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A Study on the Bearing Performance of an RC Axial Compression Shear Wall Strengthened by a Replacement Method Using Local Reinforcement with an Unsupported Roof

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Abstract: When compared with conventional replacement reinforcement methods, the method of replacement using a local reinforcement with an unsupported roof has the advantages of shortening the reinforcement cycle and reducing material loss, and many scholars have carried out useful explorations thereof. At present, the formula for the bearing capacity of reinforcement by replacing concrete in the Code does not consider the effect of stress hysteresis on the parts of the reinforcement; so when the initial stress level is greater than 0.4, the Code's strength utilization coefficient of 0.8 for the new concrete in the replaced area is on the side of insecurity. In this study, we are trying to improve and supplement the formula in the Code through the following work. Firstly, 18 groups of shear wall models were constructed using a VFEAP finite element analysis program to analyze the bearing performances of the shear walls after the replacement. The results showed that the replacement concrete strength, the initial stress level and the size of the replaced area had a significant influence on the bearing capacity of the shear wall after the replacement. Secondly, utilizing the replacement concrete strength, the initial stress level and the size of the replacement area as key parameters, then introducing the strength improvement coefficient considering the constraints of the stirrups, the modified strength utilization coefficient of new concrete in the replaced area was formulated. Finally, based on the modified strength utilization coefficient, the replacement bearing capacity formulas for the one-batch, two-batch, and three-batch replacements were derived, and an N-batch replacement bearing capacity formula was regressed and fitted on the basis of these equations, which are less discrete and more secure than the Code's formula.

Keywords: shear wall; replacement method using local reinforcement with an unsupported roof; stress hysteresis; nonlinear analysis

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1. Introduction

The replacement concrete reinforcement method is favored because it does not increase the size of the structure on which it is used and maintains its original form. There are many engineering examples of RC shear walls that have been strengthened using replacement methods at home and abroad [1,2], and a series of research results have been obtained. The current Code makes it clear that this method is suitable for the local reinforcement of concrete with low strength or serious defects in the compression area of the RC-structure-bearing members, and relevant provisions on the strength grades of non-replacement concrete and replacement concrete are described [3]. In practical engineering, compared to the conventional replacement reinforcement method, the method of replacement using local reinforcement with an unsupported roof can reduce the scope of replacement construction, allow full use of the original section to be made, and shorten the reinforcement period. Therefore, our research group performed a useful exploration of the strengthening of RC shear walls using a local strengthening replacement method without a supporting roof [4].

At present, many scholars in China mainly explore the bearing performance of shear walls strengthened using local replacement reinforcement methods [5]. Liu Yuwei [6] took a residential building under construction as the background and used a local strengthening replacement method to carry out a structural reinforcement analysis and research; finally, it was found that the vertical and lateral bearing capacity of a residential building reinforced with this method could meet the original design requirements. Wang Xin [7] conducted a low-cyclic-reciprocating-load comparison test on four shear wall specimens without initial vertical stress (two of which were locally reinforced replacements at both ends), and the results showed that the C50 concrete or modified concrete in the replaced area had good cooperation with the C20 concrete in the non-replaced area; in addition, the seismic bearing capacity, energy dissipation capacity, ductility, cracking load, and peak load of the replacement specimens were significantly improved. Stress hysteresis refers to the phenomenon where the timing of stress on the new material lags behind that of the old material due to an uncoordinated deformation between the old and new materials. Stress hysteresis may reduce the reinforcement effect of the shear wall, and it can even cause local damage in severe cases. However, up to now, there have been few studies on the cooperative work and overall performance of high-performance materials [8,9] in the replaced areas of RC shear walls and the low-strength concrete in non-replaced areas while considering stress hysteresis. In particular, the bearing capacity of RC shear walls when using high-performance materials for local strengthening replacements and the effects of stress hysteresis have yet to be thoroughly investigated.

In order to study the cooperative work performance of the replaced and non-replaced areas after replacement while considering stress hysteresis, in this study, we performed a useful exploration on it through the following work. Firstly, the VFEAP finite element analysis program was utilized to establish 18 groups of shear wall models. Based on these models, the influences of multiple factors on the bearing performance of the reinforced shear walls were analyzed, and it was pointed out that the formula in the Code is unsafe at high stress levels. Secondly, in order to improve or supplement the Code's formula, this study considered the influences of the replacement concrete strength, initial stress level, and the size of the replaced area; in addition, we introduced the strength improvement coefficient considering the constraints of the stirrups, then established a modified strength utilization coefficient of the new concrete in the replaced area. Based on the modified strength utilization coefficient, we deduced the bearing capacity formulas for the one-batch, two-batch, and three-batch unsupported locally reinforced replacements, and conducted a regression fitting for the bearing capacity formulas of the N-batch replacements. Finally, according to the existing data, the formula derived in this study was compared with the formula in the Code; the results showed that the formula derived here had a higher degree of coincidence with the test results, and the dispersion was smaller, giving it a certain reliability.

2. Analysis Method and an Example

In this study, a nonlinear finite element analysis method based on the theory of a three-dimensional solid degenerate virtual laminated unit [10] was adopted, as this allowed a better simulation of the actual force state and deformation of the structure [11]. The feasibility and accuracy of the VFEAP finite element analysis program were verified through an example, which provided a theoretical basis for the subsequent nonlinear analysis of members reinforced through the use of a replacement method using local reinforcement with an unsupported roof.

2.1. Analysis Method

A three-dimensional solid degraded virtual laminate unit was constructed by introducing the relevant assumptions into isoparametric units of a spatial solid to create three-dimensional solid degraded units using the concepts of "virtual nodes" and "virtual regions" [12]. Blocks with different geometric and material parameters were allowed to exist in the same unit and divisions into units and blocks can be performed according to the actual situation, thus improving the modeling efficiency, analysis efficiency, and greatly increasing the accuracy of the analysis and calculation. In addition, the theory of three-dimensional solid degenerate virtual laminated units involves the double nonlinearity of geometry and material, that is, the T.L. [13] method, which was employed for geometric nonlinearity, and the multiaxial strengthening plastic model from Ohtani and Chen was introduced for material nonlinearity. Finally, in the process of modeling, a three-node one-dimensional isoparametric unit was used to describe the relative positions of the steel bars in any section, allowing the true simulation of the stress conditions of the steel bars.

