

4. Number of rebars

It is described in Table 9 that the F value of Factor A was 13.72, which was higher than $F_{0.01}$ and $F_{0.05}$. This indicated that the effect of the number of rebars on the improvement in the shear strength was highly significant. As can be seen from Table 10, among the four levels of Factor A, $|\bar{x}_1 - 9.47| = 6.44$ and $|\bar{x}_1 - 9.02| = 2.69$. So A_1 was significantly higher than A_2 , A_3 , and A_4 , and more compared to the untreated (S). In addition, the enhancement effect of the number of rebars 4 (B1G4) was significantly higher than others.

5. Rebar diameter

The F value of Factor B was 9.43, which was less than $F_{0.01}$ and greater than $F_{0.05}$, indicating that the rebar diameter had a significant effect on the enhancement of the shear strength. Of the three levels of Factor B, $|\bar{x}_1 - 6.32| = 2.14$, $|\bar{x}_1 - 8.46| = 1.00$, so B_2 was significant over B_3 and the others were not.

6. Spacing of the bars

The F value of Factor C was 2.41, which was less than $F_{0.01}$ and $F_{0.05}$, suggesting that the spacing of the bars had no significant effect on the improvement in the shear strength. In general, when using the rebar implantation method to strengthen the OC, the spacing of the bars should satisfy the specifications and combine the actual conditions.

7. Summary

Combined with the above analysis, the interface shear performance was most influenced by the number of rebars. As two ways to increase the rebar planting rate, the former

was a bit more significant. This was because a growth in the number of rebars not only quickly improved the planting rate, but also made the arrangement more stable. In a multiple rebar arrangement, the rebars were able to affect a larger area of concrete, resulting in a more uniform force. Smaller diameters but a greater number could be used while keeping the rebar planting rates constant.

The weakest influence was the spacing of the bars. The effects of the anchor groups on the rebar–concrete transfer mode should be considered during the structural design.

4. Shearing Mechanism and Comparison of Results

4.1. Analysis of the Shear Resistance Mechanism

Currently, the shear theories of shear resistance are based on the shear-friction theory proposed by Birkeland [38], and on this basis, Randle [39] established the formula for shear resistance. The currently accepted shear bearing capacity of the interface consists of three components: the force of cohesion $V_{u(\text{coh})}$, the friction $V_{u(f)}$ and the pin force $V_{u(\text{da})}$ [40].

$$v_u = \frac{V_{u(\text{coh})} + V_{u(f)} + V_{u(\text{da})}}{A_{\text{cv}}} \quad (A_{\text{cv}} \text{ is the interface area})$$

The shear force transfer mechanism [21] is shown in Figure 13. When the interface undergoes relative slip u due to shear, the rebars are also driven and thus generate tension F_t and create an angle α with the rough interface. At this point, the component $F_t \cos \alpha$ of the tensile force in the direction parallel to the interface is the pin force $V_{u(\text{da})}$. Its component $F_t \sin \alpha$ in the direction perpendicular to the interface is the pressurizing effect [39]. The concrete is produced under pressure because of the extrusion of the tensile rebar. The friction $V_{u(f)}$ generates on the rough bonding interface, which is $\mu F_t \sin \alpha$, its magnitude depends on the roughness and pressure magnitude. Moreover, it decreases slightly with an increasing slip. In addition to the shear resistance, the interface itself has an adhesive cohesion $V_{u(\text{coh})}$, consisting of Van der Waals forces, chemical action, interface tension and mechanical occlusion [41]. The above constituted the shear resistance of the interface.

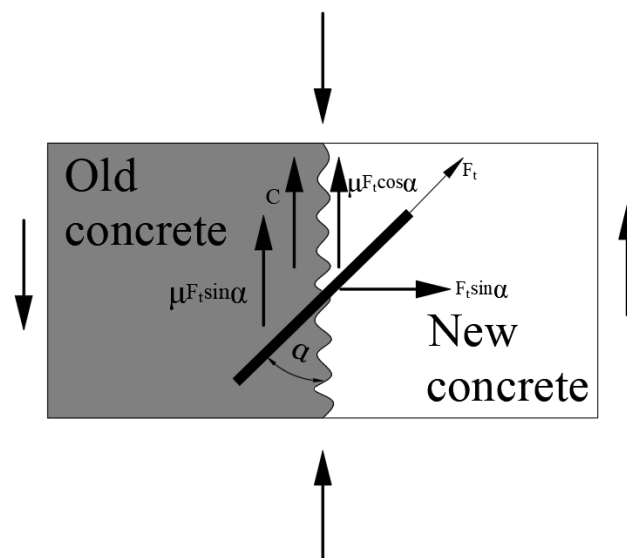


Figure 13. Shear force transmission diagram.

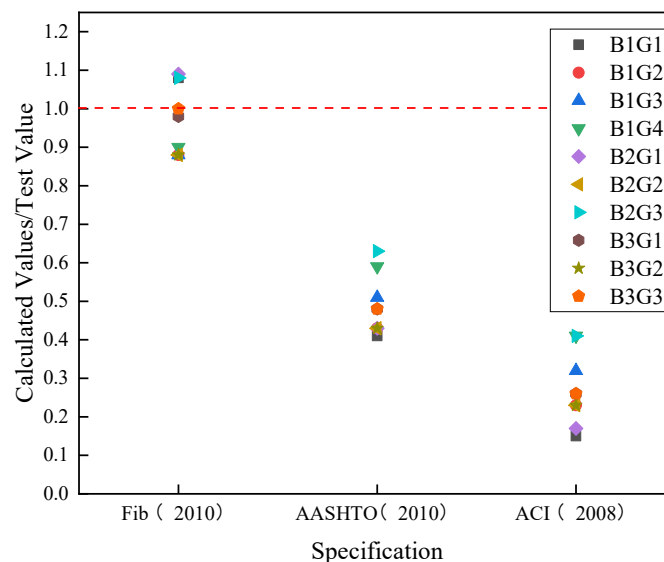
4.2. Analysis of the Application of Existing Shear Strength Calculation Formulae

Most of the existing well-established shear formulas are mostly based on the above theory, refined and extended in terms of quantification of roughness, correction of friction coefficient and different types of concrete [40]. Table 12 has shown some representative formulas for calculating the shear strength of the interface of new and old concrete at present.

Table 12. Typical formula for calculating the shear strength of the bonding interface of new and old concrete.

Specification	Calculation Formula	Annotation
Fib Model Code for Concrete Structures (2010) [42]	$\tau_u = \tau_a + \mu(\rho k_1 f_y + \sigma_n) + k_2 \rho \sqrt{f_y f_c}$	τ_a , c —Interface cohesion μ —Interface friction coefficient ρ —Bonding rate of interface σ_n —Normal stress caused by interface shear
AASHTO LRFD Bridge Design Specifications (2010) [43]	$V_u = c A_{cv} + \mu(A_{sv} f_y + P_c)$	k_1 —Bending coefficient k_2 —Interaction coefficient between steel bar and concrete under dowel action f_y —Tensile strength of rebar f_c —Axial compression strength of concrete A_{sv} , A_{vf} —Total cross-sectional area of interfacial shear rebar A_{cv} —Interfacial area P_c —Positive interface pressure caused by external force
ACI 318-08 (2008) [44]	$V_u = \mu A_{vf} f_y$	

Figure 14 expresses the ratio of theoretical value to test value. It can be seen that The Fib (2010) takes into account the coupling between friction and pin force. The shear strength value is considered to be equal to the algebraic sum of the force of cohesive, friction and pin force. It is worth mentioning that these three do not reach the maximum value at the same time during the shear resistance. The expression in the form of multiplication by a subfactor in front can better predict the shear strength in this test. For the FLWCC, the calculated value of Fib (2010) was conservative and the maximum error was controlled at about 25%, which was a better fit. The theoretical value of the AASHTO (2010) was lower and would be used to predict this test with some error. The reason was that although it used friction and pin force to resist external loads at the same time, the values of the bonding force and friction coefficient are taken roughly. The smallest theoretical value of the ACI (2008) was due to the fact that only the pinning action of the rebar is considered, ignoring the contribution of the concrete itself.

**Figure 14.** Calculated values/test values.

It can be seen that all three specifications take the total cross-sectional area of A_{cv} as a major variable, which also corresponds to the results of the factor analysis in Section 3 of this paper. In other words, the shear strength is linearly related to the rebar planting rate and the rebar diameter. Definitely, there are Chinese specifications that codify the spacing

of bars as an independent variable in the formula. Such as JTG/T J22-2008 [45]. But its calculated value differed greatly from the test value. The reason, as analyzed above, was that the effects of anchor groups affect mainly the way in which the forces are transferred between the rebar and the concrete. Thus the effect of this factor was difficult to quantify in different situations with different members, loading methods and different usage scenarios.

5. Conclusions

Based on the above results and discussions, the following conclusions can be drawn:

1. The form of shear damage at the unstrengthened FLWCC–OC interface was brittle damage. But the damage form becomes ductile after rebar implantation. This showed that rebar implantation could increase the ductility of the interface of FLWCC–OC.
2. The rebar planting rate had a significant effect on the interfacial shear strength. The strength reached a maximum of 9.47 MPa when the rate was 0.894%. Rebar was not effective at low rates. The interfacial shear strength was positively correlated with the number of rebars, which increased and then decreased with the increase in rebar diameter. The number of rebars enhanced the shear strength better than the rebar diameter for the same planting rate.
3. The shear strength was related to the spacing of the bars. In this test, the strength was maximum when the spacing was 80 mm. Too much or too little spacing affected the way the rebars transmitted force and thus the bonding action of the rebars.
4. The one-way ANOVA was used to analyze the effects of the number of rebars, rebar diameter and spacing of the bars on the interfacial shear strength. The results showed that the most obvious effect of the number of rebars was followed by rebar diameter, and the effect of the spacing of the bars was less significant.
5. The calculated strength of the Fib (2010) formula agreed well with the measured values in this text. It was recommended to refer to the Fib (2010) to calculate the interfacial shear strength of the rebar implantation when strengthening OC with FLWCC.

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Article

Nonlinear Numerical Analysis and Restoring force Model of Composite Joints with Steel Reinforced Recycled Concrete Columns and Steel Beams

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Abstract: Cyclic loading tests were conducted for eight composite joints (five interior joints and three exterior joints) with steel reinforced recycled concrete (SRRC) columns and steel beams under cyclic loading. The axial compression ratio and recycled coarse aggregate (RCA) replacement percentage were considered as the main design parameters for the above composite joints. The seismic behavior of the composite frame joints with (SRRC) columns and steel beams were studied by numerical simulation and the constitutive models of recycled aggregate concrete (RAC), steel and reinforcement were given to the corresponding elements. The experimental results were used to validate the finite element (FE) model. The results indicated that the FE model can accurately predict the deformation and stress load-strain curves of the composite joints. Then, a FE parameter analysis was carried out on the seismic behavior of the composite joints. The RAC strength, steel strength and column steel web thickness were considered as the main parameters. Results pinpointed that the increase in RAC strength and steel web thickness of the columns significantly affected the horizontal bearing capacity of the composite joints. However, with the increase in RAC and steel strength, the deformation ability and ductility of composite joints were almost unaffected. The reasonable increase in column steel web thickness can improve the seismic ductility of composite joints, evidently. On this basis, considering the stress characteristics of joint specimens, a four broken line restoring force model for steel reinforced recycled concrete column steel beam composite frame joints was established. This model includes the skeleton curve model, the stiffness degradation pattern and hysteresis rules. The calculation curve was in good agreement with experimental curves, which can well reflect the hysteretic behavior of joints subjected to low cyclic repeated loading. The above results can be used as a reference for the seismic design of composite frame joints.

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Keywords: steel reinforced recycled concrete columns; steel beams; joint; finite element analysis; restoring force model

1. Introduction

With the acceleration of urbanization, a large amount of concrete demand leads to long-term over-exploitation of natural sand and stone materials, resulting in huge energy and natural resource consumption, and at the same time, the demolition of old buildings, new building construction and other human activities generated by the building waste accumulation. In addition, various natural disasters cause a large number of buildings to collapse and become damaged, and produce a large amount of construction waste, which seriously hinders the sustainable development strategy of the ecological environment, and bring serious problems to society, the economy and the ecological environment. Therefore, it is of great scientific significance and application value to develop a new type of structure system with good seismic performance and ecological environment friendliness. The emergence of recycled concrete can effectively solve the above problems [1]. It can reprocess waste concrete, recover its original mechanical properties to the greatest extent

and form new building materials. This technology can not only reuse limited resources, but also solves some environmental problems. The promotion and application of recycled concrete technology can transform the traditional concrete production mode into the track of sustainable development.

The section steel recycled concrete column is a kind of combination form, a new type of composite member formed by configuring section steel, matching certain structural longitudinal bars and stirrups and, finally, by pouring recycled concrete. In steel structures, steel beams have good flexural performance. Compared with concrete beams, steel beams are lighter and can effectively reduce the section size of members, thus increasing the use space of the structure. In view of this, and in view of the characteristics of steel recycled concrete columns with good seismic performance and high bearing capacity, combined with the advantages of steel beams with good mechanical properties and simple construction, this paper puts forward a new structure system, the composite frame structure of steel recycled concrete columns and steel beams (SRRC). Numerous studies have been conducted on the performance of recycled aggregate concrete (RAC), including its preparation methods, physical and mechanical properties, durability, fatigue and bonding properties [2–4]. Ma et al. conducted experimental research and a theoretical analysis of RAC member sand structures [5–9]. Their results showed that the mechanical properties of RAC and its structures are inferior to those of ordinary concrete. This nature of RAC limits its engineering application, particularly in seismic structures, to a certain extent. Considering the good mechanical properties of steel and concrete composite structures, several scholars have attempted to apply RAC to composite structures to improve the mechanical properties of RAC structures and structural members. SRRC structures are among these steel and RAC composite structures [10–12]. In previous studies, SRRC columns that were tested under cyclic loading presented good seismic performance and high bearing capacity, and the results significantly promoted the application of RAC. As a new type of composite structure system, the restoring force model of the composite joints of SRRC was the basis for the analysis of the elastoplastic time history of the structure. The restoring force model should fully reflect the mechanical properties of the structure or component, such as strength decay, stiffness degradation, deformation ductility, energy dissipation capacity, damage evolution and so on, which was an important basis for the seismic performance analysis of the structure. The actual restoring force characteristic curve was very complex, so it was difficult to apply it directly to the analysis of the elastic–plastic seismic response of the structures. In order to facilitate the mathematical description and engineering application, the restoring force characteristic curve obtained by the test was abstracted and simplified in the form of mathematical modeling, and then the restoring force model of the structure or component was obtained. The research on the restoring force characteristics of SRRC was not mature. Only Xue et al. discussed the restoring force model of such joints under plane stress [13,14]. There is still a lack of research results on the restoring force model of SRRC.

Accordingly, the seismic performance of the composite joints has been further studied by the finite element method. The validity and accuracy of the refined numerical model were verified through experimental data. In addition, the influence of design parameters on the mechanical performance of composite joints was also investigated. On this basis, considering the failure process of the joints, a four-fold skeleton curve model of the composite joints was established by the regression fitting method. Moreover, through analyzing the stiffness degradation law and hysteretic rule in the whole loading and unloading process, a hysteretic curve restoring force model of the composite joint was proposed, which can provide reference for the engineering application of the joint.

2. Axial Compression Tests of the Composite Joints

2.1. Research Significance

The preparation and use of a large number of concrete materials lead to the long-term over-exploitation of natural sand and stone materials, resulting in huge energy and resource consumption. At the same time, the construction waste generated by the demolition of old

buildings, new buildings and other human activities has piled up. In addition, earthquakes, tsunamis and floods and other natural disasters caused a large number of building collapses, a large amount of construction waste, etc., which to the sustainable development of our country has brought more and more serious social, economic, ecological and environmental problems. At present, construction waste is generally treated by stacking and burying, and its comprehensive utilization rate is too low. From the above, we can see that, on the one hand, it is the over-exploitation of natural non-renewable resources such as sand and stone. On the other hand, a large amount of construction waste in the natural and ecological environment is caused by serious pollution. Therefore, how to deal with and utilize these large amounts of construction wastes reasonably and effectively, so as to realize the resource-saving and environment-friendly society and the sustainable development of the economy, has become one of the problems that governments of all countries must face and solve. In view of the good seismic performance and high bearing capacity of the steel recycled concrete column, and the advantages of the steel beam, such as good mechanical performance, being light weight, of easy construction and being effective in the reduction of the section size of the members, a new structure, the composite structure of the steel recycled concrete column and steel beam, was proposed. Compared to traditional reinforced concrete structures and steel structures, the combination structure not only has high carrying capacity, good seismic performance, good durability and fire resistance, saves steel and other advantages, but also has the remarkable characteristics of renewable concrete environmental protection, and meets the requirements of our sustainable development; therefore, it has broad application prospects for development.

2.2. Design and Production of Specimens

The low cyclic loading test and shear strength of composite joints with SRRC columns and steel beams were studied in detail by the authors [15]. Eight joints were designed, including five interior joints and three exterior joints. The effects of the recycled coarse aggregate (RCA) replacement ratio and axial compression ratio on the seismic performance of the composite joints were mainly considered [16]. The design parameters of the composite joints are shown in Table 1. The details of the size and reinforcement of composite joints are presented in Figure 1. The composite joints production and completion process is shown in Figure 2. The mix of RAC materials is demonstrated in Table 2. The design parameters of each joint are shown in Table 3.

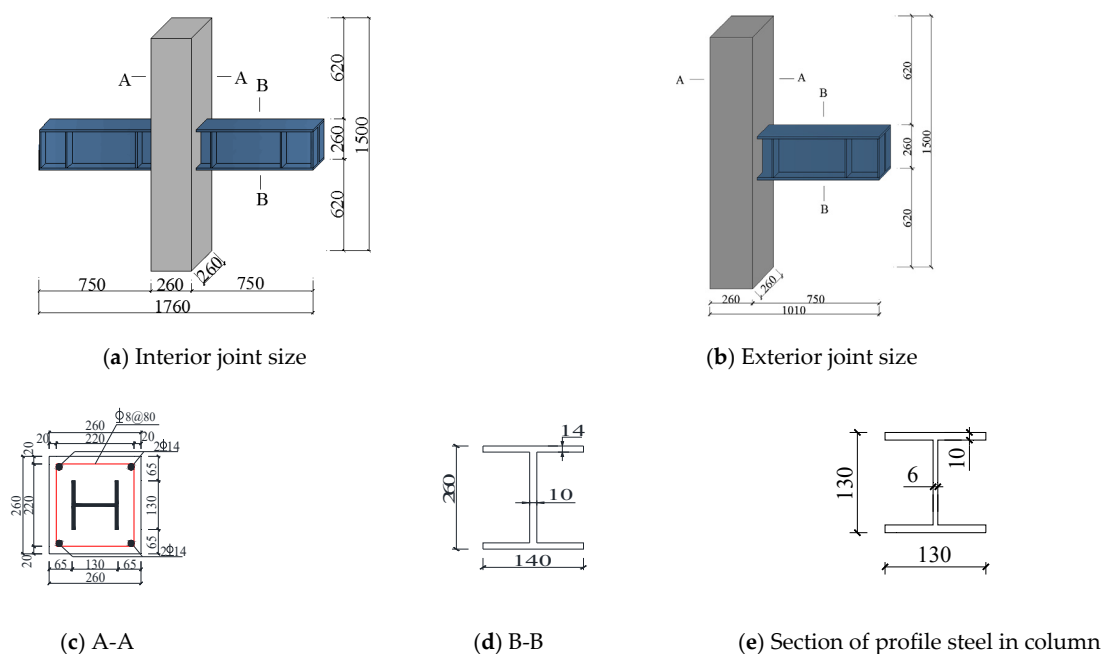
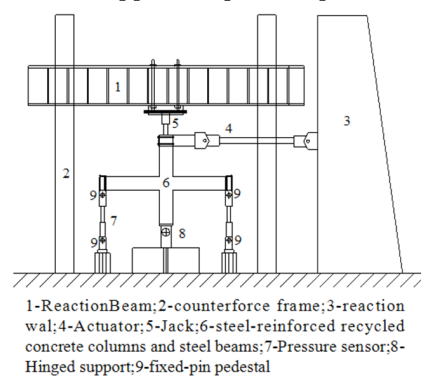
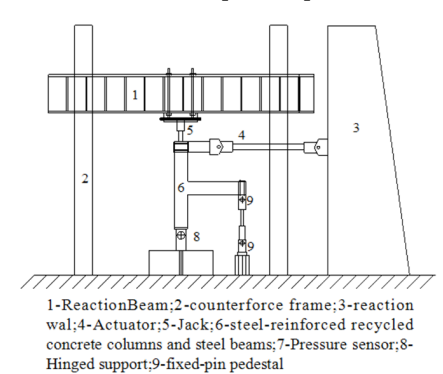


Figure 1. Size and distributed steel of specimens (mm).

Table 1. Design parameters of specimens.

Design Parameter Level	Level 1	Level 2	Level 3
RCA replacement percentage $r/\%$	0%	50%	100%
Axial compression ratio n	0.18	0.36	0.54

**(a)** Reinforced steel frame**(b)** Support template of specimen**(c)** Completed specimens**(d)** Interior joint**(e)** Exterior joint**Figure 2.** Schematic diagram of main manufacturing process and test device of test piece.**Table 2.** Mix ratio of RAC materials.