2.2. Example Verification

The test in [14] was selected for the simulation analysis to verify the feasibility and accuracy of the VFEAP finite element analysis program. The sectional dimensions and reinforcements of the members before and after the reinforcement of the enclosure are shown in Figure 1. The detailed parameters of the relevant samples, the preparation process, and the main experiments are all described in [14].

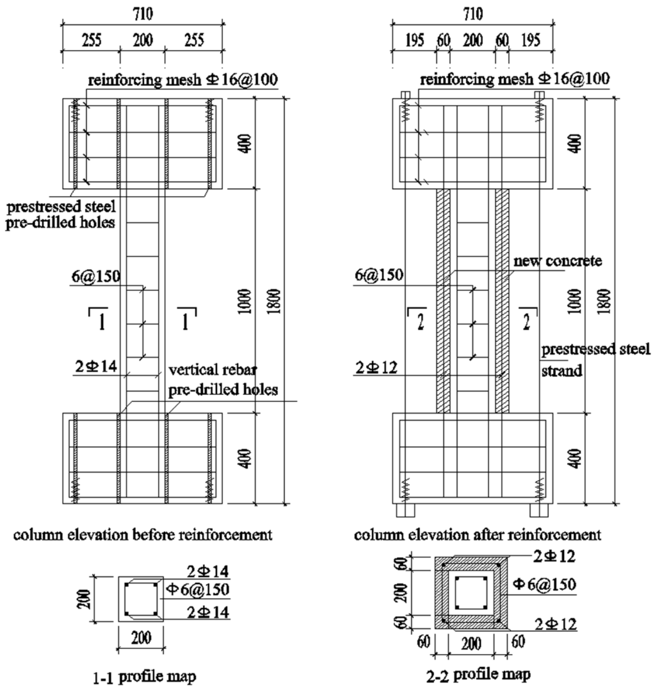


Figure 1. Cross-sectional dimensions and reinforcements of the test specimen.

In the process of a finite element analysis, it is easy for the phenomenon of stress concentration to appear; to prevent stress concentration at the bottom of concrete members in the early stage, which would lead to the termination of the calculation, a rigid pad was established at both ends of the column for force transmission. The models of the concrete and steel bars are shown in Figures 2 and 3.

According to the curve of the axial force–strain relationship of the concrete shown in Figure 4, the failure process and deformation simulated in the finite element analysis program were basically consistent with the test process. In addition, the ultimate value of the axial compression bearing capacity of the BJG specimen simulated with the finite element method was 1175 KN, and the experimental value was 1241 KN, with an error of 5.3%. The ultimate value of the bearing capacity simulated with the finite element method for a JGZ-13 member was 3684 KN, and the test value was 3795 KN, with a difference of 2.92%. The overall error was less than 6%. To sum up, the VFEAP finite element analysis program used in this study was able to simulate the actual failure process and deformation of the structure well, and the finite element analysis results had a high degree of fitting with the test results, which showed that the VFEAP finite element analysis program was completely feasible for a simulation analysis.

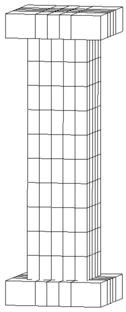


Figure 2. Concrete model.

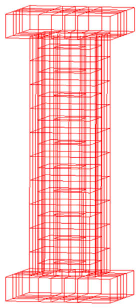


Figure 3. Finite element mesh for steel reinforcement.

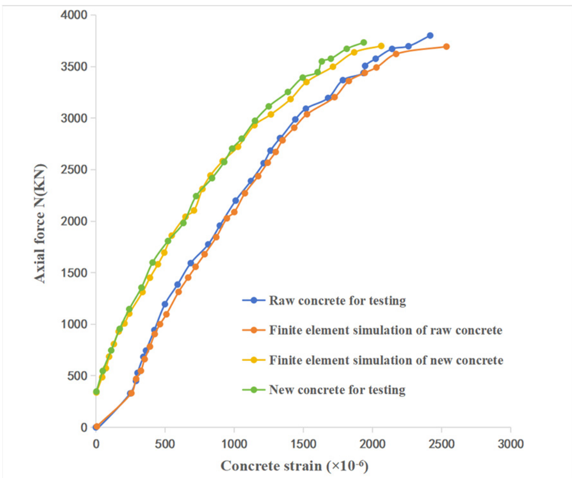


Figure 4. Curve of the relationship between an existing column and newly poured concrete using JGZ-13.

3. Analysis of the Simulation of the Whole Process of Local Strengthening Replacement of an RC Shear Wall without a Supporting Roof under Axial Compression

Based on the verification of the example in the second section, this section takes an RC shear wall reinforced using the local strengthening replacement method without a supporting roof as the research object, and the strengthening mechanism of the RC shear wall is studied using the VFEAP finite element analysis program. In addition, the influences of the initial stress level, replacement concrete strength, and the size of the replacement area on the bearing capacity of the shear wall are comprehensively analyzed. Finally, according to the results of the simulation analysis, it is pointed out that the value of the strength utilization coefficient for the replacement concrete in the Code is unsafe at high stress levels, which provides a theoretical basis for the later derivation of a calculation formula for the bearing capacity of RC shear walls strengthened using the local strengthening replacement method without a supporting roof.

3.1. Analysis Scheme and Model Design

The research object in this simulation analysis was a rectangular-section RC shear wall strengthened using a local one-batch replacement and two-batch replacement without a supporting roof. The section size and reinforcement of the specimen are shown in Figure 5.

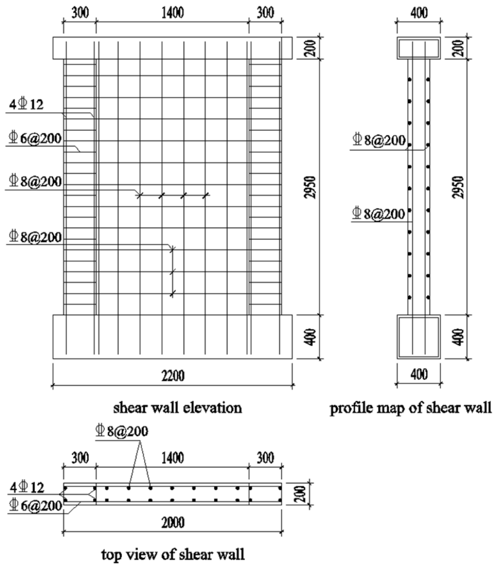


Figure 5. Shear wall section size and reinforcement.