RAC Strength Grade	RCA Replacement Percentage $r/(\%)$	Mass of Each Component in Concrete per Cubic Meter (kg)					
		Water–Cement Ratio	Cement	Sand	NCA	RCA	Water
C40	0	0.43	464	585	1187	0	195
	50%	0.43	464	585	593.5	593.5	195
	100%	0.43	464	585	0	1187	195

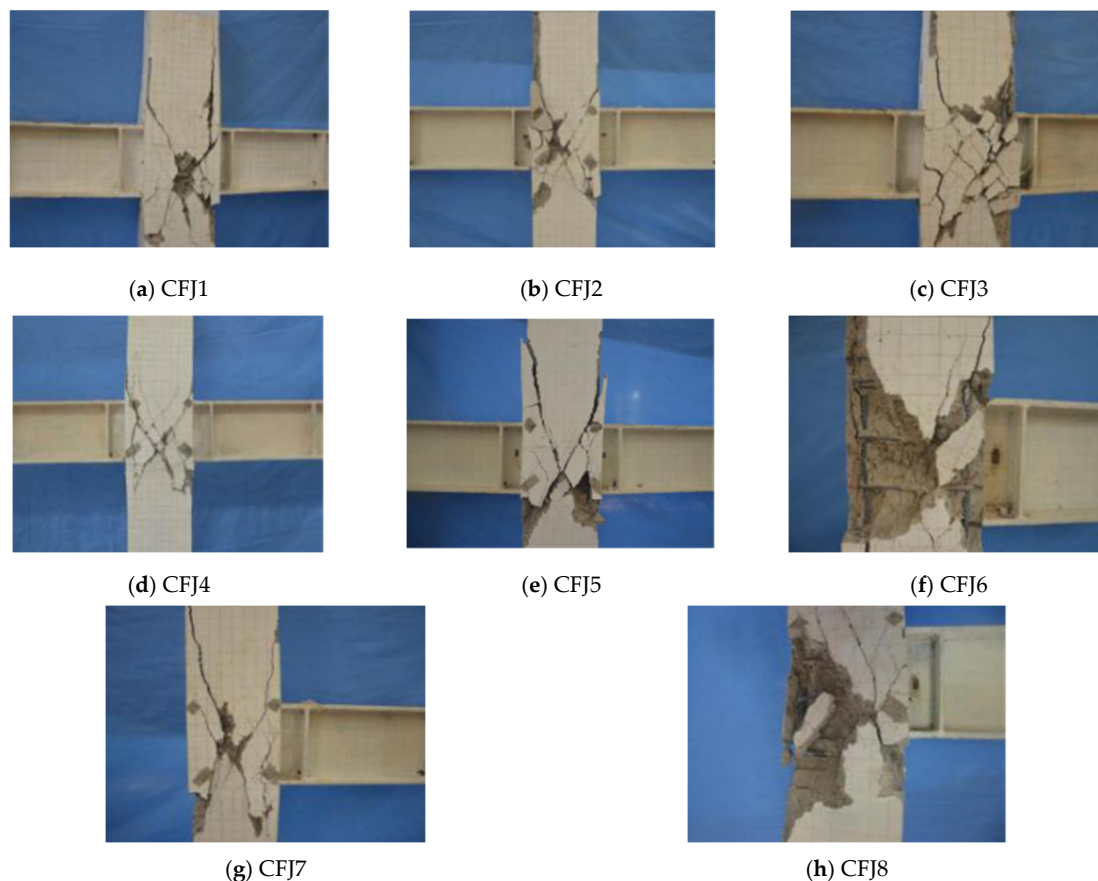
Table 3. Design parameters of joint specimens.

Specimen Number	RAC Strength Grade	RCA Replacement Percentage r (%)	Axial Compression Ratio n	Profile Steel Ratio ρ_a (%)	Joint Forms
CFJ1	C40	0	0.36	4.8	Interior joint
CFJ2	C40	50	0.36	4.8	Interior joint
CFJ3	C40	100	0.36	4.8	Interior joint
CFJ4	C40	100	0.18	4.8	Interior joint
CFJ5	C40	100	0.54	4.8	Interior joint
CFJ6	C40	100	0.18	4.8	Exterior joint
CFJ7	C40	100	0.36	4.8	Exterior joint
CFJ8	C40	100	0.54	4.8	Exterior joint

Note: $r = m_{\text{RAC}}/m$, m_{RAC} is the weight of RAC; m is the total weight of the aggregates; $\rho_a = A_a/A$, A_a is the area of the profile steel in the column; A is the total area of the cross section of the column.

2.3. Failure Modes of Specimens

From the experimental results, the typical shear failure of each composite joint occurs under low cyclic load. There were obvious shear deformation and “X” shaped cross-diagonal cracks that appeared in the joint core region. From the influence of the two designed parameters on the cracking load of joints, the RCA replacement rate has little effect, while the axial compression ratio has a great influence on the cracking of joint core region. The cracking load of joints with different RCA replacement rates was approximately 70 kN, and there was no significant difference in the crack development process and distribution characteristics. The cracking load of joints with a large axial compression ratio was significantly greater than that of joints with a small axial compression ratio. The failure characteristics of CFJ1–CFJ8 are recorded in Figure 3.

**Figure 3.** Failure modes of joints.

2.4. Comparison with Previous Studies

In order to verify the accuracy of the test and the correctness of the results, the following conclusions are drawn by comparing the experimental results and analysis in the previous literature:

- (1) The failure modes of the joints in the reinforced steel recycled concrete frame are similar to those of the ordinary reinforced steel concrete frame, both of which are shear failure in the core area of the joints, and the shear deformation in the core area is significant [17].
- (2) The failure modes of the specimens under repeated cyclic loads were the same, and obvious “X”-shaped cross-oblique cracks were formed along the diagonal. The failure modes are shear baroclinic failure, flexural shear failure and bending failure. However, due to the different structures of the specimens, the results can only serve as a reference [18].
- (3) The hysteretic curves of the joints of the recycled steel concrete frame are roughly spindle-shaped, which are similar to the hysteretic curves obtained in the references, and the mechanical properties are in line with the mechanical properties of the steel frame and reinforced concrete frame [19].

3. Model Descriptions of the Composite Joints

3.1. Constitutive Models of the Materials in the Joints

(1) Constitutive relationship of steel

ABAQUS software 6.16 (Dassault Systèmes, Vélizy-Villacoublay, France) enables a variety of constitutive models for steel materials. At present, the widely used constitutive models include elastic–plastic, whole curve, bilinear and three-fold line models. The steel used in the test was Q235 low carbon steel, which belongs to the isotropic elastic–plastic material with an obvious yield point. The stress-strain curve generally adopts a simplified secondary plastic flow model. This model encapsulated that the longitudinal stress of the steel directly enters the strengthening stage after reaching the yield stress, while the strength of the plastic flow stage was unchanged. The secondary plastic flow model can not only consider the strengthening effect of steel, but also reflects the real material characteristics. The stress-strain curve of steel is shown in Figure 4. The mathematical expression of the stress-strain is determined by Equation (1):

$$\sigma_i = \begin{cases} E_s \varepsilon_i & \varepsilon_i \leq \varepsilon_y \\ f_s & \varepsilon_y < \varepsilon_i \leq \varepsilon_{st} \\ f_s + \zeta E_s (\varepsilon_i - \varepsilon_{st}) & \varepsilon_{st} < \varepsilon_i \leq \varepsilon_u \\ f_u & \varepsilon_i > \varepsilon_u \end{cases} \quad (1)$$

where σ_i is equivalent stress of steel, f_s is the yield strength of steel, f_u is the ultimate strength of steel, E_s is elastic modulus of steel, E_t is the tangent modulus of steel, ζ is the strengthening coefficient $\zeta = 1/216$, ε_i is equivalent strain of steel, ε_y is the strain of steel at yield, ε_u is the strain at the limit of steel, $\varepsilon_{st} = 12\varepsilon_y$, $\varepsilon_u = 120\varepsilon_y$.

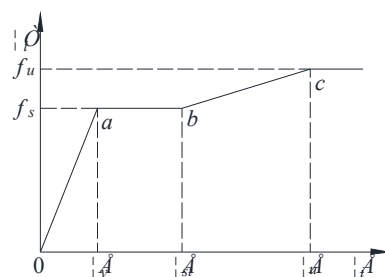


Figure 4. Stress-strain curve of steel.

The Poisson's ratio of steel is assumed to be:

$$v_s = \begin{cases} 0.285 & \varepsilon_i \leq 0.8\varepsilon_y \\ 1.075(\sigma_i/f_s - 0.8) + 0.285 & 0.8\varepsilon_y < \varepsilon_i \leq \varepsilon_y \\ 0.5 & \varepsilon_i > \varepsilon_y \end{cases} \quad (2)$$

The above steels all adopt Miss yield criterion, which is defined as a relatively general yield criterion. In a 3D space, the Miss yield surface is a cylindrical surface with $\sigma_1 = \sigma_2 = \sigma_3$ as the axis. On a 2D plane, the yield surface is an ellipse. Any stress state inside the yield surface is elastic, and any stress state outside the yield surface will cause yield. Miss yield criterion is suitable for materials with good toughness.

The spatial representation of Miss yield criterion is shown in Figure 5. The mathematical expression of equivalent stress is:

$$\sigma_s = \sqrt{\frac{1}{2}[(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2]} \quad (3)$$

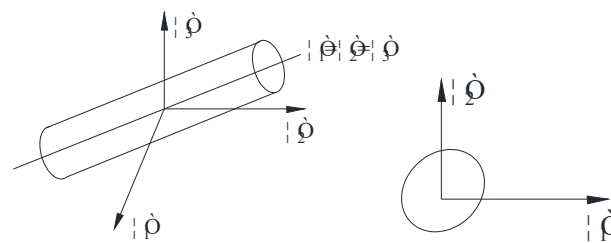


Figure 5. Schematic diagram of Miss yield criterion.

In the formula: $\sigma_1, \sigma_2, \sigma_3$ are three principal stresses.

(2) Hysteretic curve model of reinforcement

Existing studies have stated that the direct manifestation of the bond-slip between concrete and reinforcement in structural hysteretic curve analysis is the emergence of the “kneading” phenomenon. As a result, how to accurately describe the kneading phenomenon is one of the difficulties in numerical analysis. After comparison with the VECCHIO model of reference [20], the M-P model of reference and hysteretic curve model of the reference based on CLOUGH frame [21,22], Professor Zihu Fang of Shenzhen University established a hysteretic model of reinforcement that considered the bond-slip effect [23,24]. In this paper, the steel model was selected to simulate the hysteretic behavior of composite joints. The model is described in Figure 6.

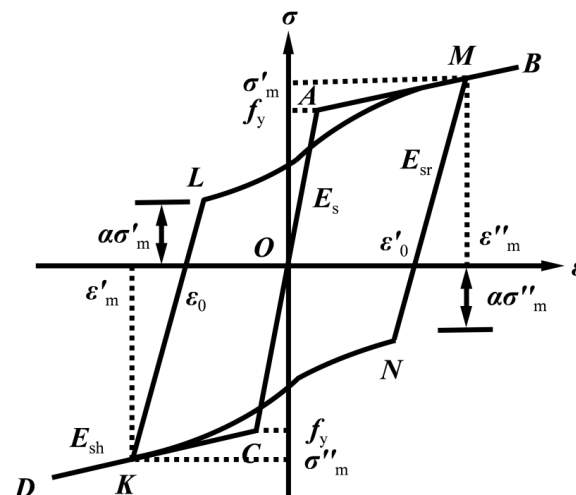


Figure 6. Present hysteretic model of bar.

(3) RAC material

The RAC was mainly distributed in the column, while the SRRC column was mainly composed of RAC, reinforcement and section steel. Therefore, the RAC part will be subject to different forms of constraints, which can be roughly divided into the section steel constraint part and reinforcement constraint part. The strength of the two constraints on RAC was different, so the reinforcement constraint area was called the weak constraint area, and the section steel constraint area was called the strong constraint area [25]. Figure 7 shows the distribution details of the SRRC column section constraint areas. For the convenience of the model, referring to the simplified method of Ellobody [26], the arc region in Figure 7 is simplified as a straight line. Based on this linear division method, the section constraint area of the SRRC column was divided as shown in Figure 8. The stress-strain curves of different concrete areas are presented in Figure 9.

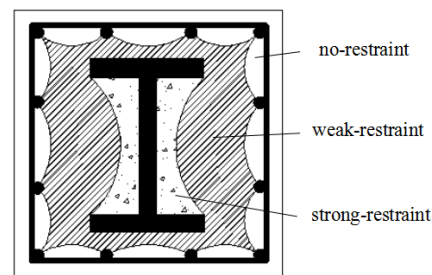


Figure 7. Section divisions of composite column.

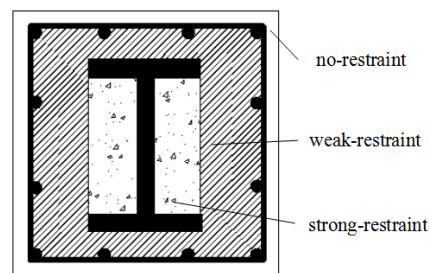


Figure 8. Regions simplified for composite sections.

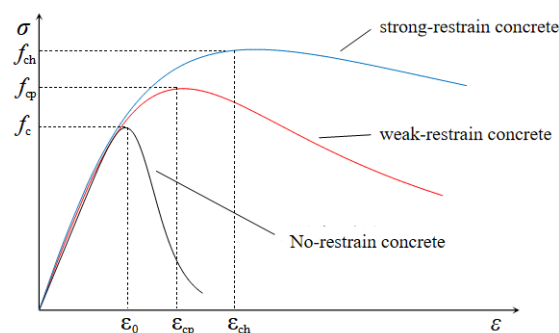


Figure 9. Stress-strain curves for concrete.

f_c is the axial compressive strength of RAC and ε_0 is the peak strain of RAC; f_{cp} is the peak stress of concrete in the weak constraint area and ε_{cp} is the peak strain of concrete in the weak constraint area; f_{ch} is the peak stress of strong constraint concrete and ε_{ch} is the peak strain of strong constraint concrete.

3.2. Definition of Concrete Damage

The plastic damage model is provided in the simulation of the concrete reciprocating load. Under the reciprocated load, the damage index of the concrete model gained a

significant impact on the simulation results [27]. In order to improve the accuracy of ABAQUS in a nonlinear calculation, domestic and foreign scholars have included the damage of concrete in the FE simulation [28–30]. At present, there are many damage models for concrete. The damage factor was calculated by the Mander method. The formula was stated as follows:

$$d_c = 1 - \frac{E_{sec2}}{E_c} \quad (4)$$

Determined peak strain ε_{cc} and reverse unloading point $(\varepsilon_{un}, \sigma_{un})$ of concrete are the key to solving the damage factor by the Mander method.

The yield function and plastic potential function of materials are the critical factors affecting the performance of RAC. This paper uses meridian plane (p, q) and partial plane (q, θ) to represent it. The yield condition criterion is scheduled as follows:

$$\sigma_1 \leq 0 : f = q - 3\alpha p + \gamma\sigma_1 - (1 - \alpha)\sigma_{ci} = 0 \quad (5)$$

$$\sigma_1 > 0 : f = q - 3\alpha p + \beta\sigma_1 - (1 - \alpha)\sigma_{ci} = 0 \quad (6)$$

Figures 10 and 11 show the plane stress yield surface and the stress yield surface on the deviatoric plane of the concrete plastic damage model.

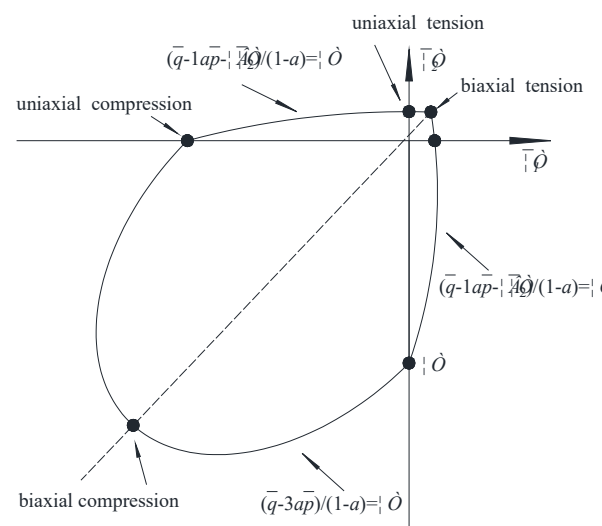


Figure 10. Yield surface in plane stress state.

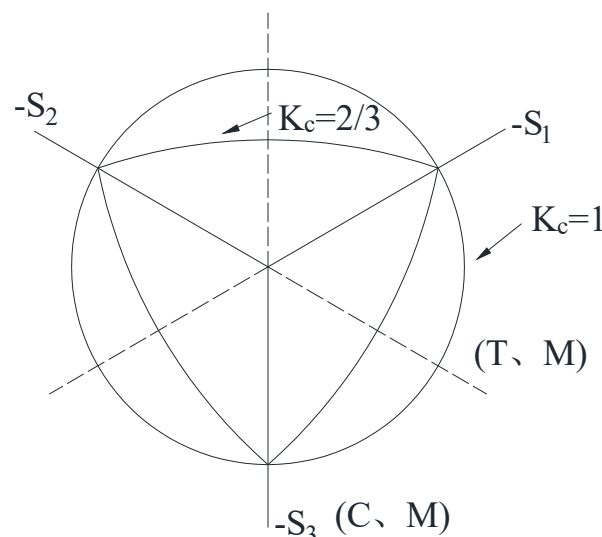


Figure 11. Yield surface in deviatoric stress state.

3.3. Interaction between the Materials

The steel reinforced part was embedded into the concrete through the embedded function in ABAQUS. There were no anti-sliding structural measures such as studs between the steel and RAC. Consequently, it was necessary to define the interaction between steel reinforced and RAC. Kinematic contact was selected for the normal direction contact of the two contact surfaces, which was also called “hard” contact. In other words, in addition to mutual contact, there was no interaction force between the surfaces as separation after contact was allowed. The Coulomb friction model was adopted for the bond-slip in the tangential direction, while the friction coefficient was set to 0.25 according to the research results [31,32].

3.4. Description of Mesh and Elements

The type of C3D8R element was adopted in the steel beam, built-in column steel and RAC, which was an eight-node hexahedron reduction integral solid element. Element type T3D2 was applied for the longitudinal reinforcement and stirrup in the composite column and the double-node three-dimensional truss element. The mesh size was 50 mm × 50 mm. The mixed model of these two element types not only improves the calculation accuracy, but also ensures the calculation speed to some extent. Figure 12 describes the FE model of the composite joints element mesh.

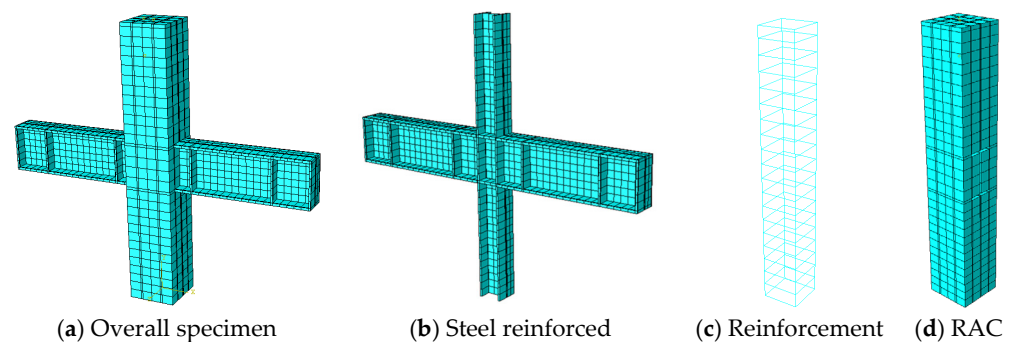


Figure 12. Mesh division of composite joints.

3.5. Axial Loading Method and Boundary Conditions

The overall FE model was modeled according to the size of the test. The boundary conditions of the model were consistent with the load conditions of the actual test. The horizontal reciprocated load and axial force were applied on the top of the column. The beam end was connected by connected rods, and the bottom of the column was hinged. The hybrid loaded method of force and displacement was adopted in the test. However, considering that it was easier to ensure calculation convergence by using displacement control in FE simulation, displacement control was used to replace the mixed control in FE simulation. Finally, the Newton iteration method was conducted to calculate the FE model.

4. Verification of FE Model on the Composite Joints

4.1. Comparative Analysis of Experimental Results and Simulation Results

Through the modeling calculation of CFJ1–CFJ8 composite joints, the load-displacement hysteretic curves and skeleton curve of each composite joint were obtained. The comparison between the test results and the FE simulation results is shown in Figures 13 and 14. For the skeleton curve, it is the track of the maximum peak value of horizontal force reached by each cyclic loading that reflects the different stages of force and deformation of components and their characteristics (strength, stiffness, ductility, energy dissipation and collapse resistance, etc.), and it is also an important basis for determining the characteristic points in the restoring force model.

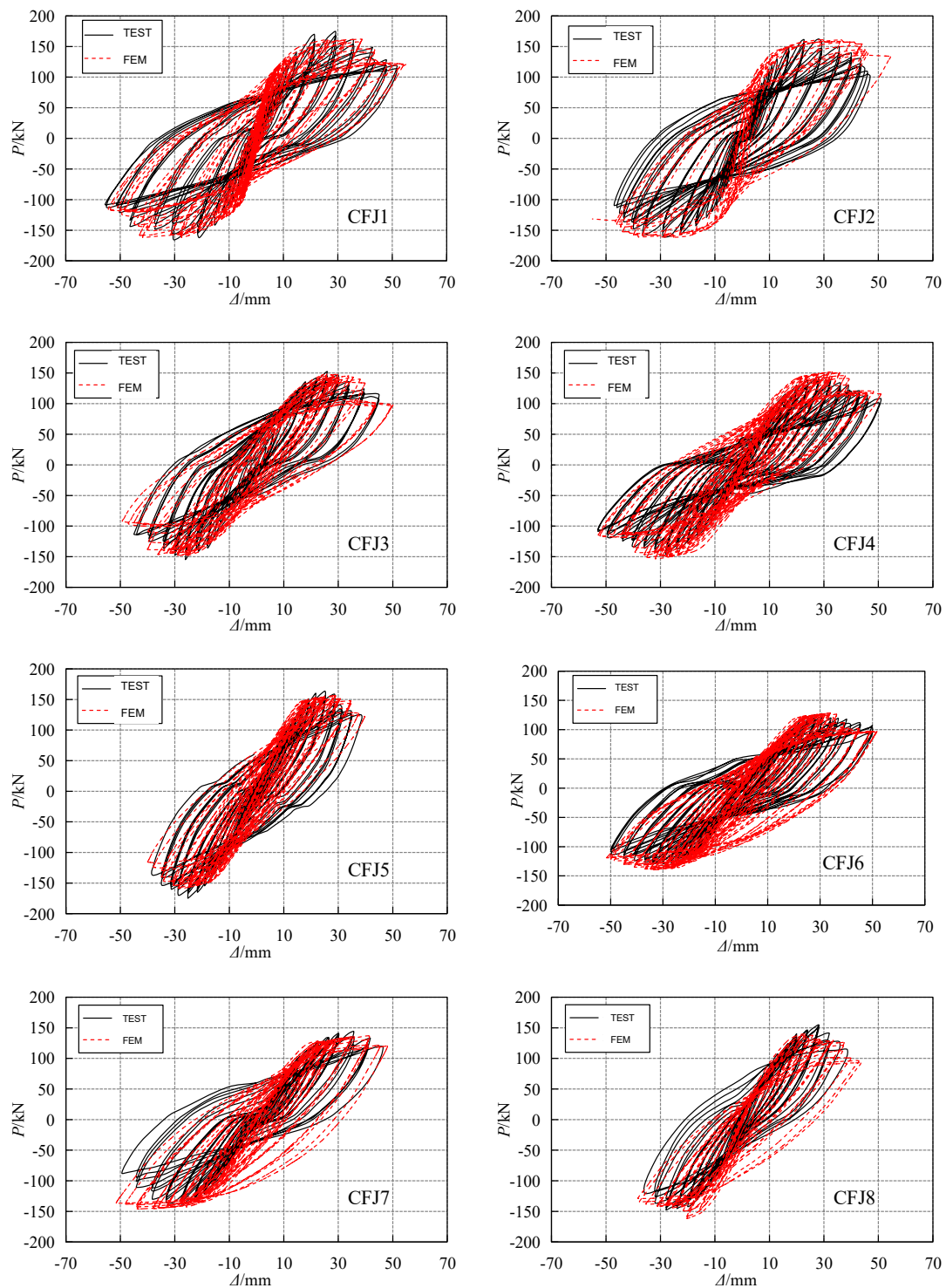


Figure 13. Comparison of test and simulation results of hysteretic curves of composite joints.