According to the existing research results, this study mainly considered the impacts of the initial stress level, the replacement concrete strength, the size of the replacement area, the replacement batch, and the longitudinal reinforcement ratio for a concealed column; the influencing factors and their levels are listed in Table 1.

Table 1. Experimental influencing factors and their levels.

Influencing Factor	Level					
Initial stress level	0.15	0.25	0.35	0.4	0.5	0.6
Replacement concrete grade	C50		C65		C80	
Replacement section size (mm × mm)	300 × 200		350 × 200		400 × 200	
Reinforcement ratio of a concealed column	0.85%		1.50%		profile steel	

According to the factors and levels selected in this table, experimental schemes with a one-batch replacement and two-batch replacement were designed using the orthogonal test method. The overall test scheme is shown in Table 2.

According to existing research results, during the replacement process of a structure, the stress of each wall segment will be redistributed, and the distribution mainly occurs in the reinforced floor and the two adjacent floors; the stress of the demolished wall will be shared by the undemolished wall, and the stress of the shear wall perimeter and that of the upper and lower floors of the connecting beams, girders, slabs, columns, and other components will be redistributed [15]. This study only considers a single shear wall, so it is assumed that stress redistribution only occurs on this shear wall; the effect of the replacement of shear walls in practical engineering will be better than that of this study.

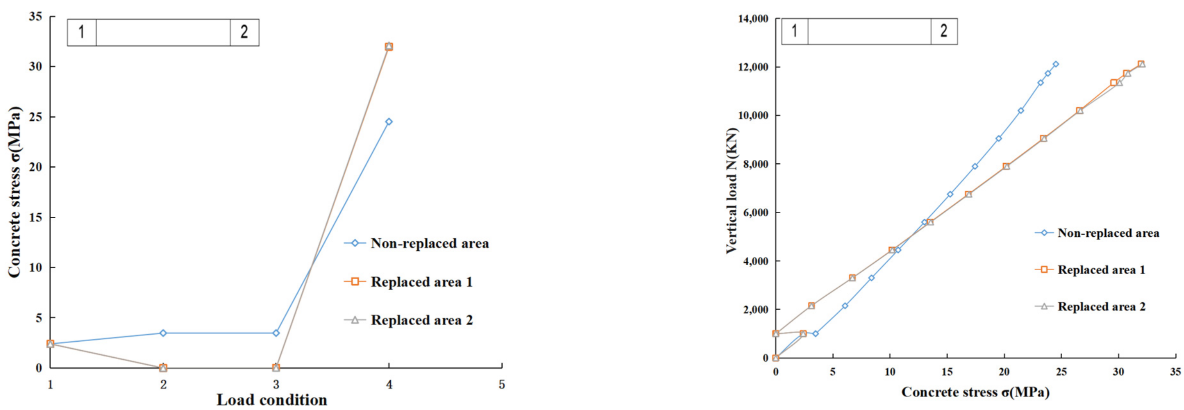
Table 2. Overall test scheme.

Specimen	Initial Stress Level	Replacement Concrete Grade	Length of Replaced Wall (mm)	Reinforcement Ratio of a Concealed Column	Replacement Batch
YPZH-1	0.15	C50	300 × 2	0.85%	one-batch
YPZH-2	0.15	C65	400 × 2	1.50%	one-batch
YPZH-3	0.15	C80	350 × 2	profile steel	one-batch
YPZH-4	0.25	C50	400 × 2	profile steel	one-batch
YPZH-5	0.25	C65	350 × 2	0.85%	one-batch
YPZH-6	0.25	C80	300 × 2	1.50%	one-batch
YPZH-7	0.35	C50	350 × 2	1.50%	one-batch
YPZH-8	0.35	C65	300 × 2	profile steel	one-batch
YPZH-9	0.35	C80	400 × 2	0.85%	one-batch
EPZH-10	0.4	C50	300 × 2	0.85%	two-batch
EPZH-11	0.4	C65	400 × 2	1.50%	two-batch
EPZH-12	0.4	C80	350 × 2	profile steel	two-batch
EPZH-13	0.5	C50	400 × 2	profile steel	two-batch
EPZH-14	0.5	C65	350 × 2	0.85%	two-batch
EPZH-15	0.5	C80	300 × 2	1.50%	two-batch
EPZH-16	0.6	C50	350 × 2	1.50%	two-batch
EPZH-17	0.6	C65	300 × 2	profile steel	two-batch
EPZH-18	0.6	C80	400 × 2	0.85%	two-batch

3.2. Analysis of the Simulation Results

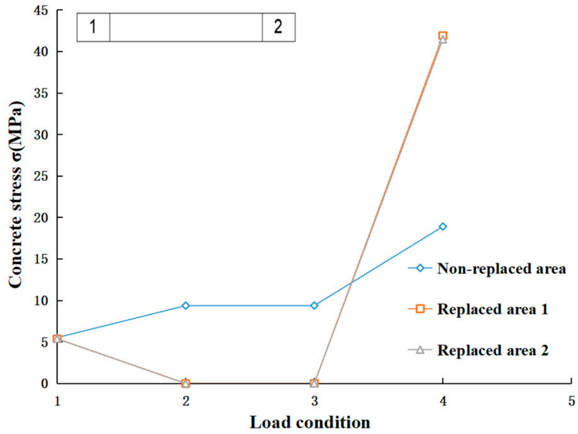
3.2.1. One-Batch Replacement without a Supporting Roof

Experiments using one-batch replacements were carried out at a low stress level (between 0.15 and 0.35). The one-batch replacements were divided into four working conditions: working condition one represented the action of initial stress; working condition two represented the removal of the wall section to be replaced; working condition three represented the casting of the wall section to be replaced with high-strength concrete; and working condition four represented the action of the ultimate load. This study took two components, YPZH-1 and YPZH-9, as representatives for analysis; their working condition stress diagrams and load–stress curves are shown in Figures 6 and 7.