The hysteretic curves in Figure 13 and the skeleton curves in Figure 14 pinpoint that the simulation results of the hysteretic characteristics of the composite joints are basically in good agreement with the experimental results. However, compared with the experimental curves, the pinch effect of the simulated hysteretic curve of the exterior joints (CFJ6–CFJ8) is far from expectation. The unloading stiffness of the simulated curve is relatively larger. The main reason may be that the exterior joints are asymmetric structures. In addition, the cumulative damage of positive loading is greater than that of negative loading in the simulation. When the cumulative damage of positive loading reaches a certain degree, the

damage degree of continuous loading to the positive is greater than that of negative loading. As a result, the unloading stiffness of the simulated curve is larger. This phenomenon is also reflected in the skeleton curves of the exterior joints. The stiffness of the simulated curves under negative loading is slightly larger than that of the experimental curves. In general, the numerical simulation is in good agreement with the experimental results, which verifies the availability and correctness of the numerical simulation method, and can be used as a reliable numerical method for further analysis in the following paragraphs.

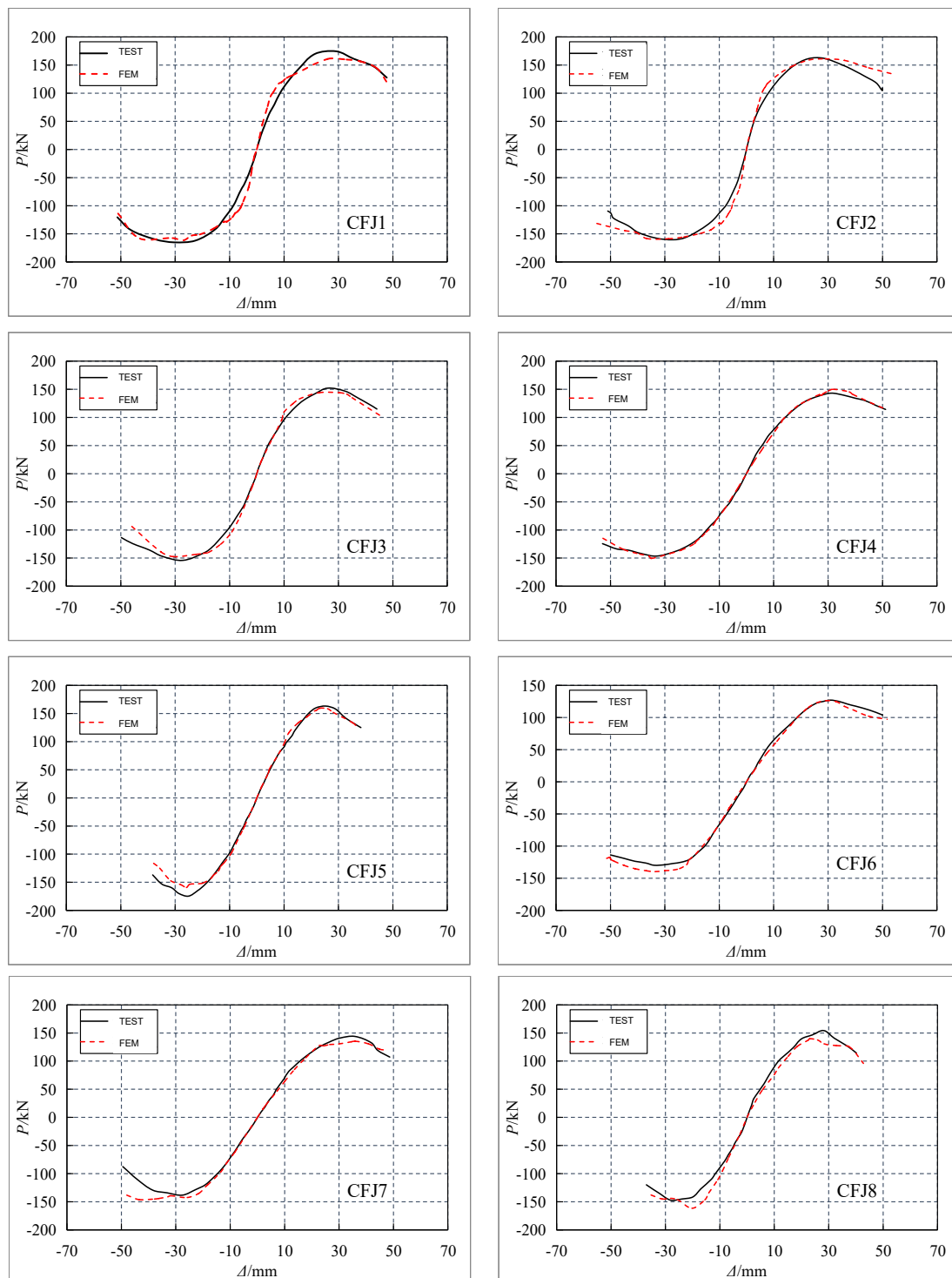


Figure 14. Comparison of test and simulation results of skeleton curves of composite joints.

4.2. Comparison of the Characteristic Values of the Composite Joints

Table 4 presents a comparative analysis on the load and corresponding displacement values of the composite joints obtained by numerical simulation and test at each characteristic point. It indicates that the load error of each characteristic point is basically within 6%, indicating that the bearing capacity consistency is fine, but some displacement differences are relatively large. The reasons may be as follows. First, there was a gap between each test equipment in the real test, resulting in a certain assembly error. Second, the bond-slip between steel bar, section steel and RAC in FE simulation was analyzed under the ideal condition, but in fact, the bond-slip of each part was quite complicated during the test process. Therefore, there is a certain difference in the bond-slip between the FE simulation and the real test. On the whole, there are some differences between the numerical analysis and the test results, but they are also within the acceptable range. It shows that the FE simulation results possess good accuracy.

Table 4. Comparison of test and simulation results of bearing capacities and displacements.

Specimen Number		Yield Load P_y/kN	Yield Displacement Δ_y/mm	Peak Load P_m/kN	Peak Displacement Δ_m/mm	Ultimate Load P_u/kN	Ultimate Displacement Δ_u/mm
CFJ1	TEST	131.15	13.37	170.34	27.04	144.77	44.66
	FEM	128.21	11.54	160.86	26.53	141.98	44.89
	$\delta/\%$	−2.24%	−13.69%	−5.57%	−1.89%	−1.93%	0.52%
CFJ2	TEST	121.65	12.79	162.14	28.67	137.42	41.32
	FEM	137.51	12.98	159.74	28.94	144.22	43.64
	$\delta/\%$	13.04%	1.49%	−1.48%	0.94%	4.95%	5.61%
CFJ3	TEST	113.67	12.94	153.19	27.28	130.21	40.51
	FEM	103.27	12.89	145.19	31.87	136.06	39.68
	$\delta/\%$	−9.15%	−0.39%	−5.22%	16.83%	4.49%	−2.05%
CFJ4	TEST	91.83	12.98	144.68	32.42	123.14	49.88
	FEM	94.82	12.15	150.64	35.08	116.86	50.51
	$\delta/\%$	3.26%	−6.39%	4.12%	8.20%	−5.10%	1.26%
CFJ5	TEST	108.76	12.25	168.93	25.05	143.59	34.94
	FEM	115.38	13.74	158.64	28.63	140.53	37.52
	$\delta/\%$	6.09%	12.16%	−6.09%	14.29%	−2.13%	7.38%
CFJ6	TEST	88.76	14.08	128.68	31.83	109.38	50.16
	FEM	90.82	14.32	135.56	33.61	120.85	49.82
	$\delta/\%$	2.32%	1.70%	5.35%	5.59%	10.49%	−0.68%
CFJ7	TEST	95.56	14.21	141.03	31.78	119.88	43.33
	FEM	90.66	13.15	140.16	35.71	115.26	44.17
	$\delta/\%$	−5.13%	−7.46%	−0.62%	12.37%	−3.85%	1.94%
CFJ8	TEST	108.06	13.39	150.85	27.45	128.22	35.01
	FEM	110.86	11.53	150.36	22.02	127.68	37.59
	$\delta/\%$	2.59%	−13.89%	−0.32%	−19.78%	−0.42%	7.37%

Note: Difference $\delta = (\text{simulation results} - \text{test results}) \times 100\% / \text{test results}$.

4.3. Deformation and Stress Characteristics

The deformation characteristics and failure modes of all composite joints obtained by FE analysis are basically consistent with the test results. Therefore, in order to avoid repeated statements, the results of interior joint CFJ5 and exterior joint CFJ7 are representative and typical, which are exhibited in Figures 15 and 16.

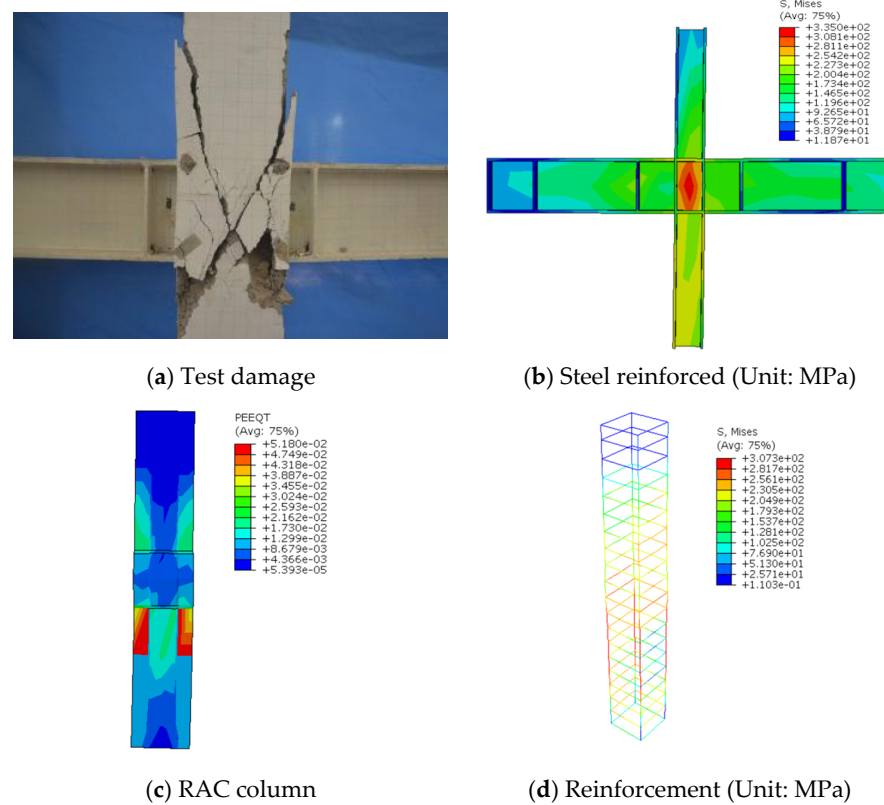


Figure 15. Comparison of test and simulation results of CFJ5.

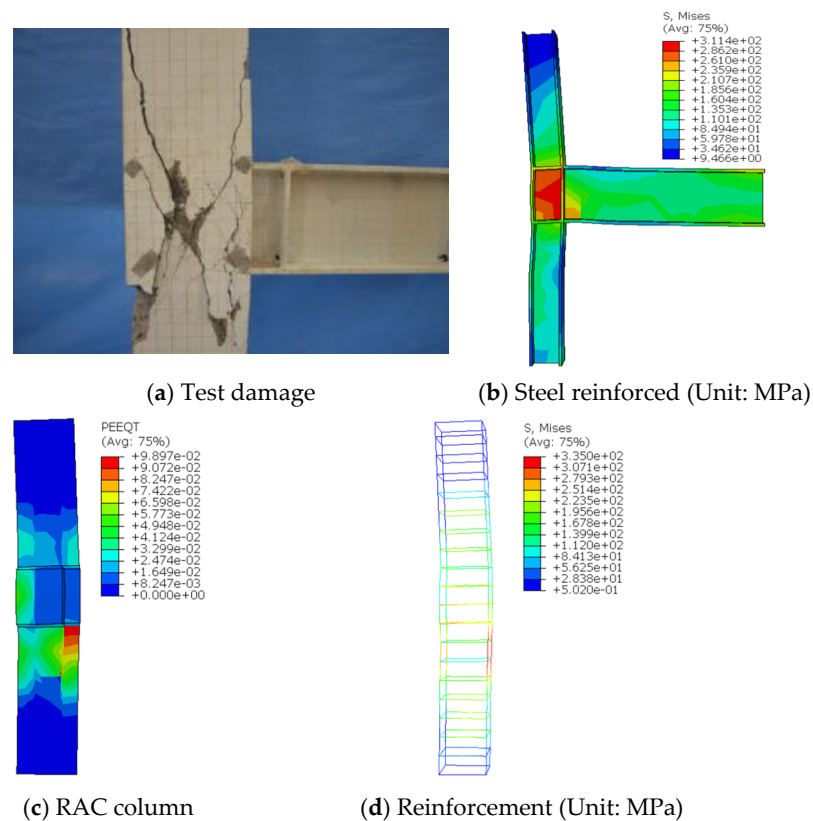


Figure 16. Comparison of test and simulation results of CFJ7.

In ABAQUS, the damage of concrete is mainly checked by two indicators: compression equivalent plastic strain (PEEQ) and tension equivalent plastic strain (PEEQT). Therefore,

PEEQT was used to reflect the failure of RAC. Figures 15 and 16 inform that when the joint was under ultimate load, the RAC in the core area of the joint formed a banded compression zone, which roughly formed oblique cracks along the diagonal direction. The compression mechanism of the oblique compression rod of the composite joint was consistent with the “X” shape diagonal cracks produced in the core area of the joint in the test. Figures 15 and 16 demonstrate the Mises stress diagrams of the steel reinforced and reinforcement cage of the composite joint model under ultimate load. It can be concluded that the yield of the steel web in the core area of the joint was gradually expanded from the middle local area to the outside. When the steel web was yielded in the core area of the joint, there was no plastic hinge at the end of the steel beam and column. It showed that the damage area mainly occurred in the core area of the joint, which was consistent with the experimental failure process. In summary, the FE model established can accurately reflect the stress of the composite joint, which can be applied to subsequent parameter analyses.

5. Parametric Analysis of FE on the Composite Joints

Due to the limitations of test conditions, financial and human resources, it is not practical and economical to carry out experimental research on each parameter affecting seismic performance. Moreover, the influence of other parameters on the seismic performance of the composite joints needs to be investigated further. Therefore, the influence of other parameters on the seismic performance of the composite joints was analyzed by ABAQUS 6.16 software in this paper.

From the load displacement skeleton curve in the above model verification, it can be concluded that the skeleton curve of the joint can reflect the full stress process of the composite joint in detail. Therefore, considering the calculation efficiency in FE parameter analysis, the pushover analysis was carried out with the interior joint as the basic specimen. The RAC strength, yield strength of section steel and web thickness of column steel web are the main analytical parameters.

5.1. RAC Strength

When including the influence of RAC strength factors, only changing this single parameter in numerical analysis leads to other parameters remaining unchanged. The RAC strength f_{cu} was 35 MPa, 45 MPa, 55 MPa and 65 MPa. The numerical simulation results are shown in Figure 17. Table 5 lists the influence of RAC strength on each characteristic point of the composite joints.

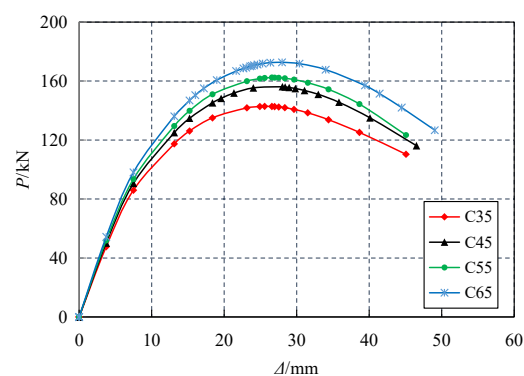


Figure 17. Influence of RAC strength on composite joints.

It can be estimated from Figure 17 and Table 5 that the strength of RAC has little effect on the initial stiffness of the composite joint. The horizontal bearing capacity of the composite joints increased with the increase the strength of RAC, while the maximum increase was 20.87% compared with the peak bearing capacity of the composite joints with strength grade C35, which indicated that the increase of the strength of RAC can significantly improve the strength of the composite joint. The displacement corresponding

to the peak load was similar, indicating that the strength of RAC has little effect on the deformation capacity of the composite joints. At the end of loading, the horizontal load also gradually increased with the increase of RAC strength, and the skeleton curve had a significant decline. The ultimate load of each specimen decreased by 22.75%, 25.60%, 24.01% and 26.64% relative to the peak load.

Table 5. Influence of RAC strength on characteristic points of composite joints.

RAC Strength	Yield Load P_y/kN	Yield Displacement Δ_y/mm	Peak Load P_m/kN	Peak Displacement Δ_m/mm	Ultimate Load P_u/kN	Ultimate Displacement Δ_u/mm
C35	117.39	13.12	142.85	25.59	110.35	45.08
C45	124.98	14.13	156.16	28.04	116.18	46.53
C55	129.55	13.98	162.36	26.59	123.38	45.08
C65	136.08	14.87	172.67	27.99	126.67	49.06

5.2. Yield Strength of Steel

Steel accounts for a large proportion of composite joints. The influence of steel strength on joints cannot be ignored. Therefore, Q235, Q345, Q390 and Q420 steels commonly applied in practical engineering were selected as the influence parameters of numerical simulation. The numerical simulation results were recorded in Figure 18, and the influence of yield strength of different types of steel on each characteristic point of composite joint is listed in Table 6.

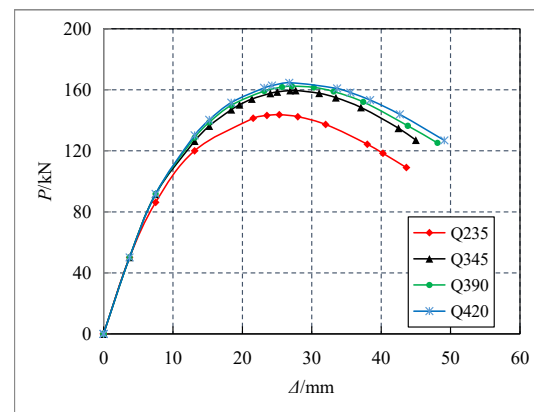


Figure 18. Influence of yield strength of profile steel on composite joints.

Table 6. Influence of yield strength of profile steel on characteristic points of composite joints.

Yield Strength of Steel	Yield Load P_y/kN	Yield Displacement Δ_y/mm	Peak Load P_m/kN	Peak Displacement Δ_m/mm	Ultimate Load P_u/kN	Ultimate Displacement Δ_u/mm
Q235	120.04	13.12	143.71	25.32	109.06	43.63
Q345	130.34	14.51	159.57	27.68	127.03	44.99
Q390	130.74	13.98	162.41	27.12	125.21	48.11
Q420	132.66	14.33	164.64	26.76	126.87	49.09

It can be seen from Figure 18 and Table 6 that the horizontal bearing capacity of the composite joints increases slightly with the increase of steel yield strength. Compared with the Q235 steel composite joint, the peak bearing capacity of the Q345, Q390 and Q420 steel composite joints was increased by 11.04%, 13.02% and 14.57%, respectively. However, there was little difference among them. The displacements of the joints with different strengths were similar, indicating that the yield strength of steel has little influence on the deformation capacity of the composite joints. The skeleton curves of the strength joints of all types of steel were significantly reduced. The ultimate load of the combined joints is 24.11%, 20.39%, 22.91%

and 22.94% lower than the peak load. It can be seen that the deterioration of the composite joints was basically gentle, demonstrating that the yield strength of steel had little effect on the ductility of the joints, and showed good ductility in general.

5.3. Thickness of Column Steel Web

In order to study the influence of different column steel web thicknesses on the performance of composite joints, this paper took $d = 4\sim 10$ mm for the parameter influence analysis, and d was taken as 4 mm, 6 mm, 8 mm and 10 mm. The numerical simulation results are shown in Figure 19. Table 7 lists the influence of column steel web thickness on each characteristic point of the composite joints.

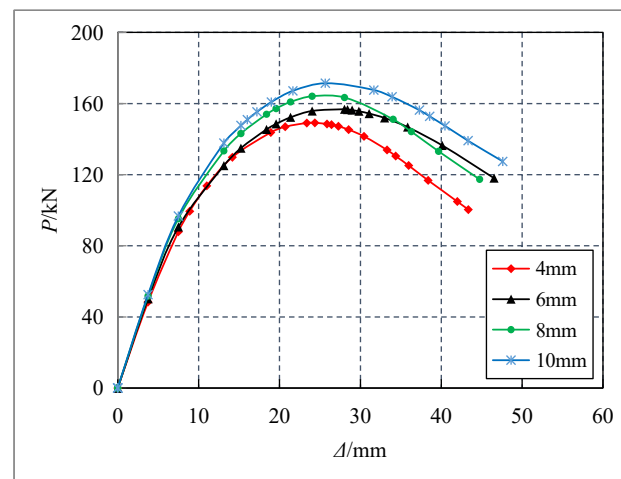


Figure 19. Influence of profile web thickness on composite joints.

Table 7. Influence of profile web thickness on characteristic points of composite joints.

Thickness d/mm	Yield Load P_y/kN	Yield Displacement Δ_y/mm	Peak Load P_m/kN	Peak Displacement Δ_m/mm	Ultimate Load P_u/kN	Ultimate Displacement Δ_u/mm
4	117.39	13.12	149.14	24.37	100.33	43.34
6	124.98	14.13	156.79	28.04	118.04	46.53
8	129.55	13.98	164.11	24.03	117.39	44.74
10	136.08	14.87	171.41	25.66	127.45	47.59

It can be estimated from Figure 19 and Table 7 that the column steel web has little effect on the stiffness of the initial elastic section of this type of composite joints, but has great influence on the peak bearing capacity and the degradation of the later bearing capacity. Compared with the peak bearing capacity of the composite joints with 4 mm web thickness, the peak bearing capacity of the composite joints with 6 mm, 8 mm and 10 mm web thicknesses increased by 5.13%, 10.04% and 14.93%, respectively, demonstrating that the thickness of column steel web had a significant effect on improving the strength of the composite joints. At the end of loading, the ultimate horizontal bearing capacity of the column increases with the increase of the steel web thickness, but the decline section of the skeleton curve was still obvious and the decline trend was similar. In summary, the increase of column steel web thickness can improve the horizontal bearing capacity of composite joints, while reasonable web thickness can also improve the seismic ductility of composite joints.

6. Restoring Force Model of Composite Joints

6.1. Establishing Method

The principal stress process and characteristics of composite joints were analyzed by low-cycle cyclic loading tests on the inner and outer joints of composite joints under different RAC replacement rates and axial compression ratios. Combined with the test

hysteretic curve and skeleton curve, the restoring force model of interior joints and exterior joints in the composite joints was proposed by using the fitting method of test data, which provided reference for the elastic–plastic seismic response analysis of the composite joints and frame.

The following assumptions are mainly the simplification and summary of the principal stress process and characteristics of composite joints, the experimental hysteretic curves and skeleton curves. They should be made first before established the restoring force model of composite joints with SRRC columns and steel beams.

- (1) Assume that the elastic limit point of the specimen coincides with the cracking point of the composite joints;
- (2) Before the cracking point, the unloading stiffness and loading stiffness of the composite joints were the same, and after the elastic section, the stiffness decreased with the increase of displacement;
- (3) The skeleton curve adopted elastic stage, crack working stage, yield strengthening stage and failure stage four-fold line model.

6.2. Establishment of Skeleton Curve Model

The skeleton curve obtained by the actual test was a continuous smooth curve without an obvious yield point. When established the restoring force model of skeleton curve, it should be simplified to a quadrangle model with stiffness degradation. The details are stated as follows:

The characteristic points of skeleton curve were cracking point $A (P_c, \Delta_c)$; yield load point $B (P_y, \Delta_y)$; peak load point $C (P_{max}, \Delta_{max})$; and limit load point $D (P_u, \Delta_u)$, where yield load point B was calculated by the “general yield moment method”, and limit load point D was 85% of the peak load. Due to the differences of skeleton curves of various composite joints, dimensionless processing was carried out for skeleton curves of various composite joints first, as shown in Figure 20.

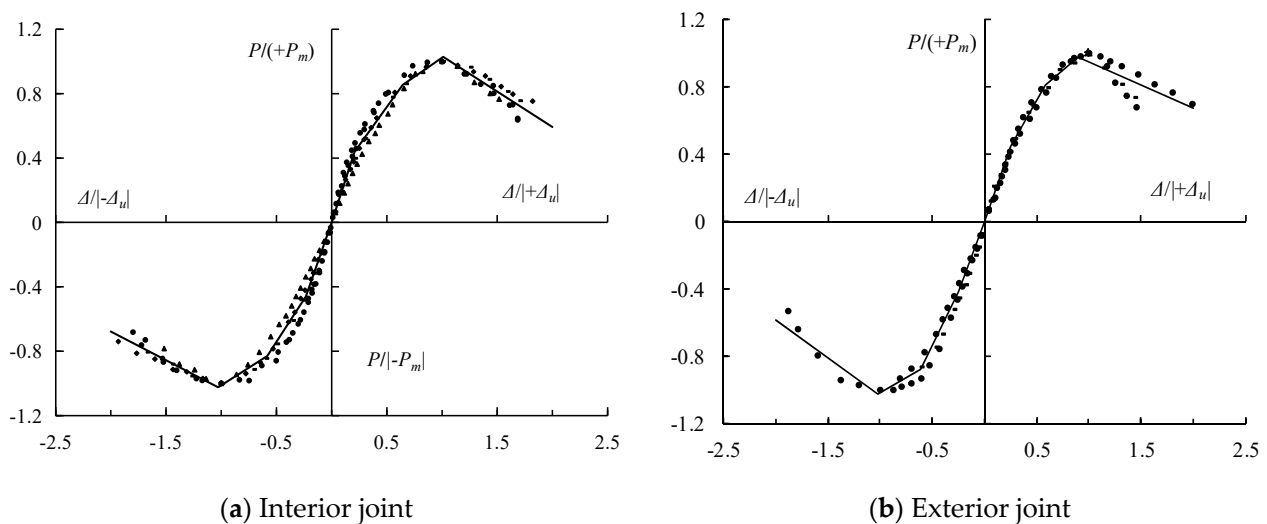


Figure 20. Dimensionless skeleton curve.

It can be estimated from Figure 20 that the data points of the skeleton curve of interior joints and exterior joints after dimensionless processing have good regularity, as shown in the real line in the graph. The linear regression method was applied to fit the data points of each stage of the skeleton curve. The four-line skeleton curve model was shown in Figure 21. OA was the elastic section from the positive loading to the cracking point, while AB was the working section with cracks under positive loading, and BC was the yield strengthening stage under positive loading, while CD was the failure stage under positive loading. OA' was the elastic section from reverse loading to cracking point, AB was the

working section with cracks under reverse loading, $B'C'$ was the yield strengthening stage under reverse loading and $C'D'$ was the failure stage under reverse loading. The fitting process of skeleton curves of interior joints and exterior joints in each stage is exhibited in Figure 22. The linear equations and stiffness in each stage of the skeleton curve model are shown in Tables 8 and 9.

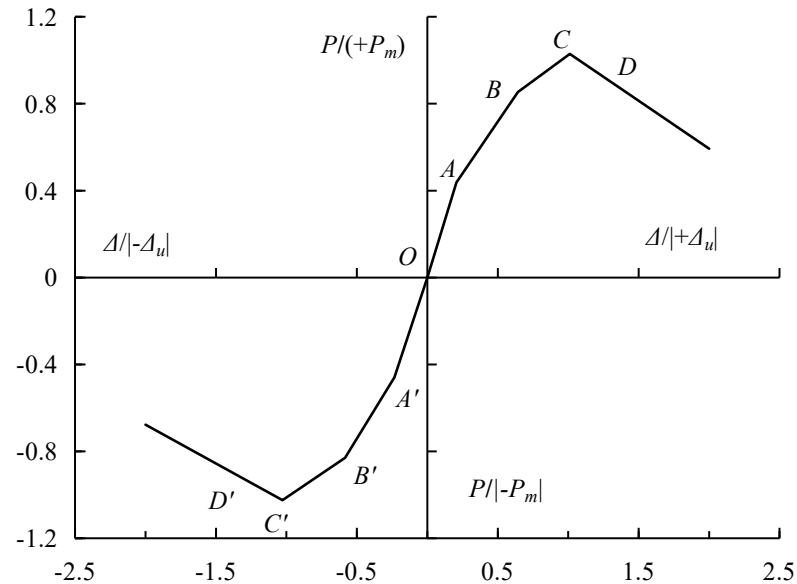


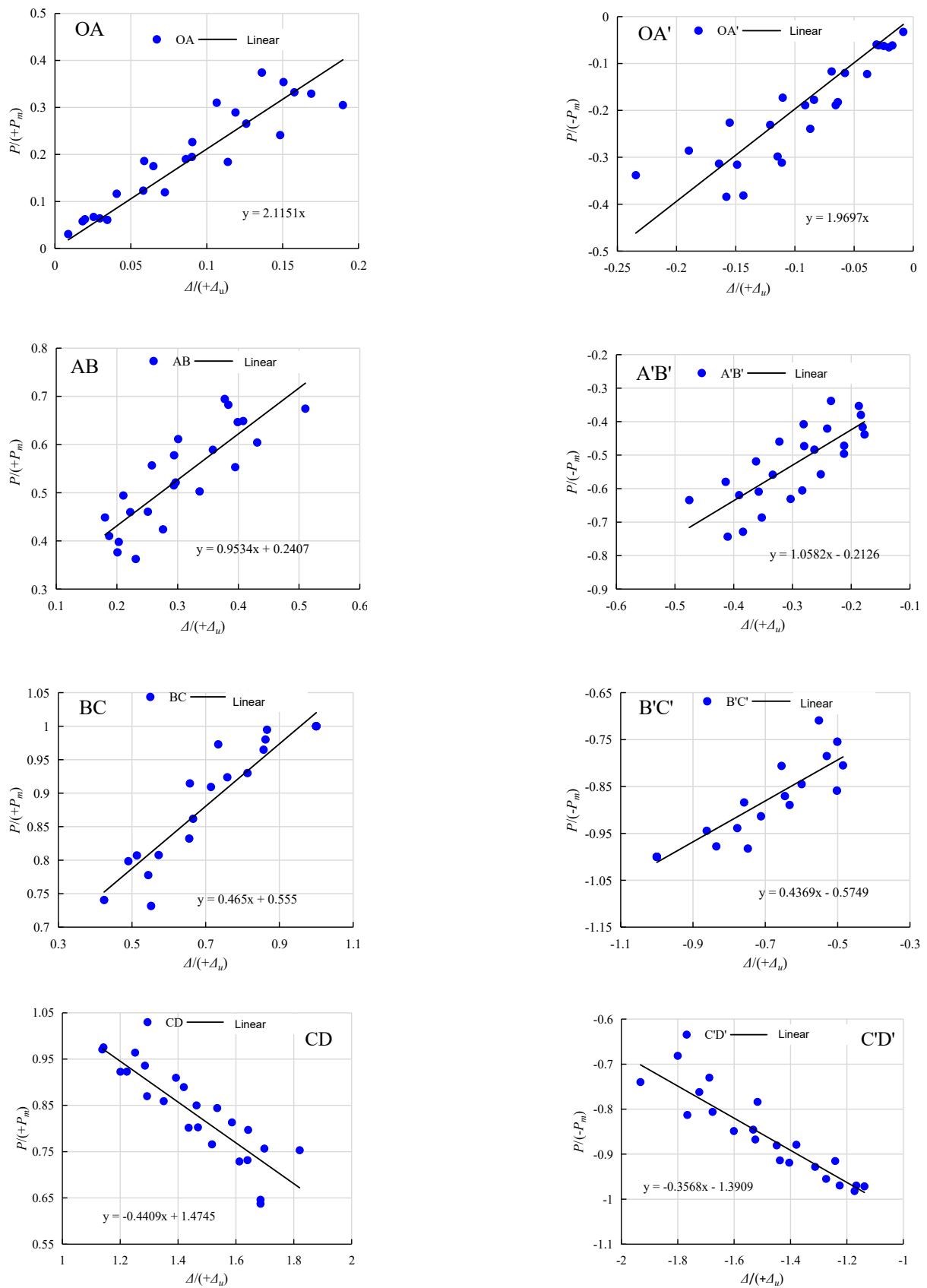
Figure 21. Skeleton curve model with four-line.

Table 8. Linear equation and stiffness of each stage in skeleton curve model of interior joint.

Line Segment	Regression Equation	Angle to Displacement Axis	Rigidity
OA	$P/(+P_m) = 2.1151\Delta/(+\Delta_m)$	65°	2.1151
AB	$P/(+P_m) = 0.9534\Delta/(+\Delta_m) + 0.2407$	44°	0.9534
BC	$P/(+P_m) = 0.465\Delta/(+\Delta_m) + 0.555$	25°	0.465
CD	$P/(+P_m) = -0.4409\Delta/(+\Delta_m) + 1.4745$	-24°	-0.4409
OA'	$P/ -P_m = 1.9697\Delta/ -\Delta_m $	63°	1.9697
A'B'	$P/ -P_m = 1.0582\Delta/ -\Delta_m - 0.2126$	47°	1.0582
B'C'	$P/ -P_m = 0.4369\Delta/ -\Delta_m - 0.5749$	24°	0.4369
C'D'	$P/ -P_m = -0.3568\Delta/ -\Delta_m - 1.3909$	-20°	-0.3568

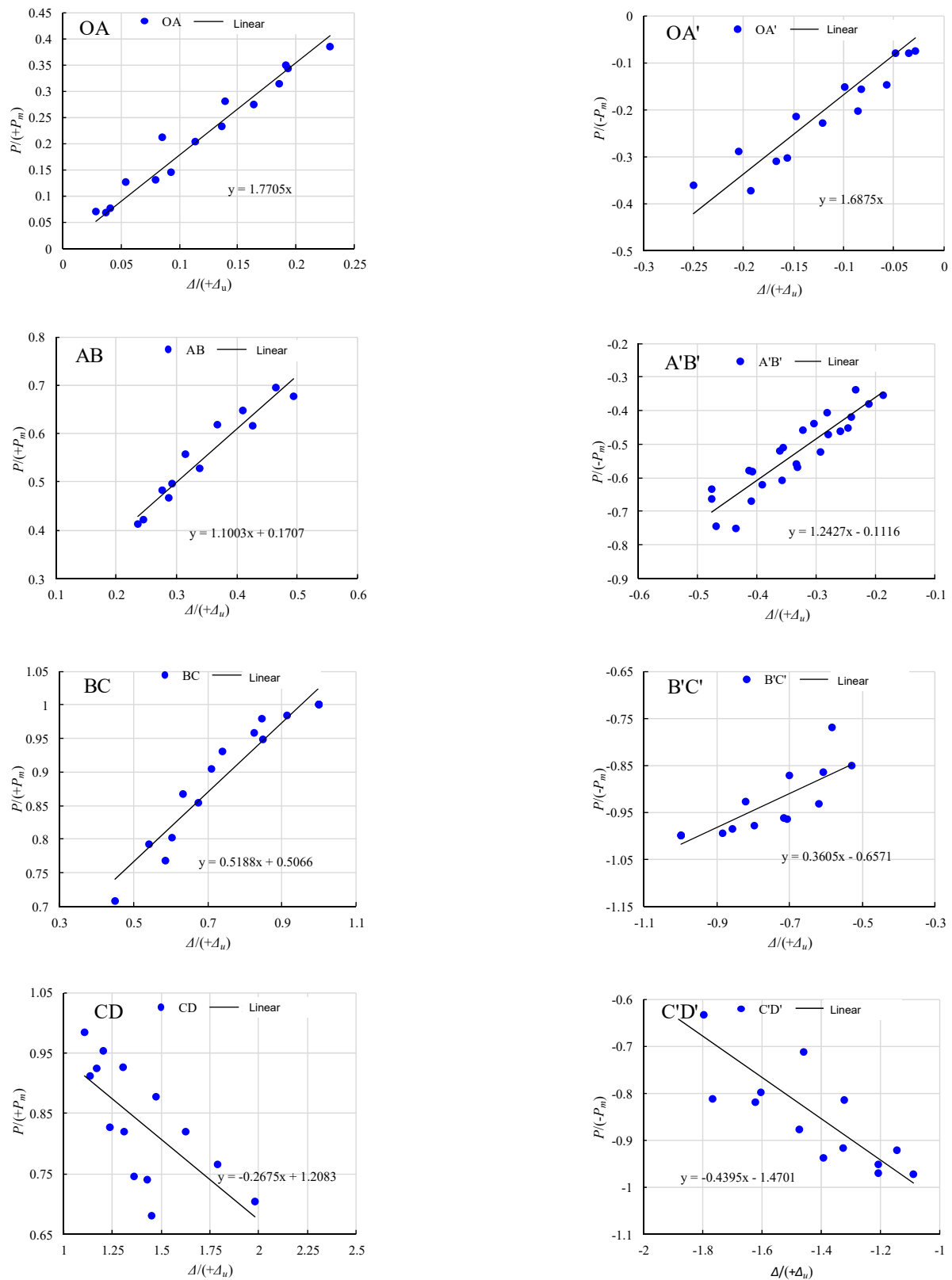
Table 9. Linear equation and stiffness of each stage in skeleton curve model of exterior joint.

Line Segment	Regression Equation	Angle to Displacement Axis	Rigidity
OA	$P/P_m = 1.7705\Delta/\Delta_m$	60	1.7705
AB	$P/P_m = 1.1003\Delta/\Delta_m + 0.1707$	48	1.1003
BC	$P/P_m = 0.5188\Delta/\Delta_m + 0.5066$	27	0.5188
CD	$P/(+P_m) = -0.4409\Delta/(+\Delta_m) + 1.4745$	-15	-0.2675
OA'	$P/ -P_m = 1.9697\Delta/ -\Delta_m $	59	1.6875
A'B'	$P/ -P_m = 1.0582\Delta/ -\Delta_m - 0.2126$	51	1.2427
B'C'	$P/ -P_m = 0.4369\Delta/ -\Delta_m - 0.5749$	20	0.3605
C'D'	$P/ -P_m = -0.3568\Delta/ -\Delta_m - 1.3909$	-24	-0.4395



(a) Each stage fitting process of interior joint skeleton curve

Figure 22. Cont.



(b) Each stage fitting process of exterior joint skeleton curve

Figure 22. Fitting process of skeleton curve.

6.3. Stiffness Degradation Rule

Through the low-cyclic loading test of composite joints, it was found that the stiffness of the composite joints decreases gradually with the increase of load and displacement loading stages. The skeleton curve restoring force model simplifies the whole loading process of the composite joints into four stages, and nonlinear fitting was carried out for the loading stiffness and unloading of the composite joints at each stage. The variation of the forward and reverse unloading stiffness of the composite joints was described in detail below. K_{12} , K_{34} and K_{56} represent the unloading stiffness under forward loading, while $K_{1'2'}$, $K_{3'4'}$ and $K_{5'6'}$ represent the unloading stiffness under reverse loading, as presented in Figure 23.

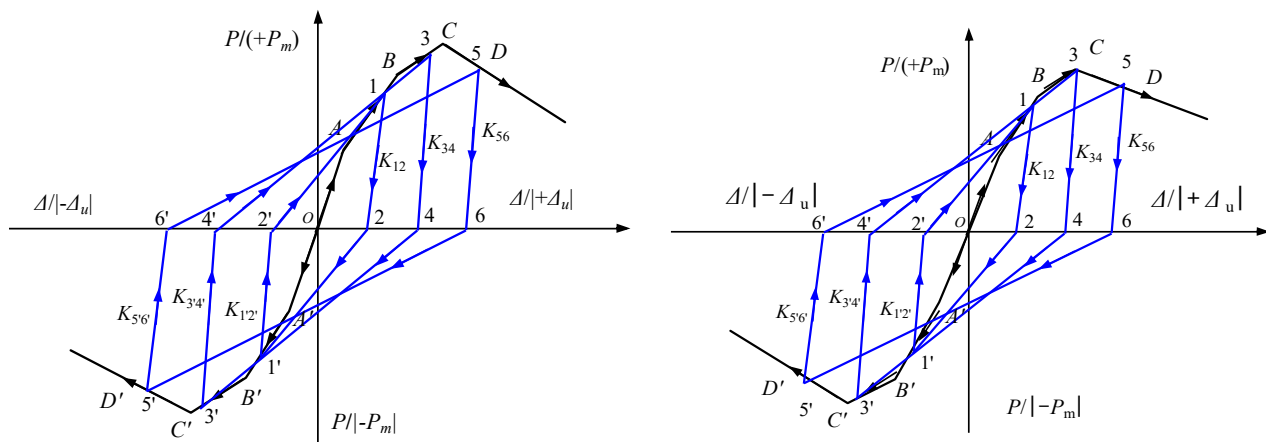
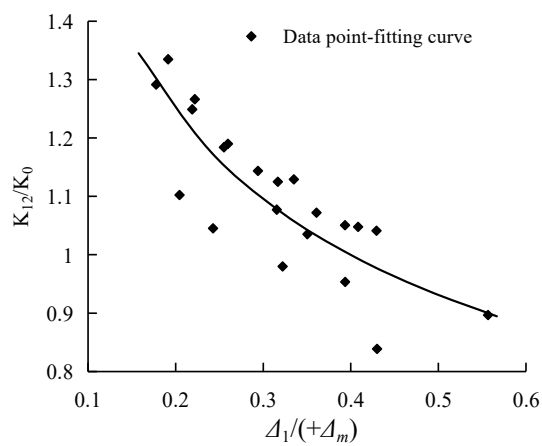


Figure 23. Stiffness degradation rule of specimen.

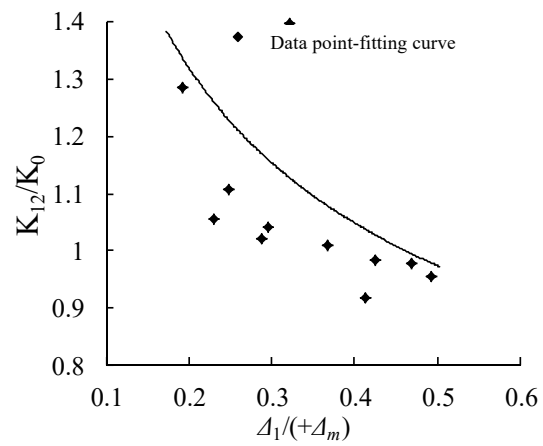
The unloading stiffness K_{12} , AB section was the working stage of positive loading with cracks. When unloading before positive loading to A, the unloading route was along OA, and the unloading stiffness was the initial stiffness K_0 of the composite joints. After the unloading of section AB, the unloading route was carried out along 12, and the unloading stiffness was K_{12} . The slope K_{12} of forward unloading route 12 was obtained by linear fitting and all measured data between unloading point 1 and point 2 when unloading load was 0. Then, the unloading stiffness of the AB section of each composite joint was normalized, while the nonlinear regression fitting was carried out by power function. The nonlinear relationship curve between K_{12}/K_0 and $\Delta_1/(+\Delta_m)$ was obtained, where K_0 was the initial stiffness of the positive loading elastic section, or the slope of the straight line obtained by linear fitting of the measured data of the positive loading elastic section of the skeleton curve. Δ_1 was the displacement corresponding to positive loading point 1. The above method was adopted to nondimensionalize the unloading stiffness of each test. Moreover, the power function was used for nonlinear regression fitting, as shown in Figure 24.

Based on the experimental research, combined with the obtained hysteretic curve and skeleton curve, the restoring force model of composite joints proposed was shown in Figure 25. The model considered the four stress stages of the composite joints during the whole loading process in the form of four broken lines, and fitted the stiffness degradation law of each stage, which can truly reflect the stiffness degradation law of the composite joints in each stress stage and was suitable for the elastic-plastic analysis of composite joints and other structures.