(**a**) (**b**)

Figure 6. (**a**) Stress–strain diagram under different load conditions for YPZH-1; (**b**) load–stress curve for YPZH-1.



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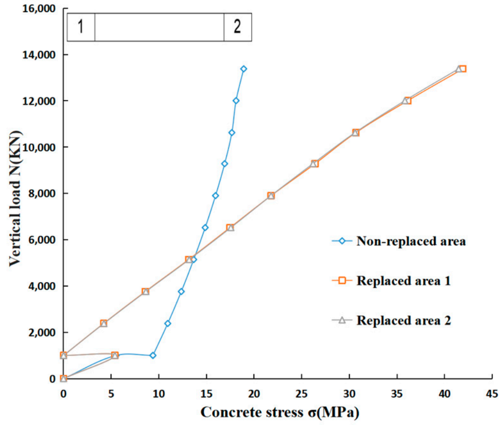


(**b**)

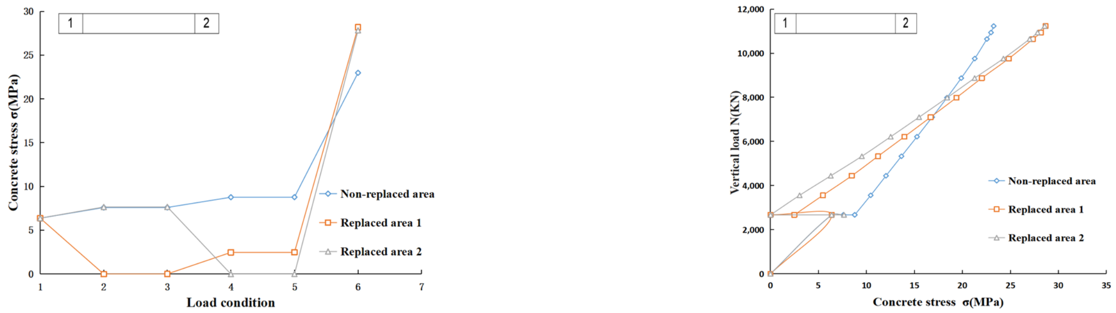
Figure 7. (**a**) Stress–strain diagram under different load conditions for YPZH-9; (**b**) load–stress curve for YPZH-9.

As shown in Figure 6, at the initial stress level of 0.15, the YPZH-1 component was simultaneously dismantled in replaced areas 1 and 2, the load removed from these two parts was completely borne by the middle area that was not replaced. The stress in replaced area 1 and replaced area 2 decreased from the initial 2.4 MPa to 0, and the stress of the middle wall increased from the initial 2.4 MPa to 3.47 MPa. As shown in Figure 7, at the initial stress level of 0.35, the YPZH-9 component was simultaneously dismantled in replaced areas 1 and 2, the stress in replaced area 1 and replaced area 2 decreased from the initial 5.4 MPa to 0, and the stress of the middle wall increased from the initial 5.4 MPa to 9.4 MPa. After removing the shear wall in the replaced area, the load borne by the replaced area was completely borne by the middle area that was not replaced; however, when the replaced area had just been refilled with high-strength concrete, it did not share the load immediately, and the stress in the non-replaced area and the replaced area remained unchanged at this time; the load was continuously applied until it was destroyed and the stress of the two parts was continuously redistributed. When the replacement had just been completed, the stress in the non-replaced area was greater than that in the replaced area. Later, because the replaced area was replaced by high-strength concrete, the stress in it gradually increased, and finally, the stress in the replaced area was greater than that in the non-replaced area. In addition, after the replacement of the shear wall, the stress difference between the non-replaced and replaced areas increased with the increase in the initial stress level and the area that had been replaced.

3.2.2. Two-Batch Replacement without a Supporting Roof

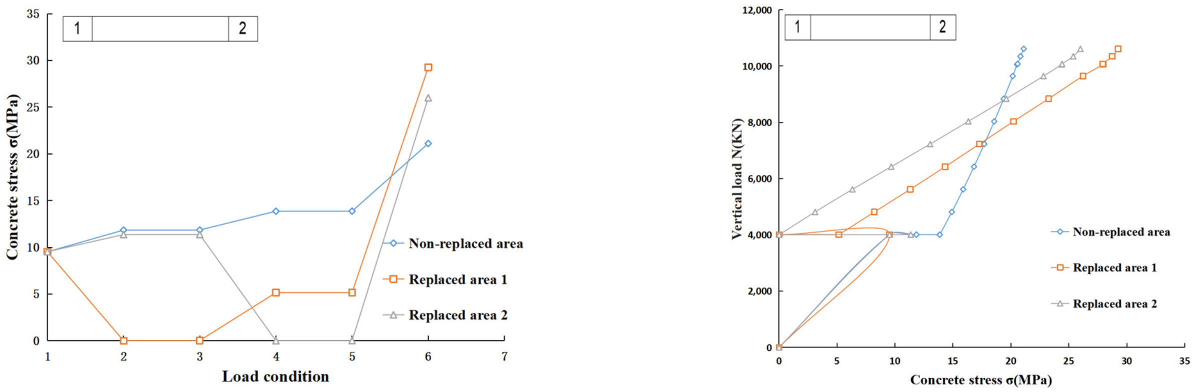
When using the method of one-batch replacement reinforcement without a supporting roof at high stress levels, the load in the replaced area will all be transferred to the non-replaced area; at this point, the stress difference between the non-replaced area and the replaced area is very large, which may lead to the crushing of the concrete in the non-replaced area. Therefore, at high stress levels, using a multi-batch replacement method can effectively reduce the stress difference between the various regions of the wall and improve the cooperative working ability of each region, thus enhancing the overall performance of the wall after reinforcement.

Two batches of experimental replacement groups were tested at high stress levels; the initial stress level was between 0.4 and 0.6. The two-batch replacement was divided into six working conditions. Working condition one represented the action of initial stress; working condition two represented the removal of the wall section of replaced area 1; working condition three represented the casting of the wall section of replaced area 1; working condition four represented the removal of the wall section of replaced area 2; working condition five represented the casting of the wall section of replaced area 2, and working condition 6 represented the action of the ultimate load. This study took two components, EPZH-10 and EPZH-18, as representatives for analysis; their working condition stress diagrams and load–stress curves are shown in Figures 8 and 9.



(**a**) (**b**)

Figure 8. (**a**) Stress diagram under different load conditions for EPZH-10; (**b**) load–stress curve for YPZH-10.



(**a**) (**b**)

Figure 9. (**a**) Stress diagram under different load conditions for EPZH-18; (**b**) load–stress curve for EPZH-18.

As shown in Figure 8, at the initial stress level of 0.4, the EPZH-10 component first demolished replaced area 1, and the stress in area 1 decreased to 0. The original 6.4 MPa stress in area 1 was borne by the middle area and area 2, and their stress increased from the initial 6.4 MPa to 7.7 MPa. After casting area 1, the stress of the middle area and area 2 had little change. In the same way, after demolishing replaced area 2, the stress in area 2 dropped to 0, and the load that was unloaded from area 2 was borne by the middle area and area 1; the stress in area 1 increased from 0 to 2.5 MPa, and the stress in the middle area increased from 7.7 MPa to 8.8 MPa. When the replacement had just been completed, the stresses in area 1, the middle area, and area 2 were 2.5 MPa, 8.8 MPa, and 0.014 MPa, respectively.

As shown in Figure 9, at the initial stress level of 0.6, the EPZH-18 component first demolished replaced area 1, and the stress in area 1 reduced to 0. The original 9.5 MPa stress in area 1 was borne by the middle area and area 2, and their stress increased from the initial 9.5 MPa to 11.4 MPa. After casting area 1, the stress of the middle area and area 2 had little change. Likewise, after demolishing replaced area 2, the stress in area 2 dropped to 0, and the load that was unloaded from area 2 was borne by the middle area and area 1; the stress in area 1 increased from 0 to 5.14 MPa, and the stress in the middle area increased from 11.4 MPa to 13.86 MPa. When the replacement had just been completed, the stresses in area 1, the middle area, and area 2 were 5.14 MPa, 13.86 MPa, and 0.017 MPa, respectively.

To sum up, when the replacement had just been completed, the stress of each wall segment was different; the stress of the shear wall in the non-replaced area was the greatest, while the stress of the shear wall in the replaced area was related to the replacement order; that is, the earlier the shear wall was replaced, the greater the stress was. Therefore, for the local replacement of the shear wall without a roof under a high stress level, the stress difference in each wall section could be reduced by increasing the number of replacement batches and optimizing the replacement sequence so as to improve the utilization rate of the concrete's strength.

3.2.3. Analysis of the Ultimate Bearing Capacity and Its Significance

The ultimate bearing capacity of each component, the stress in each region in the ultimate state, and the concrete strength utilization coefficient are shown in Table 3. According to the data in Table 3, the ultimate bearing capacity of the one-batch and two-batch local reinforcement replacements without a supporting roof was greater than the design value of 9185 KN for the bearing capacity of a shear wall, and the ultimate bearing capacity was increased by 8.8–69.8%; the ultimate bearing capacity of the profile steel reinforcement was increased by 50% on average.

Table 3. Finite element ultimate bearing capacity analysis.

Specimen	Simulation Value of the Ultimate Bearing Capacity in VFEAP (KN)	Stress in the Ultimate State (Mpa)			Concrete Strength Utilization Coefficient		
		Area 1	Non-Replaced Area	Area 2	Area 1	Non-Replaced Area	Area 2
YPZH-1	12,119	32.0	24.5	32.1	0.99	1.05	0.99
YPZH-2	14,302	40.0	24.6	39.4	0.96	1.05	0.95
YPZH-3	15,596	51.7	23.8	51.7	1.03	1.02	1.03
YPZH-4	13,556	32.6	26.3	32.8	1.01	1.12	1.01
YPZH-5	13,214	39.3	24.2	39.8	0.95	1.03	0.96
YPZH-6	14,097	46.1	24.1	46.8	0.92	1.03	0.93
YPZH-7	11,830	28.9	20.0	29.1	0.89	0.86	0.90
YPZH-8	11,561	42.8	21.4	42.7	1.03	0.91	1.03
YPZH-9	14,000	41.9	18.9	41.5	0.84	0.81	0.83
EPZH-10	11,335	28.2	23.0	27.8	0.87	0.98	0.86
EPZH-11	11,456	31.0	21.1	29.7	0.75	0.90	0.71
EPZH-12	12,620	35.9	21.4	34.4	0.71	0.91	0.69
EPZH-13	10,374	24.7	21.2	22.3	0.76	0.91	0.69
EPZH-14	10,860	28.2	21.6	27.3	0.68	0.92	0.66
EPZH-15	12,063	36.1	22.2	35.8	0.72	0.95	0.71
EPZH-16	9995	23.4	22.1	20.9	0.72	0.94	0.65
EPZH-17	11,383	27.6	22.7	26.6	0.67	0.97	0.64
EPZH-18	10,725	29.3	21.1	26.0	0.58	0.90	0.52

According to the current Code for the design of strengthened concrete structures, when the vertical RC components are strengthened using a replacement method and without a supporting roof, the strength utilization coefficient of the new concrete in the replaced area will be 0.8 when using the calculation formula for the bearing capacity of a normal section. However, as can be seen in Table 3, with the increase in the initial stress level, the strength utilization coefficient of concrete gradually decreased. When the initial stress level was less than 0.4, the strength utilization coefficient of the new concrete in the replaced area was greater than 0.8; however, when the initial stress level exceeded 0.4, the concrete strength utilization coefficient in the replaced area was mostly less than 0.8. In particular, in the subsequently replaced area, there was a greater difference between the concrete strength utilization coefficient and 0.8 with the increase in the initial stress level. To sum up, when the initial stress level was less than 0.4, it was conservative to take the strength utilization coefficient of the new concrete in the replaced area as 0.8 according to the Code; however, when the initial stress level was greater than 0.4, it was unsafe to take the value as 0.8 according to the Code.

Using the data in Table 3, an analysis of the results of the orthogonal experiment was conducted, as shown in Table 4. According to k_{ii} in Table 4, the curve of the relationship between different factors and the ultimate bearing capacity was drawn, as shown in Figures 10–12.