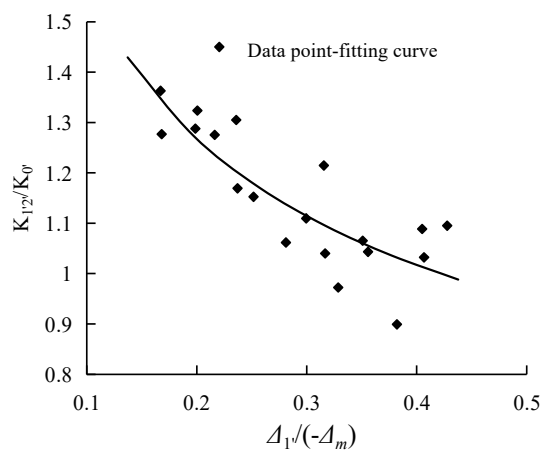
$$K_{12}/K_0 = 0.7465[\Delta_1/(+\Delta_m)]^{-0.319}$$

(a) Interior joint



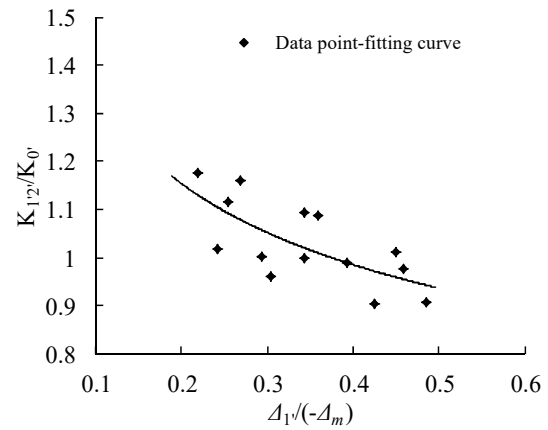
$$K_{12}/K_0 = 0.7743[\Delta_1/(+\Delta_m)]^{-0.331}$$

(b) Exterior joint



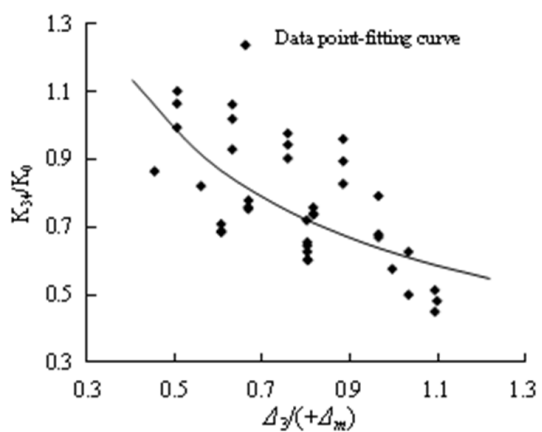
$$K_{1'2'}/K_{0'} = 0.76[\Delta_1'/(-\Delta_m)]^{-0.318}$$

(c) Interior joint



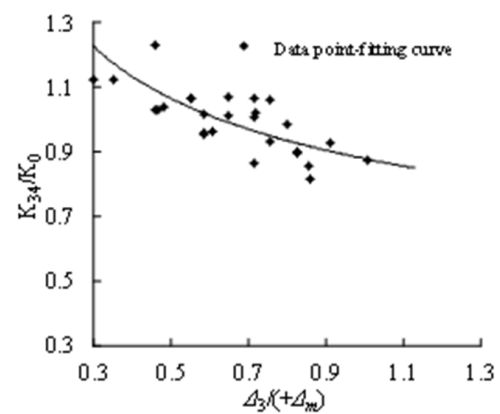
$$K_{1'2'}/K_{0'} = 0.7996[\Delta_1'/(-\Delta_m)]^{-0.228}$$

(d) Exterior joint



$$K_{34}/K_0 = 0.6228[\Delta_3/(+\Delta_m)]^{-0.659}$$

(e) Interior joint



$$K_{34}/K_0 = 0.8796[\Delta_3/(+\Delta_m)]^{-0.276}$$

(f) Exterior joint

Figure 24. Cont.

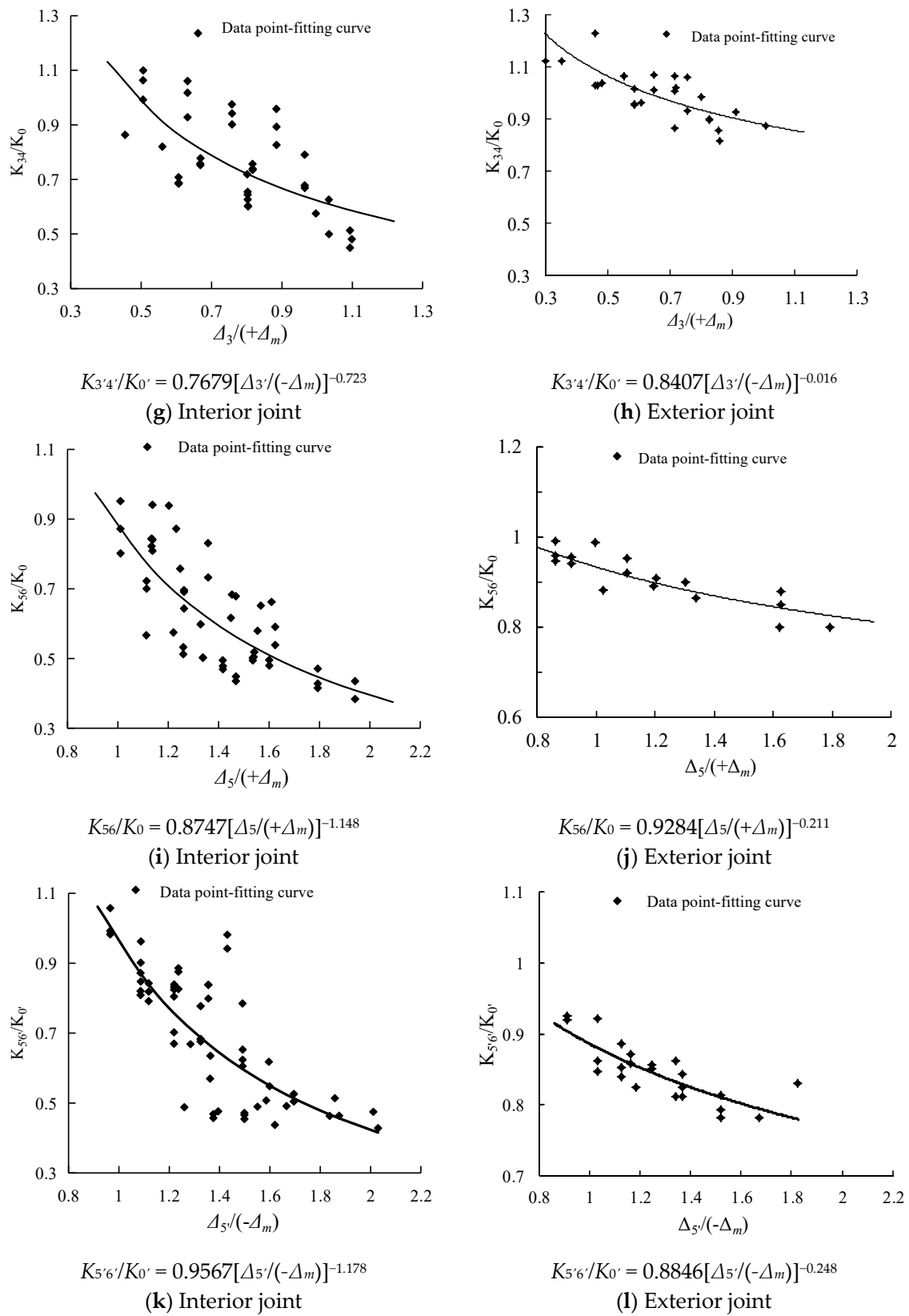


Figure 24. Stiffness degradation rule curve of unloading stiffness.

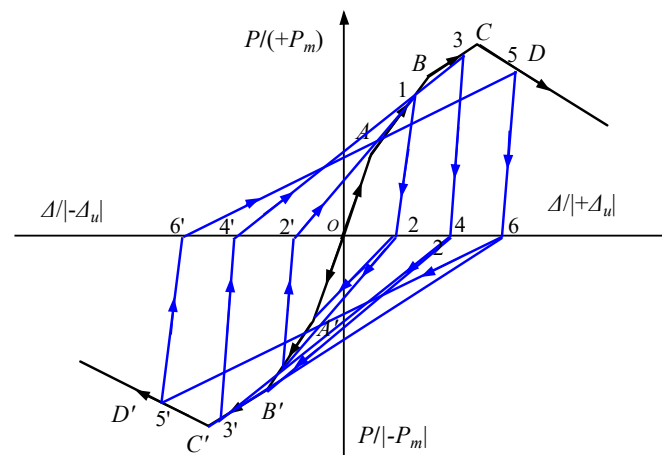


Figure 25. Four-line restoring force model of stiffness degradation.

The restoring force model was described as follows:

- (1) The restoring force model was obtained by combining the main feature points (cracking point, yield point, peak point and limit point) of the test skeleton curve and by dimensionless processing and fitting of the test data. The coordinates were dimensionless coordinates.
- (2) The skeleton curve of the restoring force model was divided into four stages. In the figure, OA and OA' were elastic sections under positive and negative loading, respectively, AB and A'B' were working sections with cracks under positive and negative loading, respectively and BC and B'C' were yield strengthening sections under positive and negative loading, respectively, while CD and C'D' were failure sections under positive and negative loading, respectively.
- (3) When the specimen is loaded and unloaded in the elastic segment (OA and OA'), the unloading stiffness was the elastic stiffness of the specimen. When the specimen was loaded to the elastic segment (AD and A'D'), the unloading stiffness gradually degrades.

Therefore, the restoring force model established in this paper mainly has the following characteristics. In the quasi-static test, the loading and unloading stiffness of the composite joints before reaching the cracking load were both initial stiffness. After reaching the cracking load, the stiffness degradation phenomenon of the composite joints is considered. After cracking, the stiffness of forward and reverse loading gradually degrades with the increase of displacement, and the degradation rate accelerates with the loading process.

6.4. Validation of Restoring Force Model

According to the regression equations of the skeleton curve model at each stage shown in Table 8, the established skeleton curve model was compared with the experimental skeleton curve, as exhibited in Figure 26.

The calculated hysteretic curve of each composite joint can be acquired by the four-fold restoring force model proposed in Figure 20. The calculated hysteretic curve was compared with the experimental hysteretic curve, such as Figure 27.

From Figures 26 and 27, it can be seen that the skeleton curve and hysteretic curve calculated by both interior joints and exterior joints were in good agreement with the overall trend of the test skeleton curve and hysteretic curve. The results demonstrated that the four-fold restoring force model proposed can well reflect the hysteretic characteristics of composite joints with SRRC columns and steel beams under low cyclic loading.

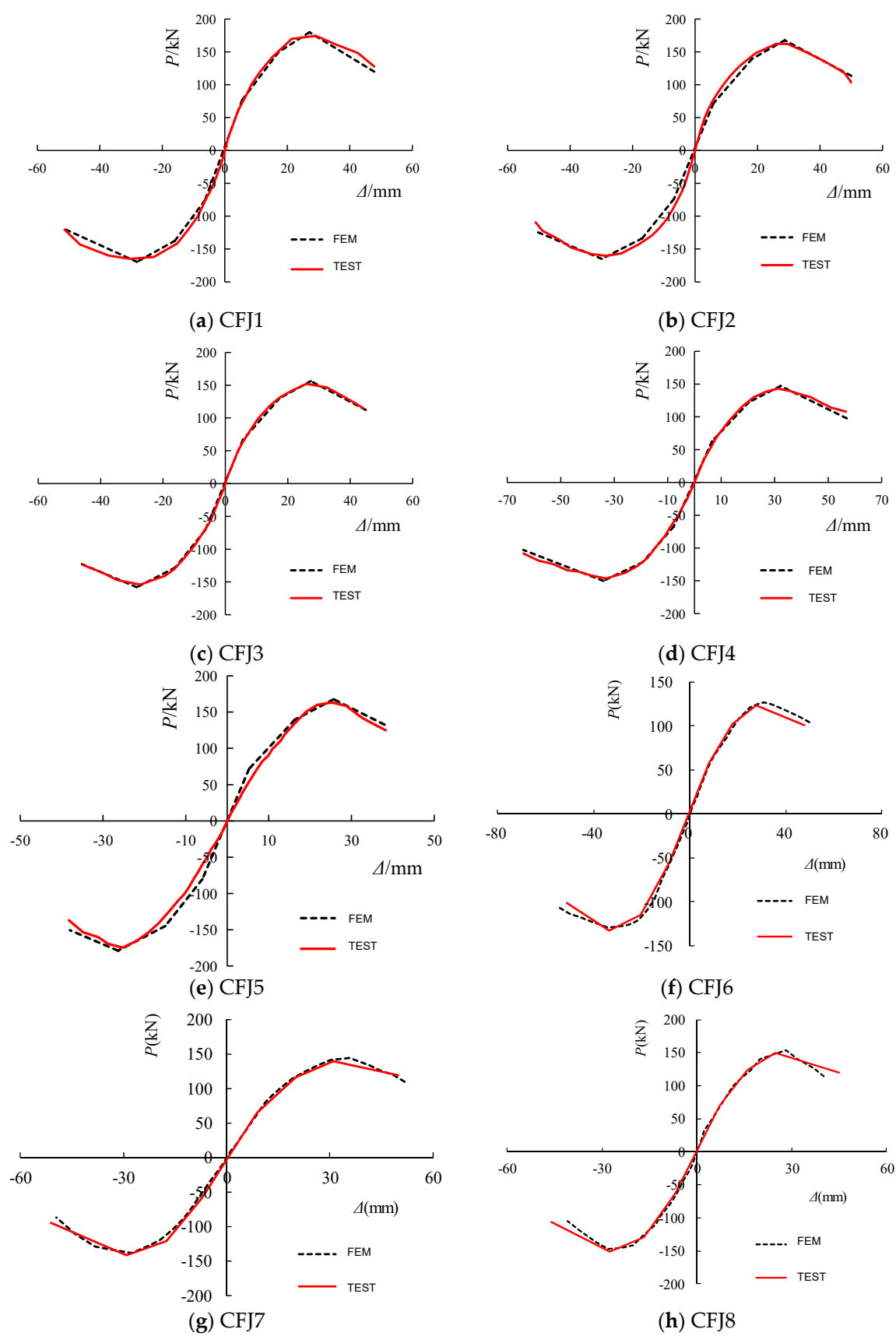


Figure 26. Comparison of skeleton curve between calculation results and test results.

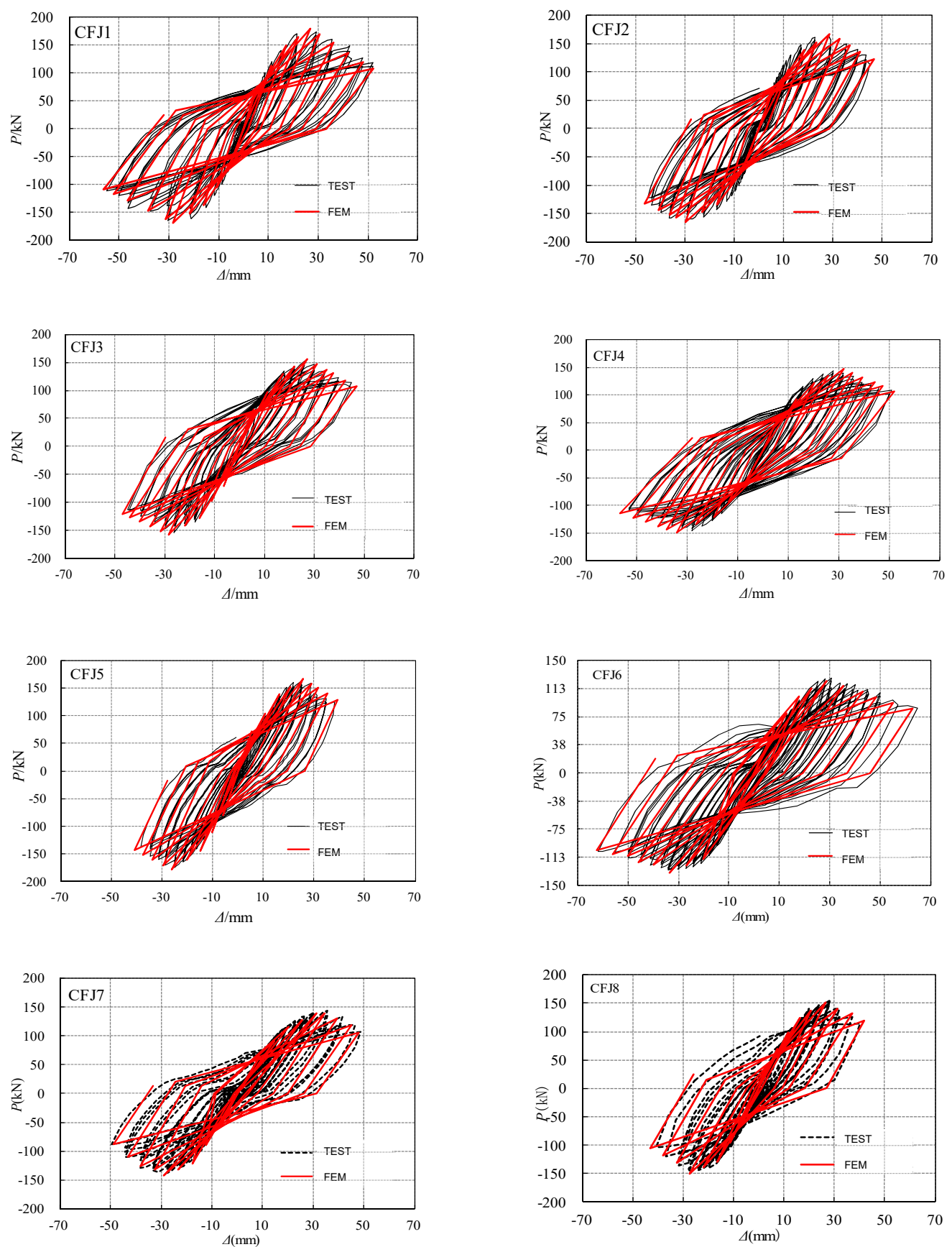


Figure 27. Comparison of hysteretic curve between calculation results and test results.

7. Results of the Study

7.1. Conclusions

Based on the experimental study, the failure mechanism of composite joints with SRRC columns and steel beams is studied, the mechanical model is established and the calculation method of bearing capacity is verified. The refined numerical model of composite joints with SRRC columns and steel beams was established, and the design parameters were extended and analyzed. The main conclusions are as follows:

- (1) The finite element calculation results are in good agreement with the experimental results, which shows that the simulation method in this paper has good rationality and applicability. The modeling method can provide a reference for the numerical analysis of these kind of joints.
- (2) With the increase of RAC strength, the horizontal bearing capacity of the composite joints increases significantly, but the stiffness of the elastic section increases slightly, and the RAC strength has little effect on the deformation capacity of composite joints.
- (3) The increase of steel yield strength has no obvious effect on the horizontal bearing capacity and ductility of the composite joints.
- (4) The increase of the thickness of column steel web can improve the horizontal bearing capacity of composite joints, and reasonable thickness can improve the seismic ductility of the composite joints.
- (5) By fitting the experimental data of eight composite joints, a hysteretic curve restoring force model of composite joints under repeated load was proposed. The hysteretic characteristics of the composite joints under low cyclic loading can be well reflected by restoring the force model curve.

7.2. Recommendations

On the one hand, the model has good seismic performance. On the other hand, it can solve the problem of waste concrete disposal. It meets the requirements of national green environmental protection and sustainable development and has broad development prospects. In this paper, the composite frame joints of reclaimed steel column and steel beam are studied by combining experiment and theory. Although some achievements have been made, there are still many problems to be further studied.

- (1) The influence of the bond slip between the steel and concrete, the friction coefficient value and how to extend the restoring force model proposed for other typologies of composite joints should be investigated systematically and deeply in future work.
- (2) The proposed nodal restoring force model in the combined framework is mainly proposed based on the test results. Although the model curve is in good agreement with the test curve, the number of specimens is small and has not been verified by numerical simulation, which needs to be further studied and modified.
- (3) The composite joints of the reclaimed steel reinforced concrete column and steel beam studied in this paper are planar joints with a strong axis direction, and the subsequent mechanical properties of the composite frame joints in the form of spatial joints need to be studied.
- (4) At present, there is a certain basis for the research on the mechanical properties of recycled concrete and related components, but the research content on the durability of recycled concrete is scant, and should be further studied in the future.

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Abbreviations

SRRC	Steel Reinforced Recycled Concrete
RAC	Recycled Aggregate Concrete
FE	Finite Element
RCA	Recycled Coarse Aggregate

List of Symbols

r	RCA replacement percentage
n	Axial compression ratio
ρ_a	Profile steel ratio
σ_i	Equivalent stress of steel
f_s	Yield strength of steel
f_y	Yield strength of steel
E_s	Elastic modulus of steel
E_t	Tangent modulus of steel
ζ	Profile steel index
ε_i	Equivalent strain of steel
ε_y	Strain of steel at yield
ε_u	Strain at the limit of steel
σ_s	Stress of steel
ε_0	Peak strain
ε_{cc}	Peak strain
f_{cp}	Peak stress of concrete in the weak constraint area
ε_{cp}	Peak strain of concrete in the weak constraint area
f_{ch}	Peak stress of strong constraint concrete
ε_{ch}	Peak strain of strong constraint concrete
$\sigma_1, \sigma_2, \sigma_3$	Three principal stresses
d_c	Damage factor

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Article

Cyclic Performance of Prefabricated Shear Wall Connected to Columns by Rectangular Concrete-Filled Steel Tube Keys

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Abstract: A prefabricated frame–shear wall structure is a major structure in an assembled building. To find a method of connecting a shear wall and columns that can both reduce the amount of wet work required and maintain adequate stiffness, we propose connecting the shear wall to the vertical frame using rectangular concrete-filled steel tube keys (RCFSTKs). Static tests of a cast-in-place frame–shear wall structure without keys and a prefabricated structure with RCFSTKs were conducted to compare their seismic performance. The feasibility of the new method was analyzed. Finite element models were then set up to determine if plain concrete blocks between RCFSTKs could be removed and to identify the influence of different parameters on the cyclic performance of the assembled structure. The results show that the use of RCFSTK is practical. Compared with a dimensionally similar cast-in-place shear wall–column construction, a prefabricated shear wall connected to columns by RCFSTKs has a fuller hysteresis curve, better ductility, slightly higher energy dissipation, and slightly slower degradation of stiffness and bearing capacity. The removal of inter-key concrete blocks significantly reduces bearing capacity and initial stiffness. The cyclic performance of the assembled structure is primarily influenced by the number of RCFSTKs, the thickness of the steel tube key wall, and the axial compression ratio, with less effect from key unit height, steel strength, and concrete grade in RCFSTKs.

Keywords: RCFSTKs; frame column; shear wall; fabricated concrete structure; cyclic performance

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1. Introduction

Assembled concrete structures have the advantages of fast construction, high production efficiency and good engineering quality. They have been widely used in various buildings. A frame–shear wall structure is composed of frames and shear walls. Due to the combination effect [1], it has the advantages of flexible building layout, large spacing and good lateral resistance. An assembled frame–shear wall structure combines the advantages of the assembled concrete structure and the frame–shear wall structure, and it has been proved to have good cyclic performance [2,3]. This promotes their application in areas requiring seismic fortification.