Table 4. Analysis of the results of the orthogonal experiment.

Factor		Initial Stress Level	Strength of the Replacement Concrete	The Size of Replaced Area
Orthogonal analysis of one-batch replacement	K1	42,017	37,505	37,777
	K2	40,867	39,077	40,640
	K3	37,391	43,693	41,858
	k_{11}	14,005.7	12,501.7	12,592.3
	k_{22}	13,622.3	13,025.7	13,546.7
	k_{33}	12,463.7	14,564.3	13,952.7
	Range (R)	1542	2062.6	1360.4
Orthogonal analysis of two-batch replacement	K1	35,411	31,704	34,781
	K2	33,297	33,699	33,475
	K3	32,103	35,408	32,555
	k_{11}	11,803.7	10,568	11,593.7
	k_{22}	11,099	11,233	11,158.3
	k_{33}	10,701	11,802.7	10,851.7
	Range (R)	1102.7	1234.7	742

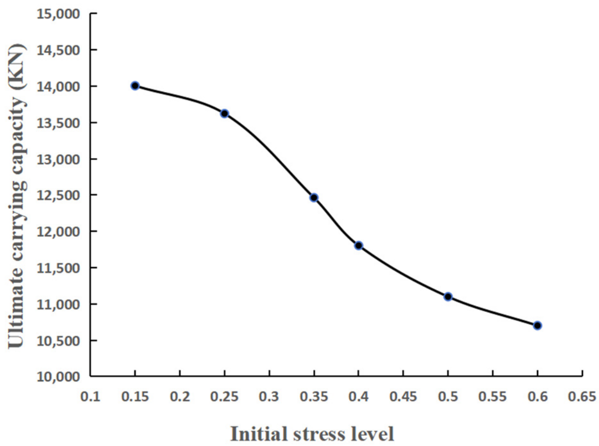
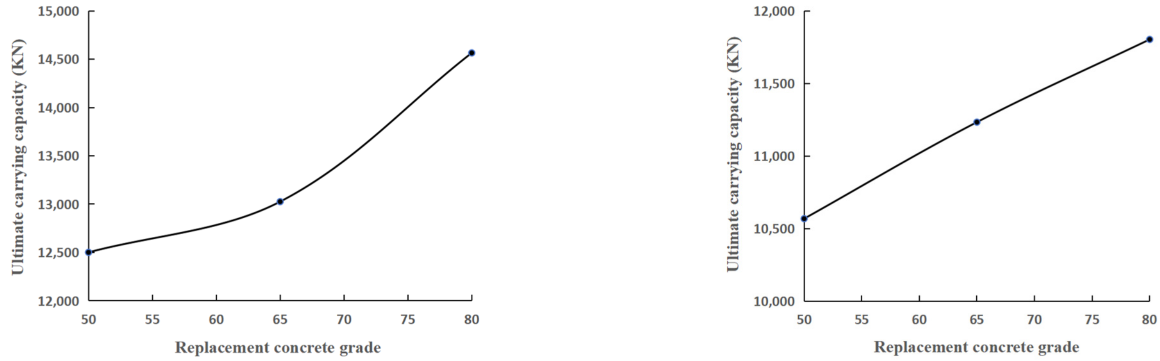


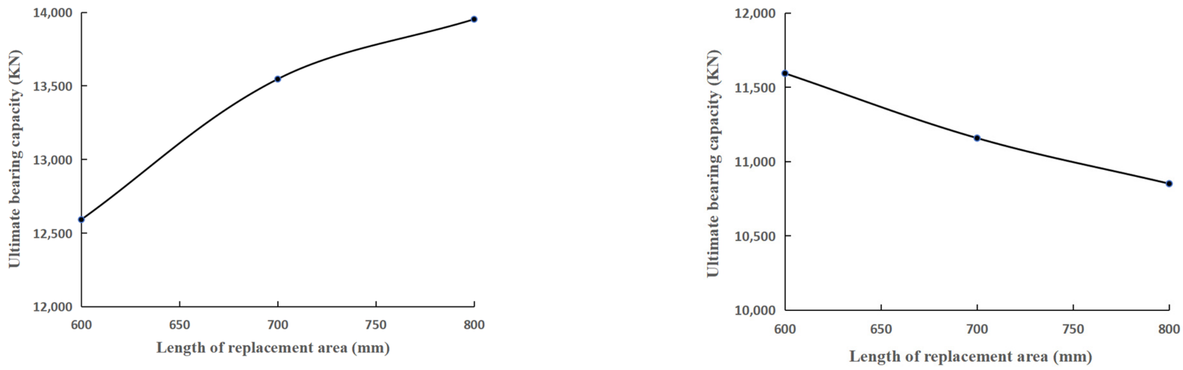
Figure 10. Relationship between the initial stress level and the ultimate bearing capacity.

As can be seen in Figure 10, the ultimate bearing capacity of the strengthened components decreased with the increase in the initial stress level. At a stress level of 0.15, the ultimate bearing capacity was 14,005 KN. At a stress level of 0.6, the ultimate bearing capacity was 10,701 KN, which was a reduction of 23.6%. It can be seen that the initial stress level had a significant impact on the ultimate bearing capacity of the strengthened components. The initial stress level represents the original stress state of the shear wall before replacement. The higher the initial stress level, the more cracks inside the shear wall develop, and the worse the reinforcement effect of the shear wall. Therefore, in order to obtain an ideal reinforcement effect, the replacement of the shear wall should be carried out at a low stress level.



(**a**) (**b**)

Figure 11. (**a**) For a stress level ranging from 0.15 to 0.35, the relationship between the concrete strength at displacement and the ultimate bearing capacity is shown; (**b**) for a stress level ranging from 0.4 to 0.6, the relationship between the concrete strength at displacement and the ultimate bearing capacity is shown.



(**a**) (**b**)

Figure 12. (**a**) For stress levels ranging from 0.15 to 0.35, the relationship between the displacement area and the ultimate bearing capacity is shown; (**b**) for stress levels ranging from 0.4 to 0.6, the relationship between the displacement area and the ultimate bearing capacity.