Prefabricated shear walls, prefabricated frame beams and columns are the important components of a prefabricated frame–shear wall structure. How to connect them is the key technical problem of this structure. Two widely used assembly methods for such a structure exist in China. One is to prefabricate a shear wall, frame columns, and frame beams as a complete unit for subsequent assembly; this method is suitable when the wall, columns, and beams are small. Researchers [1–5] conducted a series of studies on this kind of structure and found that its cyclic performance was similar to that of a cast-in-place one. The second method is to prefabricate a wall, columns, and beams as independent units for subsequent assembly; this method is suitable when these components are larger. The feasibility of this

method has been demonstrated by Zhao et al. [6]. There is less research into assembled frame–shear wall structures from other countries where the assembly method mainly used is to attach shear walls to the outside of the frame plane [7,8].

We study the second case. For the second method, it is necessary to study how to ensure a rigid connection between the shear wall and columns. Tong et al. [9] investigated a single-story structure of walls and columns connected with straight bars, U-shaped bars, straight U-shaped bars, and link bars using compression–shear tests and static tests. Gao et al. [10–12] examined the feasibility of connecting the shear wall and columns with U-bar in the graduate student apartment building of Shenyang Jianzhu University. Anion et al. [13] proposed setting keys on the surface of the wall and columns. The preceding studies all require pouring concrete or grout in or around the connection interface, which necessitates a relatively large amount of wet work. To reduce the amount of wet work and increase industrialization, Chinese researchers proposed using pure steel members to connect walls and columns. Gao et al. [14] investigated the mechanical properties of this type of structure using numerical models and found that its stiffness was very low and did not meet the level required to resist deformation. So, it becomes a topic of concern to find a method of connecting a shear wall and columns that can both reduce the amount of wet work required and maintain adequate stiffness.

Because the method using pure steel members to connect walls and columns can not meet the requirement of stiffness, we propose a method that uses rectangular concrete-filled steel tube keys (RCFSTKs). This paper investigates the cyclic performance of a prefabricated shear wall and column unit connected by RCFSTKs to determine the feasibility of using RCFSTKs in connecting prefabricated components in order to develop a wall and column connection method that facilitates more rapid assembly.

2. Structure of Wall and Columns Connected by RCFSTKs

Frame–shear wall structure showing the wall and columns connected by RCFSTKs is as Figure 1. The part embedded in the wall–column interface consists of a steel shear plate, a bottom steel plate, and an anchor steel plate welded together (Figure 1a). The RCFSTK is formed by filling the steel tube with concrete (Figure 1b). The plain concrete block inserted between the keys is shown in Figure 1c. If there are no RCFSTKs or plain concrete blocks, there will be gaps between the surface of the column and shear wall, which will reduce the bearing capacity and initial stiffness. The embedded parts are fixed at the sides of the wall and the columns. During assembly, the RCFSTKs are first welded between the steel shear plates, and then the plain concrete blocks are placed between the RCFSTKs, as shown in Figure 1; the joint gap is then filled with grout. This method can provide enough stiffness because the gap of column and shear wall are all filled with steel, concrete or grout. In addition, it can also reduce the wet work because the plain concrete blocks are prefabricated and only a small amount of wet gout is used. The internal force between the shear wall and the column includes shear, axial tension and compression force along the RCFSTKs and moment. The tension force is transferred by the steel of RCFSTKs, the moment is transferred by the RCFSTKs, and the other forces are transferred by the RCFSTKs, plain concrete blocks and grout.

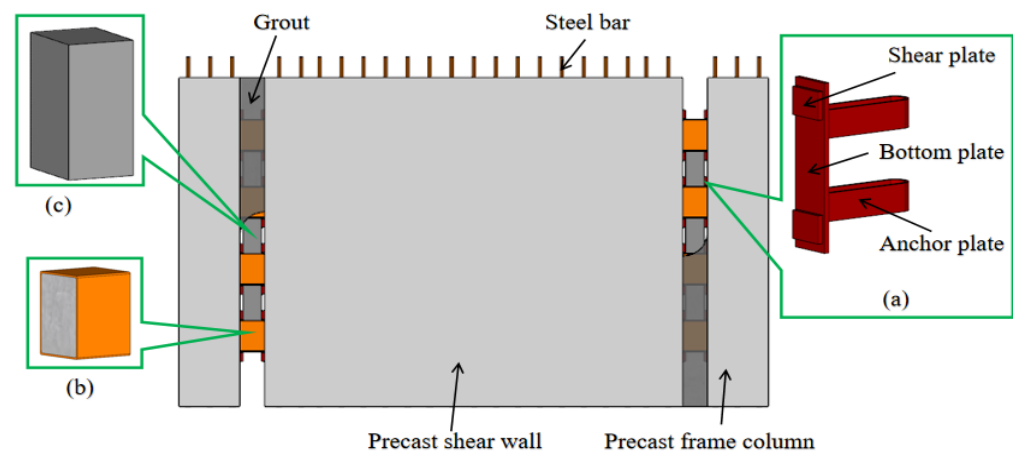


Figure 1. Frame–shear wall structure showing the wall and columns connected by RCFSTKs. (a) Embedded part; (b) RCFSTKs; (c) Plain concrete block.

3. Test Program

3.1. Test Specimen Design

A testing program was designed for specimens of two different single-story shear wall structures, XJ and ZP. Wall XJ was a single-story cast-in-place shear wall, while wall ZP was a prefabricated shear wall attached to the supporting columns with RCFSTKs. The dimensions are shown in Table 1.

Table 1. Design parameters of specimens.

Wall ID	Specimen Type	Wall and Column Connection Method	Wall Section $b \times h$ (mm)	Column, Loading Beam, and Ground Beam Sections $b \times h$ (mm)
XJ	cast-in-place	cast-in-place	1080 $\times$ 200	400 $\times$ 400
ZP	prefabricated	RCFSTKs	800 $\times$ 200	400 $\times$ 400

The dimensions and reinforcement of the two specimens are shown in Figures 2 and 3. The concrete grade was C40; the steel plates were 4 mm thick Q235; the stirrups were HPB300; the tension bars were HRB400.

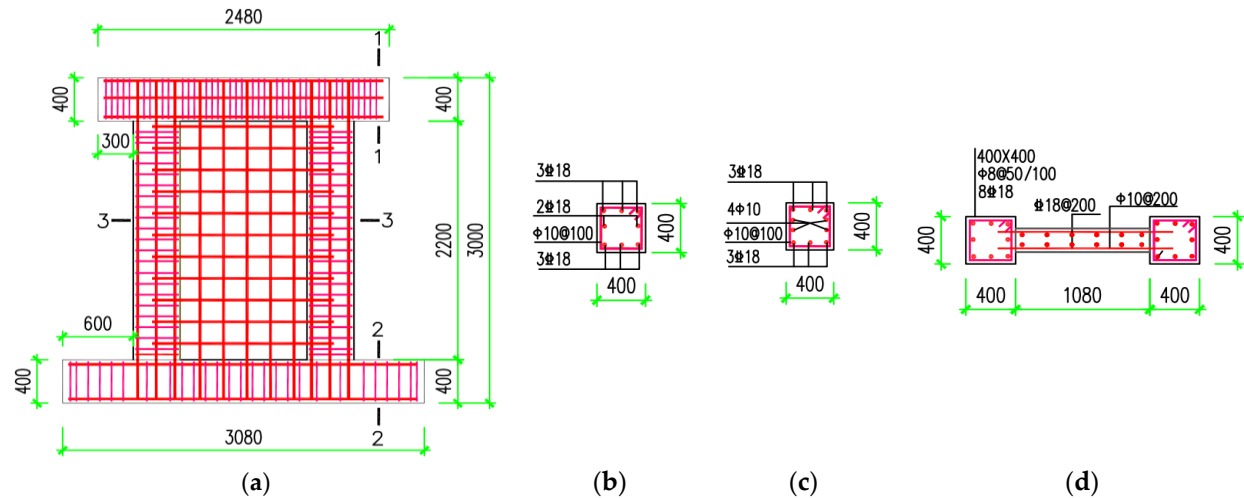


Figure 2. Geometric dimensions and reinforcement of specimen XJ. (a) Geometric dimensions and reinforcement of specimen XJ; (b) 1–1; (c) 2–2; (d) 3–3.

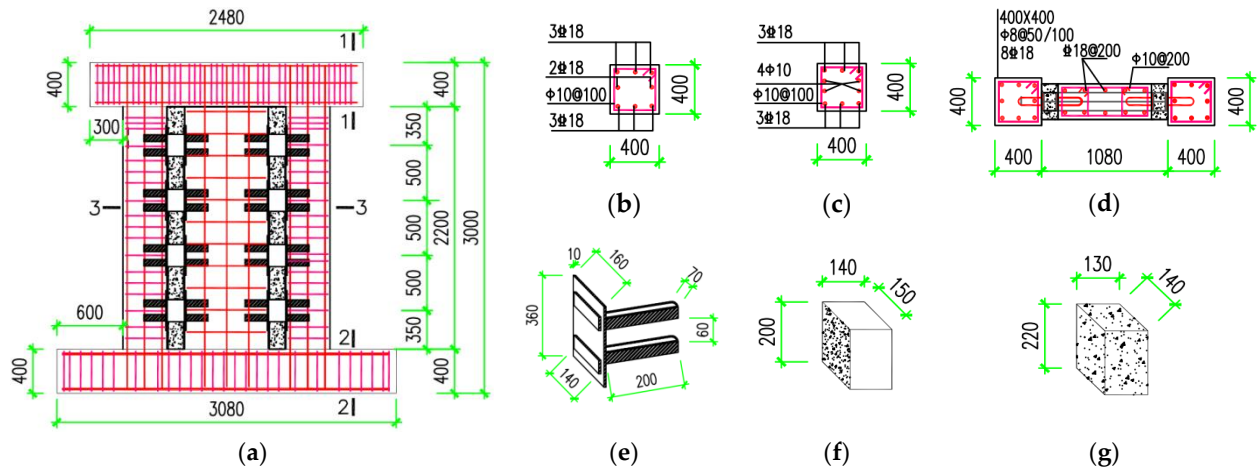


Figure 3. Geometric dimensions and reinforcement of specimen ZP. (a) Geometric dimensions and reinforcement of specimen ZP; (b) 1–1; (c) 2–2; (d) 3–3; (e) Embedded part; (f) RCFSTK; (g) Plain concrete block.

3.2. Mechanical Properties

The testing results of the mechanical properties are shown in Table 2.

Table 2. Mechanical properties of concrete, steel bars, and steel plate.

Materials	Modulus of Elasticity E (10^5 N/mm ²)	Yield Strain ϵ_y ($\mu\epsilon$)	Yield Strength f_y (MPa)	Tensile Strength f_u (MPa)	Cube Compressive Strength of Concrete f_{ck} (MPa)
Concrete	0.34	-	-	-	42
Steel plate	2.1	1304	274	352	-
Steel bar (A8)	1.4	3100	434	584	-
Steel bar (C10)	2.2	2400	528	604	-
Steel bar (C18)	1.7	2335	397	563	-

3.3. Loading Program

The test loading device is shown in Figure 4. The specimen was fixed to the test bench by lateral struts, ground beams, ground anchor bolts, and compression beams. The vertical load was applied through the reaction beam, jack, pressure sensor, steel distribution beam, and loading beam, and a sliding plate support was set between the pressure sensor and the steel distribution beam. The horizontal load was applied through the loading frame, an MTS system horizontal actuator, loading head, tension bar, and loading plate.

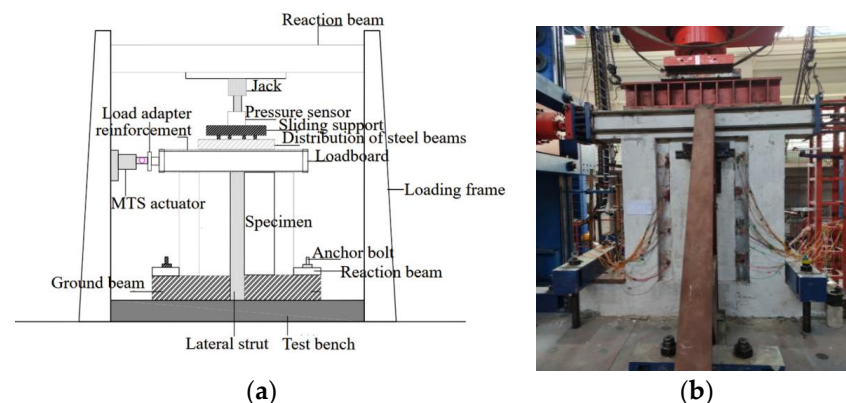


Figure 4. Test loading device. (a) Loading device; (b) Live loading photo.

3.4. Loading System

The test loading system is shown in Figure 5. Before the test, a pre-load was applied to check the equipment for connection reliability and data measurement accuracy. In test loading, the predetermined vertical load was applied and remained constant, and then the horizontal load was applied. The controlled loads were 1000 kN vertical and 200 kN horizontal. The horizontal load was cycled once per stage, and the vertical load was calculated using an axial compression ratio of 0.4, totaling 4000 kN. Until the specimen yielded (at the horizontal load F_y), the loading was changed to cyclic displacement loading, with the grade difference of yield displacement Δ and cycling twice per stage. The test ended when the bearing capacity dropped to 85% of the peak load [15,16]. The push direction was positive, while the pull direction was negative.

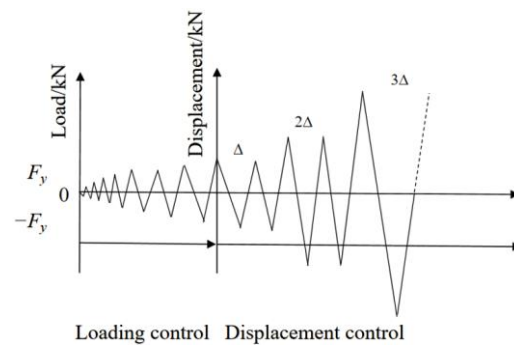


Figure 5. Horizontal loading system.

3.5. Measurement Program

Figure 6 shows the arrangement of sensors for testing specimen ZP. Six displacement gauges measured the horizontal displacement of the loading beam (D1), the horizontal displacement of the shear wall (D2–D4), and the vertical and horizontal displacements of the ground beam (D5–D6). Eight reinforcement strain gauges were arranged to measure the strain of longitudinal bars at the outer corners of the column footings (S1–S2, S1'–S2') and at the outer footings of the walls (S3–S4, S3'–S4'). Four strain gauges were arranged at the upper and lower ends of the steel plates of RCFSTK A. Four strain gauges (A1–A4) measured the strain generated by bending stress, and one strain flower (A5) was placed at the center of the side steel plate to measure the strain generated by shear stress. The other RCFSTKs (B–H) were arranged with the same strain gauge and flower patterns as RCFSTK A.

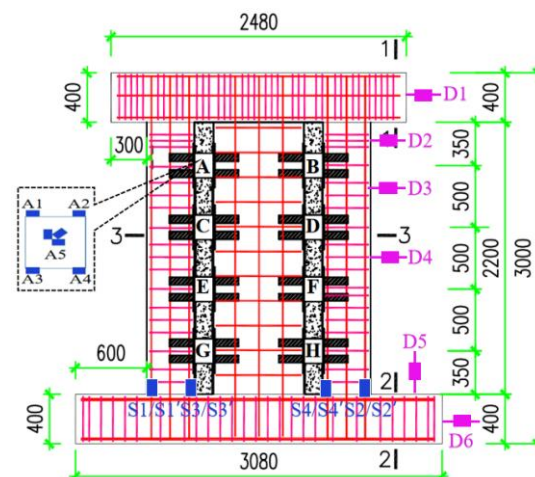


Figure 6. Arrangement of specimen ZP displacement meters and strain gauge; the displacement meters and steel strain gauge arrangement of specimen XJ were the same as for specimen ZP.

4. Specimen Damage and Comparative Analysis

4.1. Specimen Damage

4.1.1. Specimen XJ

There was little sign of damage as loading commenced. When loaded to -750 kN, a slight diagonal crack appeared in the upper left corner of the shear wall. When loaded to 1200 kN, a diagonal crack appeared in the lower right corner of the shear wall. When loaded to -1200 kN, a vertical crack appeared in the middle left side of the shear wall. When loaded to 1600 kN, the specimen yielded, and the loading became displacement loading. When loaded to -1Δ , a longer diagonal crack appeared in the right side of the shear wall oblique cracks. When loaded to $+2\Delta$, oblique cracks appeared at the foot of the right frame column, and as displacement increased, oblique cracks developed and extended to the left and right sides of the shear wall. When loaded to -2Δ , additional horizontal bending cracks appeared in the frame column, and the foot of the left column, the right side of the wall, and the right column were obviously damaged. When loaded to $+3\Delta$, the foot of the column and the foot of the wall were seriously damaged, and the bearing capacity was $<85\%$ of the maximum. When the displacement returned to zero, the test ended. The final damage of the specimen XJ is shown in Figure 7.



Figure 7. Final damage of specimen XJ.

4.1.2. Specimen ZP

There was also little sign of damage as loading started. When loaded to 600 kN, vertical cracks appeared in the center of the shear wall. When loaded to 1000 kN, vertical cracks appeared in the horizontal direction of the shear wall at about a quarter of the wall length. When loaded to 1200 kN, diagonal cracks appeared at the upper part of the foot of the frame column. When loaded to 1600 kN, the specimen yielded, and the loading became displacement loading. When loaded to -1Δ , horizontal cracks appeared in the lower left part of the shear wall, and diagonal cracks appeared at the lower end of the vertical cracks and reinforced concrete blocks in the middle of the shear wall. When loaded to $+2\Delta$, more diagonal cracks appeared in the shear wall, and diagonal cracks appeared in the upper part of the right frame column. When loaded to -2Δ , more uniform damage occurred in both the shear wall and frame column. When loaded to $+3\Delta$, cracks developed and extended rapidly in the specimen, and the damage was obvious. The bearing capacity was $<85\%$ of the maximum value. When the displacement returned to zero, the test was stopped. The final damage of the specimen ZP is shown in Figure 8.



Figure 8. Final damage of specimen ZP.

4.2. Comparative Analysis of Damage

The damage to specimens XJ and ZP were compared. The locations of the first crack in the frame column were different. The first crack in XJ appeared at the foot of the frame column, while the first crack in ZP appeared at the upper part of the column foot and was more evenly distributed vertically along the column. The vertical distributions of cracks in the two types of shear walls were different: the cracks in ZP were more evenly distributed along the height of the wall, while the cracks in XJ were distributed mainly in the lower part of the wall. The reason for these differences is that when an RCFSTK transfers the internal forces between the frame column and the shear wall, it produces a concentrated force perpendicular to the frame column. This changes the location where the maximum bending moments of the frame columns and the shear wall appear so that both the columns and the wall of ZP are subjected to more uniform bending moments in the vertical direction than XJ. Therefore, the fabricated frame–shear wall structure using RCFSTKs to connect the wall and columns, ZP, withstands the applied force better than XJ.

The sequence of the occurrence of vertical and diagonal cracks in the shear wall differed between specimens XJ and ZP. In XJ, diagonal cracks appeared first and then vertical cracks; in ZP, that order was reversed. The corresponding horizontal loads were different for the appearance of vertical cracks, with the loads being 1200 kN for XJ and 600 kN for ZP. The load was greater for XJ than for ZP because the elastic modulus of the grout was less than that of the concrete, and there might also be gaps in the joint parts introduced during assembly, which together led to the uncoordinated vertical deformation of the assembly.

5. Analysis of Test Results

5.1. Load–Displacement Hysteresis Curve

The load–displacement hysteresis curves of specimens XJ and ZP are shown in Figure 9.

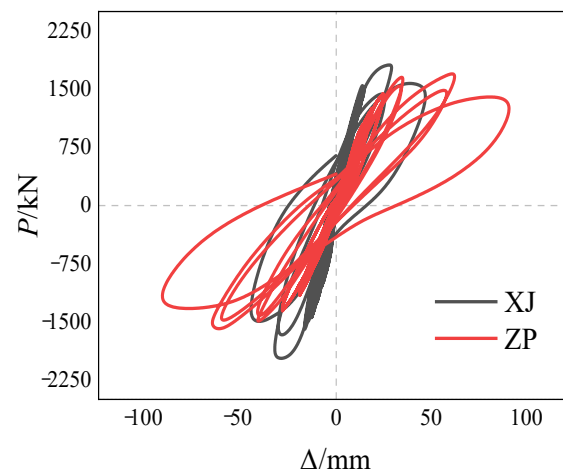


Figure 9. Load–displacement hysteresis curves of specimens XJ and ZP.

5.2. Skeleton Curve

Figure 10 shows the load–displacement skeleton curves of specimens XJ and ZP. Table 3 lists the corresponding characteristic index values. Characteristic index values were defined according to Reference [17]: peak load is the maximum load on the specimen; damage load is 85% of the peak load; yield load is calculated by the energy equivalence method; the ductility coefficient is the ratio of damage displacement to yield displacement; initial stiffness is the ratio of 20% of yield load to the corresponding displacement.

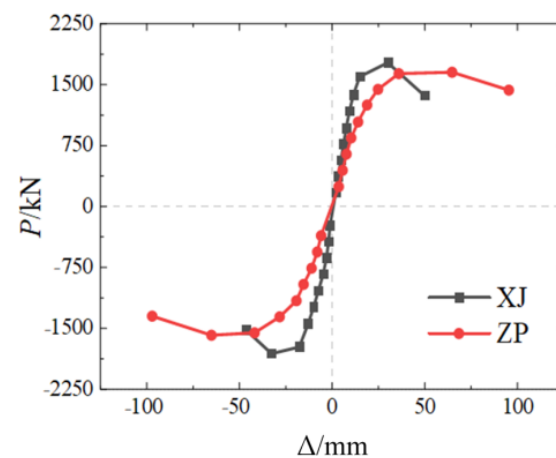


Figure 10. Load–displacement skeleton curves of specimens XJ and ZP.

Table 3. Characteristic index values in the load–displacement skeleton curves of specimens XJ and ZP.

Specimen ID	Yield Load (kN)	Ultimate Load (kN)	Failure Load (kN)	Initial Stiffness (kN/mm)	Ductility Ratio
XJ	1583	1769	1503	131	2.7
ZP	1443	1653	1405	101	3.3

Figure 10 and Table 3 illustrate that: the load–displacement skeleton curves of each specimen have distinct elastic, elastoplastic, and decreasing sections. The skeleton curve of ZP was below that of XJ in both elastic and inelastic rising sections; yield load, ultimate load, failure load, and initial stiffness were 9%, 6%, 6%, and 23% less, respectively. However, the inelastic decreasing section of ZP was significantly above XJ and decreased slowly; the ductility ratio was 22% greater than that of XJ. This was due to the elastic modulus of the grout being less than that of the concrete. Voids might also have been introduced

into the joint parts during assembly, which resulted in a lower initial stiffness of the fabricated structure and less bearing capacity than that of the cast-in-place structure, but the deformation capacity and ductility of the fabricated structure increased.

In summary, the initial stiffness of the fabricated frame shear wall using RCFSTKs to connect the wall and columns (ZP) was less than that of the cast-in-place structure (XJ). However, the difference in initial stiffness was not great, and decreasing the stiffness or adjusting factors that affect stiffness enable the design requirements to be met. The difference between the bearing capacity of ZP and that of XJ was small, while the ductility of ZP was significantly better than that of XJ.

5.3. Stiffness Degradation

The stiffness degradation curves of specimens XJ and ZP are shown in Figure 11. The secant stiffness K (referred to simply as stiffness) indicates the stiffness degradation of the specimens; it is calculated by Equation (1) [18,19]:

$$K = \frac{|P_i| + |-P_i|}{|\Delta_i| + |-\Delta_i|} \quad (1)$$

where P_i and $-P_i$ are the forward and reverse ultimate cyclic loads in cycle i , and Δ_i and $-\Delta_i$ are the displacements corresponding to these loads.

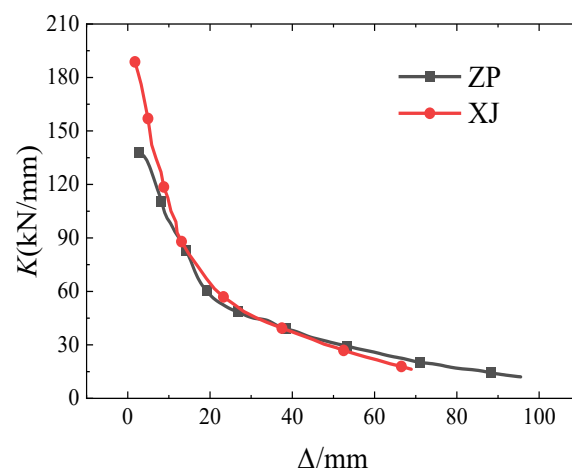


Figure 11. Stiffness degradation curves of specimens XJ and ZP.

Figure 11 shows that at the beginning of loading, the slopes of the stiffness degradation curves of specimens XJ and ZP were steep, indicating that the stiffness of each specimen decreased rapidly. When the displacement was <5 mm, the stiffness of ZP decreased less rapidly than that of XJ. Halfway through loading, the slopes of the stiffness degradation curves of each specimen became less steep, indicating that the stiffness of each specimen decreased slowly; differences between XJ and ZP in stiffness were small. In the later stage of loading, the slopes of the stiffness degradation curves of each specimen further decreased, implying that stiffness degradation changed slowly for both specimens and tended to be the same.

In conclusion, the rate of stiffness degradation of ZP, with the wall attached to the frame columns using RCFSTKs, was slower than that of XJ at the early stage of loading, and the difference in stiffness degradation between XJ and ZP decreased as the load increased and finally converged. The reason for the difference in stiffness degradation at the early stage of loading is similar to that given in the Section Skeleton Curve; the eventual convergence of stiffness degradation is due to the pores in the joint part of ZP being gradually compacted under the horizontal reciprocating load. The difference in elastic modulus between grout and concrete has little effect on the stiffness of the overall structure in the inelastic phase.

5.4. Bearing Capacity Degradation

The bearing capacity degradation coefficients of specimens XJ and ZP are shown in Table 4. The bearing capacity degradation coefficient indicates the degree of continuous degradation in structural bearing capacity caused by plastic deformation of the structure during equal amplitude cyclic loading. It is defined as the ratio of the second to the first cyclic ultimate load [20,21].

Table 4. Bearing capacity degradation coefficients of specimens XJ and ZP.

Specimen ID	1 Δ	−1 Δ	2 Δ	−2 Δ
XJ	0.94	0.94	0.89	0.86
ZP	0.95	0.92	0.96	0.96

Note: Since the damage of the third cycle was severe and the bearing capacity had decreased to <85% of the ultimate load, the third cycle was only cycled once.

It can be seen from Table 4 that in cycles +1 Δ and −1 Δ , the bearing capacity degradation coefficients of specimens ZP and XJ were similar. The reason is that in the first cycle, specimen deformation was primarily elastic deformation. In cycles +2 Δ and −2 Δ , the bearing capacity degradation coefficient of ZP was greater than that of XJ. The reason is that there were gaps between the RCFSTKs, the plain concrete blocks, and the wall and columns. The bearing capacity degradation of ZP is therefore greater than that of XJ.

5.5. Energy Dissipation

The energy dissipation coefficients E of specimens XJ and ZP are shown in Table 5. E are calculated for the cycle of the first line in the table. They measured the energy dissipation capacity of the specimens [22,23]. E is the ratio of the total energy ($S_{(ABC + CDA)}$) to the elastic energy ($S_{(OBE + ODF)}$) of a hysteresis loop. A greater value of E indicates a greater energy dissipation capacity of the structure. The equation for E is shown as Equation (2), and a graph is shown in Figure 12.

$$E = \frac{S_{(ABC + CDA)}}{S_{(OBE + ODF)}} \quad (2)$$

Table 5. Energy dissipation coefficients of specimens XJ and ZP.

Specimen ID	1 Δ	2 Δ	3 Δ
XJ	0.58	0.69	1.23
ZP	0.58	0.75	1.28

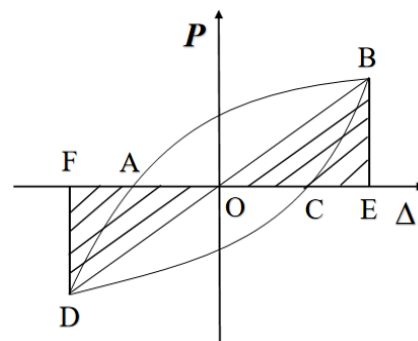


Figure 12. Graphical representation of energy dissipation coefficient calculation.

Table 5 shows that the energy dissipation capacity of specimen ZP was greater than that of XJ, but the overall difference was not significant with the maximum difference <10%. This is because the RCFSTK bonds in ZP affect structural energy dissipation, but

the proportion of RCFSTK bonds in the overall structure is small, so the increase in energy dissipation capacity is insignificant.

5.6. Steel Strain and Steel Tube Stress

To analyze stress conditions in the frame columns and shear walls during the loading of specimens XJ and ZP, the load–strain curves of the reinforcements of each specimen at S1–S2 (at the foot of the column, shown in Figure 6) and S3–S4 (at the foot of the wall) were obtained (Figure 13).

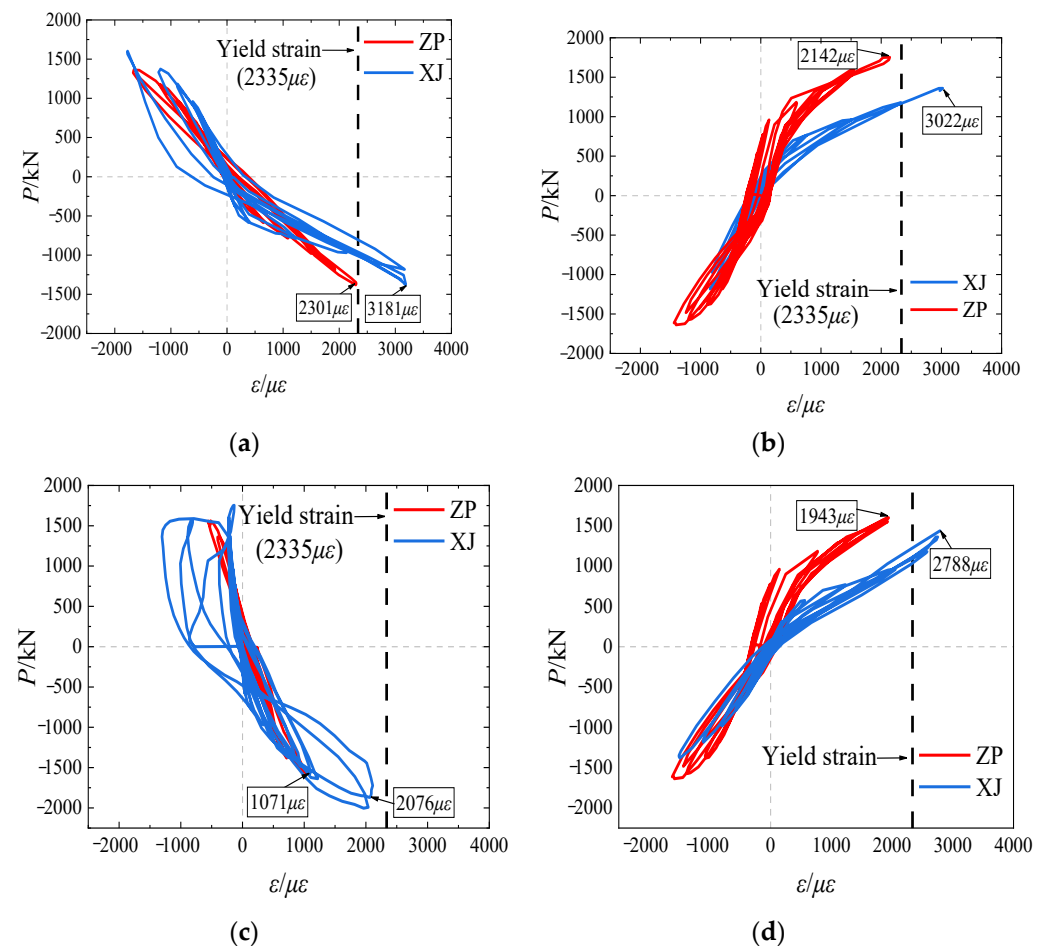


Figure 13. Typical load–strain curves for the steel bars of each specimen. (a) Strain of steel bar at S1; (b) Strain of steel bar at S2; (c) Strain of steel bar at S3; (d) Strain of steel bar at S4.

Figure 13 shows that the reinforcement in both the frame column and the shear wall of specimen XJ yielded or nearly yielded during loading, whereas the reinforcement at the same locations in ZP did not. This indicates that the overall force of the prefabricated frame–shear wall structure using RCFSTKs is uniform, and the force dissipation is better than that of the cast-in-place structure.

To analyze the stress conditions in the interface of the wall and the column for specimen ZP, the load–strain curves of the steel plates in RCFSTKs A1–D1 (upper and lower steel plate ends, shown in Figure 6) and the load–stress curves of the steel plates in E5–H5 (side steel plate centers) were plotted (the stresses were calculated from the strains measured by the strain gauge flowers) (Figures 14 and 15).

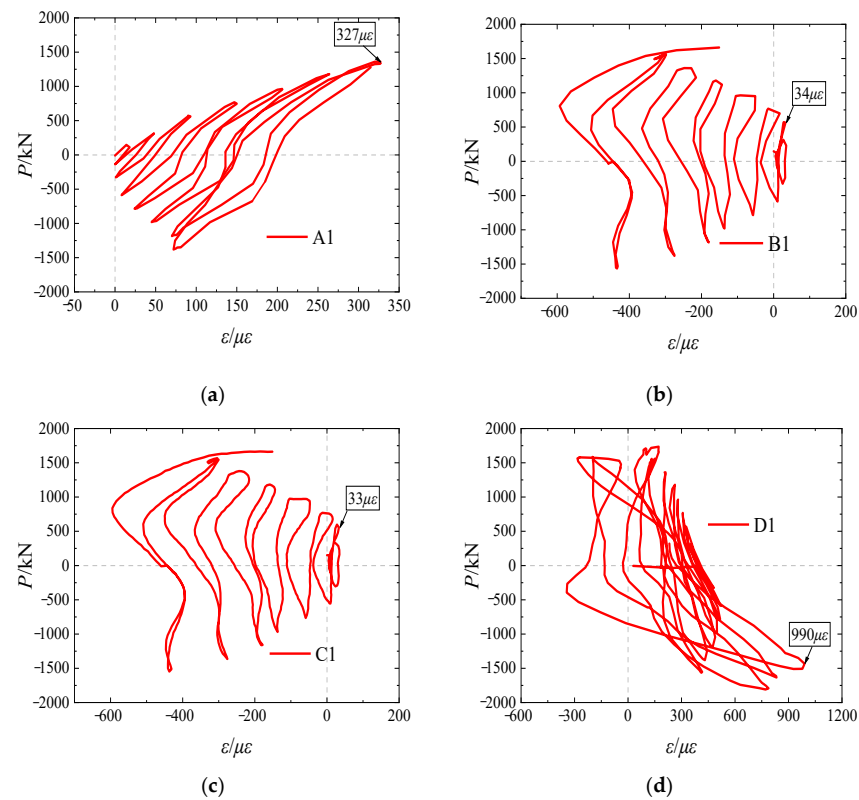


Figure 14. Load–strain curves of the steel plates at the top and bottom of the RCFSTKs. (a) Steel plate strain at RCFSTK A1; (b) Steel plate strain at RCFSTK B1; (c) Steel plate strain at RCFSTK C1; (d) Steel plate strain at RCFSTK D1.

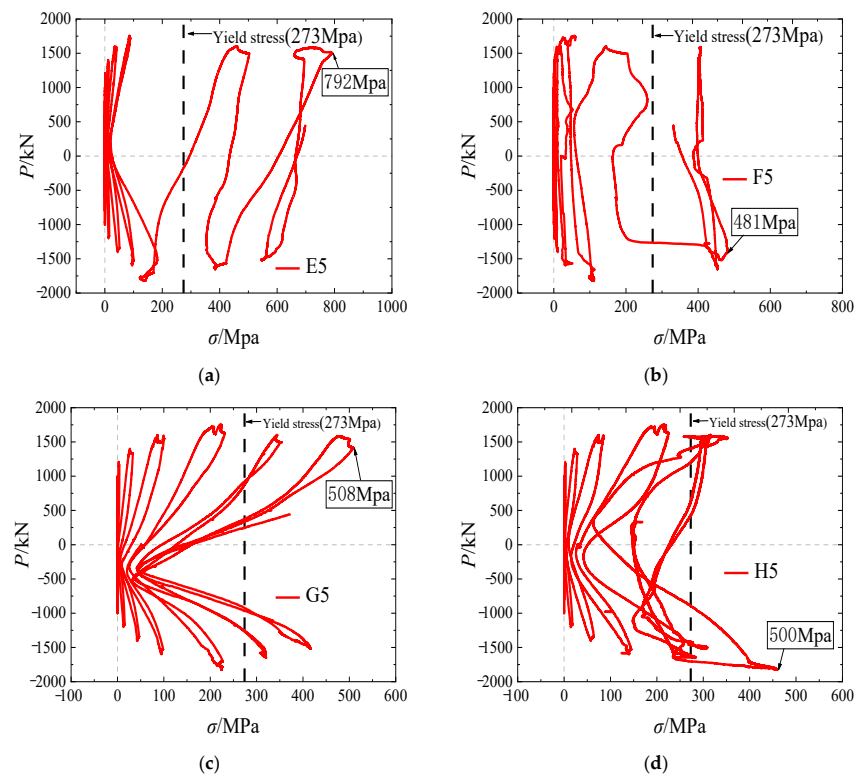


Figure 15. Load–stress curves of the steel plates at the side center of the RCFSTKs. (a) Steel plate stress at RCFSTK E5; (b) Steel plate stress at RCFSTK F5; (c) Steel plate stress at RCFSTK G5; (d) Steel plate stress at RCFSTK H5.

Figure 14 shows that the maximum strain values at the top and bottom ends of the steel plates of the RCFSTK were much lower than the yield strain of the steel plates, so the plates did not yield. Figure 15 shows that the maximum stress values at the center of the side steel plates of RCFSTK were greater than the yield stress of the steel plates, so the side steel plates yielded. This means that RCFSTK failure is predominantly shear deformation and is capable of coordinating the shear deformation of the shear wall and columns.

6. Finite Element Parametric Analysis

6.1. Finite Element Modeling Methods and Verification

We set up several finite element models of the test specimen walls using Abaqus, as described in References [24–27]. The model boundary conditions were consistent with the test arrangement. The attributes of C3D8R were assigned to the concrete solid units, the linear steel units were T3D2, and the steel plate shell units were S4R. Concrete was used in the plastic damage model, steel in the bilinear model, and steel plate in the trilinear model [28]. The material behavior is shown as follows: Figure 16a is the concrete compression behavior; Figure 16b is the concrete tension behavior; Figure 16c is the reinforcement; Figure 16d is the steel plate. The material parameters in the nonlinear principal structure model were defined according to the measured material properties shown in Table 2. The steel bar, anchoring steel plate, and bottom steel plate were embedded in the concrete of the wall and columns using Embedded Region. The end of the steel tube and the bottom steel plate of the embedded elements were connected by binding (Tie). Friction contact with a friction coefficient of 0.6 was set for the interfaces between the steel tube and the concrete, between the concrete in the steel tube and the embedded elements, and between the plain concrete block and the steel tube.

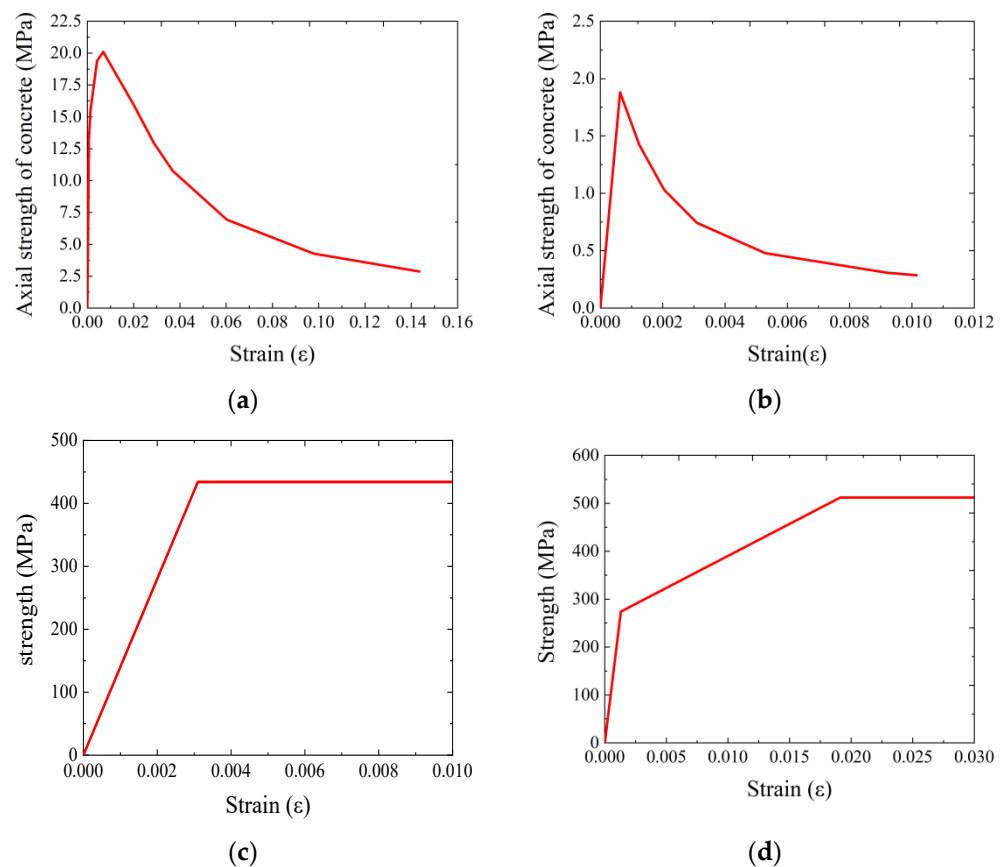


Figure 16. Constitutive Relations. (a) Concrete compression behavior. (b) Concrete tensile behavior. (c) Reinforcement behavior. (d) Steel plate behavior.

The finite element simulation and physical test load–displacement hysteresis curves and skeleton curves are shown in Figures 17 and 18, respectively, for comparison. The characteristic index values of the test and simulated load–displacement skeleton curves are shown in Table 6. The error of each index was generally $\leq 10\%$, and the maximum was $<14.6\%$. These results show that the model is accurate and can be used for subsequent studies.

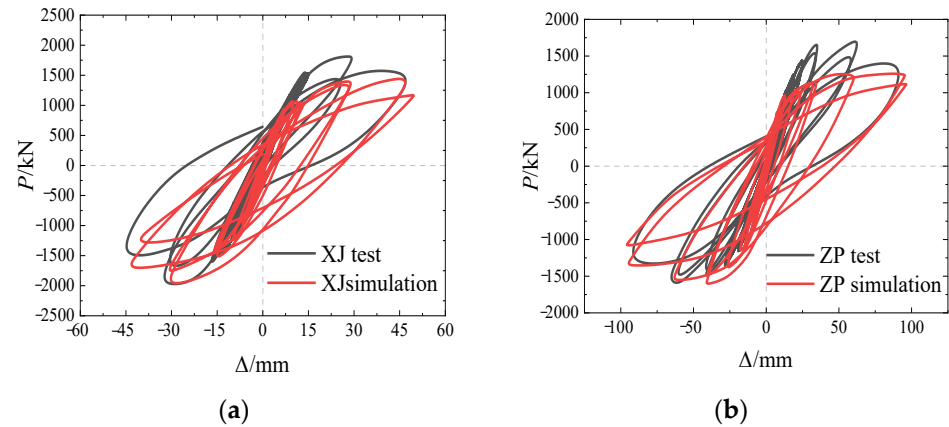


Figure 17. Physical test and simulated hysteresis curves for comparison. (a) XJ; (b) ZP.

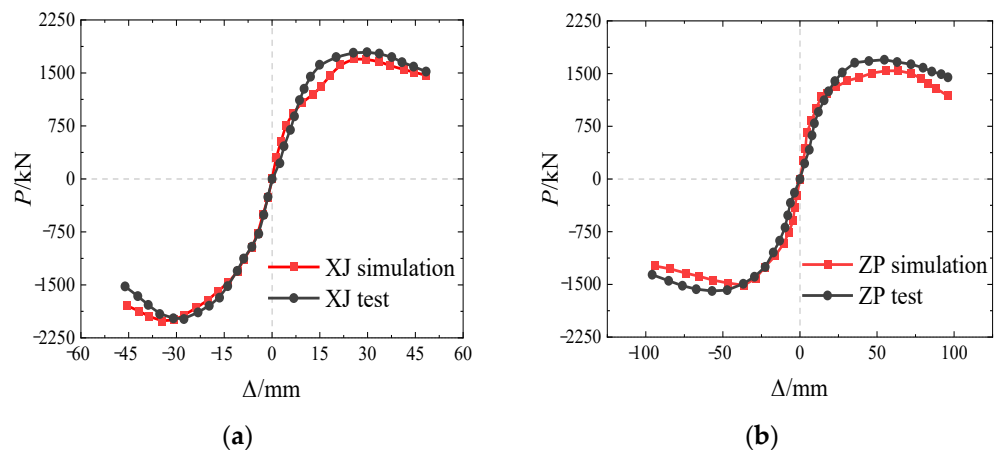


Figure 18. Physical test and simulated load–displacement curves for comparison. (a) XJ; (b) ZP.