As can be seen in Figure 11, the ultimate bearing capacity of the strengthened components increased with the increase in the replacement concrete strength. At stress levels from 0.15 to 0.35, the replacement concrete strength of C50 had an ultimate bearing capacity of 12,501 KN, while the replacement concrete strength of C80 had an ultimate bearing capacity of 14,564 KN, which was an enhancement of 16.5%. At stress levels from 0.4 to 0.6, the replacement concrete strength of C50 had an ultimate bearing capacity of 10,568 KN, while the replacement concrete strength of C80 had an ultimate bearing capacity of 11,802 KN, which was an enhancement of 11.7%. It can be seen that the influence of the replacement concrete strength on the ultimate bearing capacity was more significant at low stress levels than at high stress levels.

As can be seen in Figure 12, at low stress levels, the ultimate bearing capacity of the strengthened components increased with the increase in the size of the replacement area; however, at high stress levels, the ultimate bearing capacity decreased with the increase in the size of the replacement area. At stress levels from 0.15 to 0.35, when the total length of the replaced wall on both sides was 600 mm, the ultimate bearing capacity was 12,592 KN; when the total length of the replaced wall on both sides was 800 mm, the ultimate bearing capacity was 13,952 KN, which was an enhancement of 10.8%. At stress levels from 0.4 to 0.6, when the total length of the replaced wall on both sides was 600 mm, the ultimate bearing capacity was 11,593 KN; when the total length of the replaced wall on both sides was 800 mm, the ultimate bearing capacity was 10,851 KN, which was a reduction of 6.4%. At low stress levels, the cracks and deformations in the shear wall are small, and the stress distribution in the shear wall is more uniform. At this time, the bearing capacity of the shear wall increases with the increase in the size of the replaced area. In contrast, at high stress levels, the shear wall may have developed obvious cracks and deformations, resulting in an uneven stress distribution in the shear wall, and cooperative work between the old and new materials becomes more difficult. At this time, the bearing capacity of the shear wall decreases with the increase in the size of the replacement area.

4. Calculation Formula for the Bearing Capacity of a Local Strengthening Replacement of an RC Shear Wall with an Unsupported Roof under Axial Compression

According to the simulation analysis in the previous section, at high stress levels, the strength utilization coefficient of the new concrete in the replaced area was lower than the value of 0.8 in the Code. Therefore, at high stress levels, the Code's value of the strength utilization coefficient of the new concrete in the replaced area is unsafe. In addition, the Code's formula does not take the stress hysteresis of the reinforced components into account but simply discounts the strength of the new concrete in the replaced area and then accumulates the bearing capacity of the new and old concrete. Based on these two points, this study comprehensively considered the influences of stress hysteresis, the initial stress level, and the strength of the replacement concrete to derive the calculation formulas for the bearing capacity of a normal section with the one-batch, two-batch, and three-batch replacements in order to perfect and supplement the formula in the Code.

In this study, the original column concrete adopts the constrained concrete constitutive model [16] considering the constraints of the stirrups in the reinforcement area. The model reflected the improvement of the concrete strength and ductility due to the stirrups in the original column by introducing the strength improvement coefficient K. The calculation formulas of the constrained concrete constitutive model are detailed in [16]. However, the unconstrained concrete constitutive model was adopted for the new concrete. The model did not consider the improvement of the concrete strength or ductility due to the stirrups. The calculation formulas of the unconstrained concrete constitutive model were similar to those of the constrained concrete constitutive model calculation; except, they did not take the strength improvement coefficient K into account. The constitutive curve is shown in Figure 13. The model for Figure 13 can be found in [16]. A bilinear constitutive model that described complete elastoplasticity was adopted for the steel bars, and the constitutive curve is shown in Figure 14.

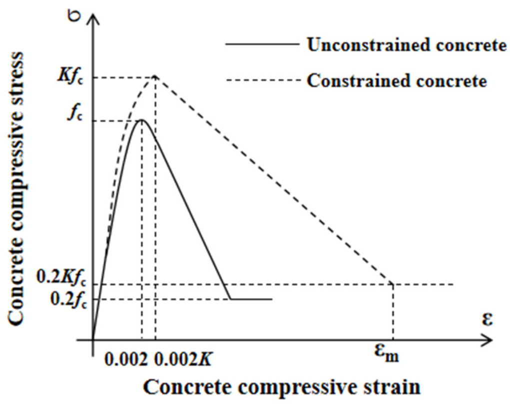


Figure 13. Kent–Park concrete model.

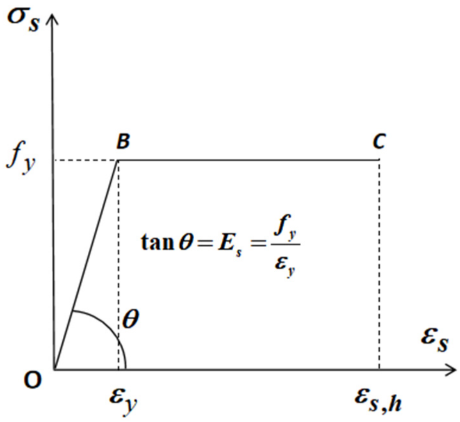


Figure 14. Fully plastic model.

4.1. Theoretical Analysis

In this section, the derivation of the formula for the ultimate bearing capacity of the three-batch RC shear walls under axial compression was taken as an example. (Due to the similar ideas regarding the derivation of the formulas, the derivation processes for the bearing capacities of the one-batch and two-batch replacements are not repeated, and the corresponding formulas will be given directly in the following section.) Finally, based on the formulas for the bearing capacities of the one-batch, two-batch, and three-batch replacements, a formula for the ultimate bearing capacity of the N-batch replacements was summarized.

4.1.1. Example of the Derivation of a Bearing Capacity Formula for a Locally Reinforced Replacement with an Unsupported Roof

The shear wall strengthened using a three-batch replacement was simplified as the theoretical model shown in Figure 15.

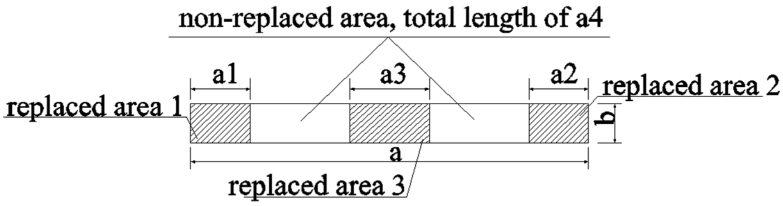


Figure 15. Theoretical modeling of the three-batch replacement reinforcement.