Table 6. Comparison results of physical tests and simulation.

Wall ID	Result	Yield	Load (kN) Ultimate	Failure	Initial Stiffness (kN/mm)	Ductility Ratio	Energy Dissipation Coefficient
XJ	test	1583	1769	1503	131	2.7	1.23
	simulation	1353	1658	1409	143	2.9	1.31
	error	14.5%	6.3%	6.3%	9.2%	7.4%	6.5%
ZP	test	1443	1653	1405	101	3.3	1.28
	simulation	1298	1519	1291	112	3.7	1.34
	error	10.0%	8.1%	8.1%	10.9%	12.1%	4.7%

6.2. Necessity of Both RCFSTKs and Plain Concrete Blocks

Although using RCFSTKs and plain concrete blocks to connect the wall and columns can reduce the wet work, making the connection is not straightforward. To determine whether the RCFSTKs and plain concrete blocks must be installed together, as in the physical test, or whether, to simplify construction, the RCFSTKs could be installed without concrete blocks or concrete blocks could be installed without RCFSTKs, we set up an Abaqus model of ZP and three models with equivalent dimensions to simulate the effects

of modifying the shear wall–column interface on skeleton curves and characteristic indexes. Details of the models are shown in Table 7.

Table 7. Model characteristics.

Model	Model Type	Model Characteristics
ZP	Assembly	Set plain concrete blocks between RCFSTKs
JJWK	Assembly	No plain concrete blocks between RCFSTKs
QZYK	Assembly	Gaps between the wall and columns
QZWK	Assembly	No gaps between the wall and columns

The load–displacement skeleton curves of the four models are shown in Figure 19, and the characteristic index values of each model are shown in Table 8.

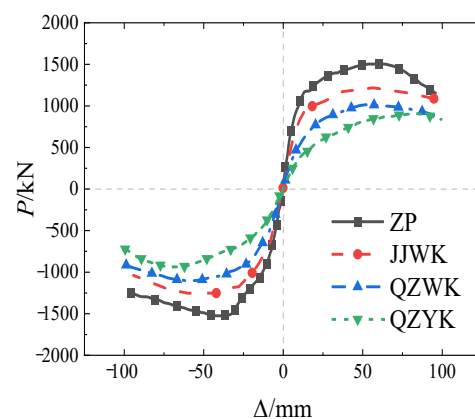


Figure 19. Load–displacement skeleton curves of each model for comparison.

Table 8. Characteristic indexes given by load–displacement skeleton curves for each model.

Model	Ultimate Load (kN)	Failure Load (kN)	Yield Load (kN)	Initial Stiffness (kN/mm)
ZP	1519	1291	1298	138
JJWK	1193	1014	951	102
QZYK	884	752	683	43
QZWK	1028	874	833	66

Figure 19 and Table 8 show that, in absolute terms, the model load–displacement skeleton curve of JJWK (without plain concrete blocks between RCFSTKs) was significantly lower than that of ZP. The yield load, ultimate load, failure load, and initial stiffness of JJWK were 26.7%, 21.5%, 21.5%, and 26.1% less than those of ZP. The load–displacement skeleton curve of QZWK (without RCFSTKs and without gaps between the wall and columns) was lower than that of ZP, and the yield load, ultimate load, failure load, and initial stiffness were all significantly less than those of ZP. The QZWK curve was higher than that of specimen QZYK (with gaps between the wall and column) but lower than the ZP curve, and its yield load, ultimate load, failure load, and initial stiffness were 35.8%, 32.3%, 32.3%, and 52.2% less than those of curve ZP. These results indicate that it is essential that the wall–column connection includes both RCFSTKs and plain concrete blocks; if not, the bearing capacity and initial stiffness of the wall and column connection will be significantly insufficient.

The corresponding plastic strain clouds of each model were obtained for the breaking points of the models (Figure 20) to determine the causes of the preceding results.

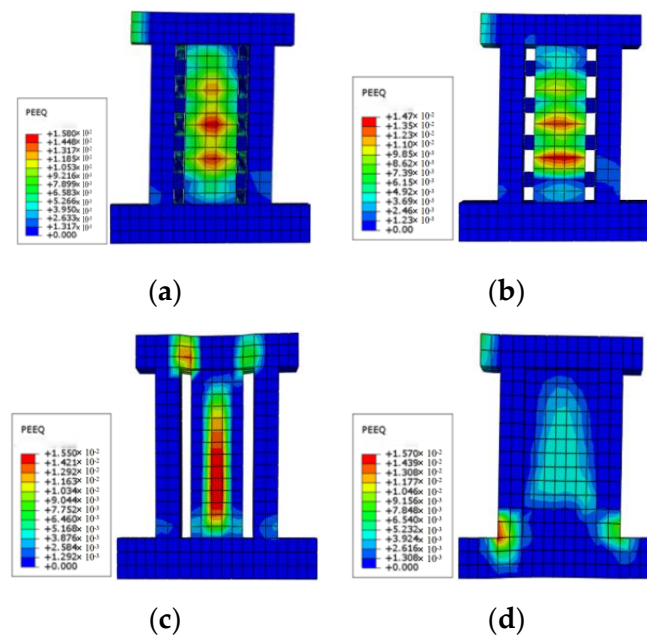


Figure 20. Concrete plastic strain cloud charts for each model. (a) Model ZP; (b) Model JJWK; (c) Model QZYK; (d) Model QZWK.

Figure 20 shows that the concrete damage mechanisms of models JJWK and ZP were similar. Both show the damage to the shear wall was distributed between the RCFSTKs, which indicates that the RCFSTKs coordinate the deformation between walls and columns. Because there were no plain concrete blocks between RCFSTKs in JJWK, the initial stiffness and bearing capacity were less than those of ZP. The damaged area of QZWK was concentrated at the foot of the column and showed prominent bending damage; the shear wall was less affected by the forces, so its load–displacement skeleton curve was higher than that of QZYK. In QZYK, there was a gap between the wall and the column, so the loaded beam and shear wall both show the damage caused by the action of axial force (i.e., the damaged area was distributed vertically), and its load–displacement skeleton curve was the lowest.

6.3. Analysis of Influencing Factors

We set up 20 models to study the effects of various factors on the cyclic performance of the shear wall using RCFSTKs. Parameters that were varied in the models were the number of RCFSTKs n , RCFSTK unit height h , RCFSTK steel tube wall thickness t , the axial compression ratio of ZP ζ , the reinforcing steel strength Q , and the concrete grade G . The values of the characteristic indexes of the load–displacement skeleton curves for each model are shown in Table 9. Only one parameter was varied in each model and the default parameter values were the settings for model ZP.

Table 9. Values of characteristic indexes of load–displacement skeleton curves for each model.

Parameter Value	Yield Load (kN)	Ultimate Load (kN)	Failure Load (kN)	Initial Stiffness (kN/mm)	Ductility Ratio	Energy Dissipation Coefficient
$n = 2$	1224	1486	1263	95	2.7	0.39
$n = 4$	1242	1508	1282	158	2.8	0.47
$n = 6$	1413	1885	1603	160	3.0	0.58
$n = 8$	1421	1889	1606	160	3.2	0.67
$h = 15$	1356	1874	1593	160	3.0	0.65
$h = 20$	1421	1889	1606	160	3.2	0.67
$h = 25$	1533	2010	1709	162	2.8	0.62
$t = 4$	1280	1519	1291	112	3.7	1.34
$t = 8$	1421	1889	1606	160	3.2	0.67
$t = 12$	1228	1755	1492	162	3.8	0.84
$\zeta = 0.2$	1209	1698	1443	160	3.6	0.68
$\zeta = 0.4$	1421	1889	1606	160	3.2	0.67
$\zeta = 0.5$	1507	1984	1686	160	2.8	0.54
Q-Q235	1421	1889	1606	160	3.2	0.67
Q-Q390	1568	1998	1698	163	2.9	0.63
Q-Q420	1593	2048	1741	167	2.8	0.61
G-C25	1562	2050	1743	142	2.8	0.61
G-C30	1553	1909	1623	153	2.9	0.63
G-C40	1421	1889	1606	160	3.2	0.67
G-C50	1411	1879	1597	164	3.4	0.73

6.3.1. Influence of the Number of RCFSTKs

Figure 21 shows the load–displacement skeleton curves for each model with vertical and cyclic loading when the number of RCFSTKs (n) was 2, 4, 6, or 8.

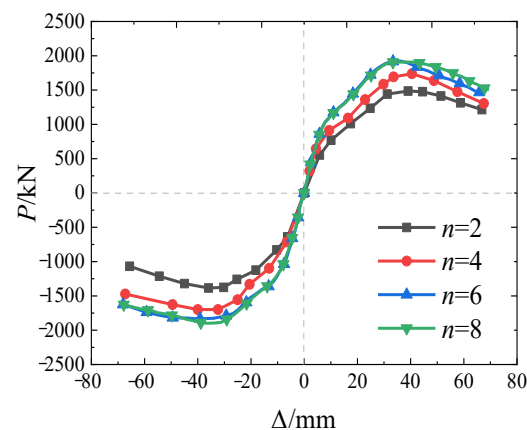
**Figure 21.** Load–displacement skeleton curves for models with different numbers of RCFSTKs.

Figure 21 and Table 9 show that bearing capacity increased as the number of RCFSTKs increased. When the number of RCFSTKs increased from 2 to 6, the values of all characteristic indexes of the specimen increased; the yield load, ultimate load, and failure load of the models increased by 15.4%, 26.9%, and 26.9%; the initial stiffness increased by 68.4%; and the ductility ratio and energy dissipation coefficient increased by 11.1% and 48.7%. When the number of RCFSTKs increased from 6 to 8, the energy dissipation coefficients increased by 15.5%, and the other characteristic index values increased by <5%. This is explained as follows. When the number of RCFSTKs is small, they cannot coordinate the deformation between walls and columns. The values of each characteristic index therefore increase as the number of RCFSTKs increases. When the number of RCFSTKs is sufficient to coordinate the deformation between walls and columns, further increasing the number

becomes equivalent to increasing the steel content inside the structure that resists structural stress, and only the energy dissipation capacity and ductility of the structure increase.

In summary, an increase in the number of RCFSTKs significantly influenced the cyclic performance of the structure. An increase within a particular range will enhance various cyclic performance indexes of the structure. However, beyond this range, continuing to increase the number of RCFSTKs will only increase the ductility and energy dissipation capacity of the structure.

6.3.2. Influence of RCFSTK Unit Height

Figure 22 shows the load–displacement skeleton curves for the models in which the unit height of the RCFSTKs was varied ($h = 15, 20$, and 25 cm) under vertical load and cyclic loads with other parameters were unchanged.

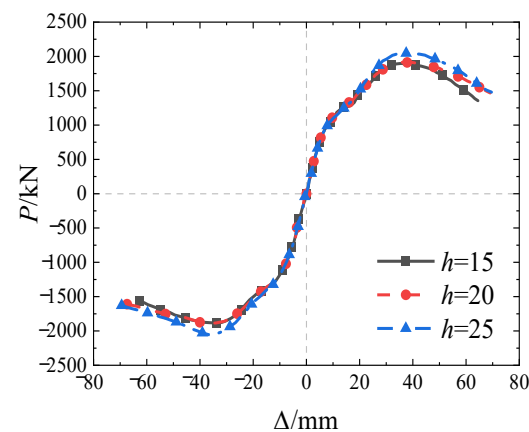


Figure 22. Load–displacement skeleton curves for models with different RCFSTK unit height.

Figure 22 and Table 9 show that there were no significant differences in the load–displacement skeleton curves of the models. When the unit height of the RCFSTK was increased from 15 to 25 cm, the yield load, ultimate load, and failure load of the specimen increased by 13.1%, 7.3%, and 7.3%, respectively. Since the RCFSTK deformation was primarily shear deformation, increasing its unit height improved its resistance to shear deformation, so the overall structural load carrying capacity was enhanced. The initial stiffness, ductility ratio, and energy dissipation coefficient of the models all increased and then decreased as the unit height increased. However, the overall change was small. When the RCFSTK unit height increased from 15 to 20 cm, the ductility ratio increased by 6.7%, but the other index values increased by <3%. When the RCFSTK unit height increased from 20 to 25 cm, the ductility ratio decreased by 12.5%, and the energy dissipation coefficient decreased by 7.3%; all other index values decreased by <4%. The main reason for this is that an increase in RCFSTK unit height is effectively an increase in steel content inside the structure, so the index values increase. However, when the unit height exceeds a particular value, deformation and damage of the steel plate are reduced. The damage is transferred to the concrete, so the index values decrease. In summary, the change in RCFSTK unit height has little effect on the cyclic performance of the structure in general; as the section height increases, the bearing capacity of the structure improves, and the ductility ratio and energy dissipation capacity first increase and then decrease; the initial stiffness is almost unchanged.

6.3.3. Influence of Wall Thickness of RCFSTKs

Figure 23 shows the load–displacement skeleton curves of the models in which the RCFSTK steel tube wall thickness t was 4, 8, and 12 mm under vertical load and cyclic loads with all other parameters unchanged.

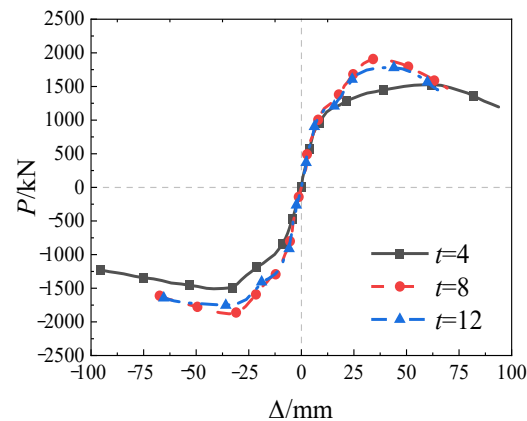


Figure 23. Load–displacement skeleton curves for models with different RCFSTK steel tube wall thickness.

Figure 23 and Table 9 show that the wall thickness of the steel tube has an obvious impact on the load–displacement skeleton curve. The yield load, ultimate load, and failure load of the models first increased and then decreased, and when the wall thickness of the steel tube increased from 4 to 8 and then to 12 mm, the values of the indicators first increased respectively by 11.0%, 24.4%, and 24.4% and then decreased by 13.6%, 7.1%, and 7.1%. The initial stiffness of the models consistently increased; when the wall thickness of the steel tube increased from 4 to 12 mm, initial stiffness increased by 15.2%. The ductility ratios of the models and the energy dissipation coefficients first decreased and then increased. When the wall thickness of the steel tube increased from 4 to 8 and then to 12 mm, these indicators first decreased respectively by 15.8% and 50% and then increased by 18.8% and 25.4%. The principal reason is that the RCFSTKs undergo shear deformation, and increasing the wall thickness of the steel plate improves the capability of the RCFSTK to resist shear deformation. However, as the wall thickness increases, the damage is transferred to the concrete, so the index value has a turning point.

In summary, these three models show that the change in steel tube wall thickness has a pronounced effect on the seismic performance of the structure. As the steel tube wall thickness increases, the bearing capacity of the structure first increases and then decreases, while the ductility and energy dissipation capacity first decrease and then increase, and the initial stiffness always increases.

6.3.4. Influence of Wall Thickness of RCFSTKs

Figure 24 shows the load–displacement skeleton curves of each model under vertical and cyclic loads with axial compression ratios (ζ) 0.2, 0.4, and 0.5, and other parameters unchanged.

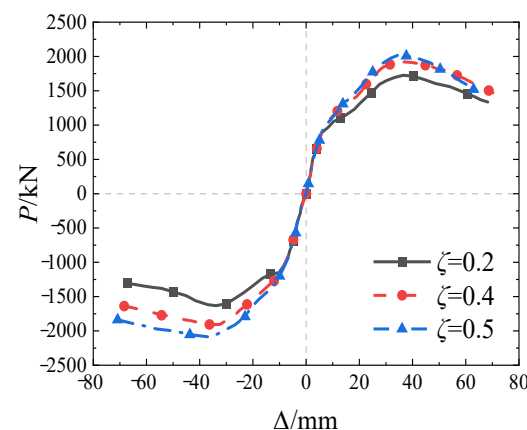


Figure 24. Load–displacement skeleton curves of models with different axial pressure ratios.

Figure 24 and Table 9 show that the load–displacement skeleton curves of the models differed significantly with different axial pressure ratios. When the axial pressure ratio increased from 0.2 to 0.5, the yield load, ultimate load, and failure load of the models increased by 24.6%, 16.8%, and 16.8%, respectively, while the ductility ratio and energy dissipation factor decreased by 22.2% and 20.6%, respectively, with no change in initial stiffness. The reasons for these changes are similar to those for the changes in the cast-in-place structure caused by different axial pressure ratios. Change in axial pressure ratio has a pronounced effect on the cyclic performance of the structure. As the axial pressure ratio increases, the bearing capacity of the structure increases, while the ductility and energy dissipation capacity decrease, and the initial stiffness remains unchanged.

6.3.5. Influence of RCFSTK Steel Strength

Figure 25 shows the load–displacement skeleton curves of each model with RCFSTK steel strengths (Q) of Q235, Q390, and Q420 under vertical load and cyclic load and all other parameters remained unchanged.

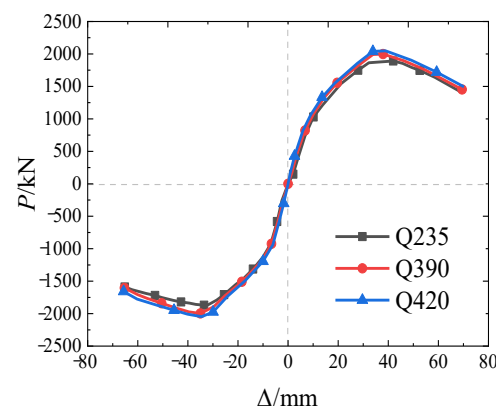


Figure 25. Load–displacement skeleton curve of models with different RCFSTK steel strengths.

Figure 25 and Table 9 show that the load–displacement skeleton curves of the specimens differed slightly. When the RCFSTK steel strength increased from Q235 to Q420, the yield load, ultimate load, failure load, and initial stiffness of the models increased by 12.1%, 8.4%, 8.4%, and 4.1%, respectively. The displacement ductility ratio and energy dissipation coefficients decreased by 12.5% and 9.0%. The principal reason for these changes is that the RCFSTKs are mainly subjected to shear deformation and increasing steel strength can increase RCFSTK resistance to shear stress and thus decrease deformation. Overall, the change in the steel strength of the RCFSTKs has little effect on the cyclic performance of the structure. As steel strength increases, bearing capacity and initial stiffness of the structure increase, while the ductility ratio and energy dissipation capacity decrease.

6.3.6. Influence of RCFSTK Concrete Grade

Figure 26 shows the load–displacement skeleton curves of each model with concrete grades in RCFSTK (G) of C25, C30, C40, and C50 under vertical and cyclic loads and other parameters remained unaltered.

Figure 26 and Table 9 show that the load–displacement skeleton curves of the models differed slightly. When the strength of the concrete inside the steel tube increased from C25 to C50, the yield load, ultimate load, failure load, and initial stiffness of the specimen increased by 9.6%, 8.4%, 8.4%, and 12.3%, respectively, and the displacement ductility ratio and energy dissipation coefficient decreased by 21.4% and 19.7%, respectively. The reason for these changes is similar to the reason for changes in RCFSTK steel strength. The change in RCFSTK concrete grade generally had little effect on the cyclic performance of the models. As the concrete grade of the shear wall and columns increases, the bearing capacity and the initial stiffness of the structure increase, while the ductility ratio and energy dissipation capacity decrease.

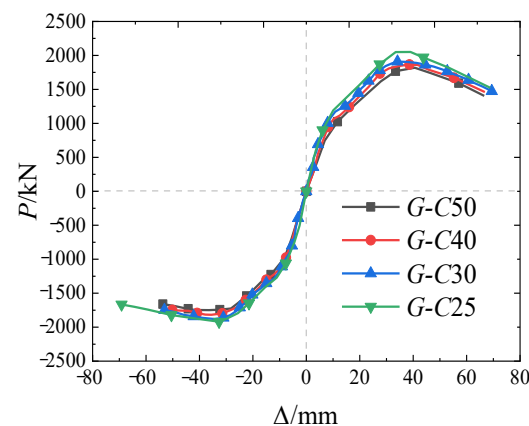


Figure 26. Load–displacement skeleton curves of models with different RCFSTK concrete grades.

7. Conclusions

To find a method of connecting a shear wall and columns that can both reduce the amount of wet work required and maintain adequate stiffness, we propose connecting the shear wall to the vertical frame using rectangular concrete-filled steel tube keys (RCFSTKs). Static tests and finite element models were used to study their cyclic performance. The feasibility of the new method was analyzed. In addition, the influence of different parameters on the cyclic performance of the assembled structure was identified. The following conclusions can be obtained.

- (1) In terms of transferring force, RCFSTK produces a concentrated force perpendicular to the frame column when transferring the internal force between the column and the wall. This force changes the locations of the maximum bending moments of the column and the wall, making the bending moments more uniform in the vertical direction and improving the mechanical properties of the wall and the column.
- (2) The RCFSTKs yielded in the center of their lateral steel plates, while the ends of the steel plates on the upper and lower surfaces did not yield. This indicates that the RCFSTKs are predominantly subjected to shear deformation and are important in coordinating shear deformation of the walls and columns.
- (3) The prefabricated frame–shear wall structure using RCFSTKs, compared with the cast-in-place structure, has a fuller hysteresis curve, greater total energy consumption, better ductility, slightly higher energy consumption capacity, and slightly lower stiffness degradation and bearing capacity. The difference in bearing capacities was slight, so practical use of the proposed prefabricated structure is feasible.
- (4) Removing the RCFSTKs or the plain concrete blocks between them, or both, will significantly decrease the bearing capacity and initial stiffness of the prefabricated column–wall unit by 21.5%–52.2%. Thus, both RCFSTs and the concrete blocks between them are necessary. How to simplify the construction merits further investigation.
- (5) The cyclic performance of the prefabricated structure is influenced largely by the number of RCFSTKs, steel tube wall thickness, and axial compression ratio while to a lesser extent by the key height, steel strength, and concrete grade of the RCFSTKs.

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