(1) The stress state analysis of the shear wall before replacement.

The axial pressure on the shear wall before the replacement is

$$N_1 = A_{c1}\sigma_c^1 + A_{s1}\sigma_s^1 \tag{1}$$

where σ_c^1 and σ_s^1 are, respectively, the compressive stress of the concrete and the longitudinal reinforcement of the shear wall before replacement; A_{c1} and A_{s1} are, respectively, are the cross-sectional areas of the concrete and the longitudinal reinforcement of the shear wall before replacement; and A_{c1} = a × b.

According to the constitutive relation of constrained concrete, we can obtain

$$\sigma_{c4}^1 = \sigma_c^1 = Kf_c\left[2\left(\frac{\varepsilon_c^1}{\varepsilon_{c0}}\right) - \left(\frac{\varepsilon_c^1}{\varepsilon_{c0}}\right)^2\right] \tag{2}$$

$$\varepsilon_c^1 = \left(1 - \sqrt{1 - \frac{1}{K}\sigma_c^1/f_{c0}}\right)\varepsilon_{c0} = \left(1 - \sqrt{1 - \frac{1}{K}\beta}\right)\varepsilon_{c0} \tag{3}$$

where σ_{c4}^{1} is the concrete stress of the non-replaced area at the first stage, the letter in the subscript represents the concrete and the number represents the area, such as "c2" is the concrete in the replaced area 2; the superscript represents the stage reached, such as "2" means it is in the second stage; and the descriptions of stress and strain in the following text is the same. ε_{c}^{1} is the concrete strain in the shear wall at the first stage. β is the initial stress level of the concrete in the shear wall, whereby $\beta = \sigma_{c}^{1}/f_{c0}$. f_{c0} is the compressive strength of the concrete before the replacement, which is the same later. K is the concrete strength improvement coefficient considering the effect of constraints due to the stirrups in the reinforcement area, $K = 1 + \frac{\rho_{S1} f_{yh}}{f_c}$, and it is the same later. ρ_{s1} and f_{yh} are the stirrup ratio and stirrup yield strength in the reinforcement area; $\varepsilon_{c0} = 0.002K$.

(2) The stress state analysis after removing and re-casting replaced area 1.

After removing replaced area 1, the stress unloaded from it is shared by replaced area 2, replaced area 3, and the non-replaced area; at this time, the stress and strain of replaced area 1 are both zero, and the stress and strain of the other areas are as follows:

$$\sigma_{c2}^{2} = \sigma_{c3}^{2} = \sigma_{c4}^{2} = \sigma_{c}^{1} + \sigma_{c}^{1}\frac{a_1}{a - a_1} = \frac{a}{a - a_1}\sigma_{c}^{1} \tag{4}$$

$$\varepsilon_{c2}^{2} = \varepsilon_{c3}^{2} = \varepsilon_{c4}^{2} = \left(1 - \sqrt{1 - \frac{a}{a - a_1}\frac{1}{K}\beta}\right)\varepsilon_{c0} \tag{5}$$

(3) The stress state analysis after removing and re-casting replaced area 2.

Similarly, after removing replaced area 2, the stress unloaded from it is shared by replaced area 3, the non-replaced area, and the newly added replaced area 1; at this time, the stress and strain of replaced area 2 are both zero, and the stress and strain of the other areas are as follows:

$$\sigma_{c1}^{3} = \sigma_{c2}^{2}\frac{a_2}{a - a_2} = \frac{a \times a_2}{(a - a_1) \times (a - a_2)}\sigma_{c}^{1} \tag{6}$$

$$\sigma_{c3}^{3} = \sigma_{c4}^{3} = \sigma_{c4}^{2} + \sigma_{c2}^{2}\frac{a_2}{a - a_2} = \frac{a^2}{(a - a_1) \times (a - a_2)}\sigma_{c}^{1} \tag{7}$$

$$\varepsilon_{c1}^{3} = \left(1 - \sqrt{1 - \frac{a \times a_2}{(a - a_1) \times (a - a_2)}\frac{1}{K}\beta}\right)\varepsilon_{c0} \tag{8}$$

$$\varepsilon_{c3}^{3} = \varepsilon_{c4}^{3} = \left(1 - \sqrt{1 - \frac{a^2}{(a - a_1) \times (a - a_2)}\frac{1}{K}\beta}\right)\varepsilon_{c0} \tag{9}$$

(4) The stress state analysis after removing and re-casting replaced area 3.

Similarly, after removing replaced area 3, the stress unloaded from it is shared by the non-replaced area and the newly added replaced area 1 and replaced area 2; at this time, the stress and strain of replaced area 3 are both zero, and the stress and strain of the other areas are as follows:

$$\sigma_{c1}^{4} = \sigma_{c1}^{3} + \sigma_{c3}^{3}\frac{a_3}{a - a_3} = \frac{a^2 \times (a_2 + a_3) - a \times a_2 \times a_3}{(a - a_1) \times (a - a_2) \times (a - a_3)}\sigma_{c}^{1} \tag{10}$$

$$\sigma_{c2}^{4} = \sigma_{c3}^{3}\frac{a_3}{a - a_3} = \frac{a^2 \times a_3}{(a - a_1) \times (a - a_2) \times (a - a_3)}\sigma_{c}^{1} \tag{11}$$

$$\sigma_{c4}^{4} = \sigma_{c4}^{3} + \sigma_{c3}^{3}\frac{a_3}{a - a_3} = \frac{a^3}{(a - a_1) \times (a - a_2) \times (a - a_3)}\sigma_{c}^{1} \tag{12}$$

$$\varepsilon_{c1}^{4} = \left(1 - \sqrt{1 - \frac{a^2 \times (a_2 + a_3) - a \times a_2 \times a_3}{(a - a_1) \times (a - a_2) \times (a - a_3)}\frac{1}{K}\beta}\right)\varepsilon_{c0} \tag{13}$$

$$\varepsilon_{c2}^{4} = \left(1 - \sqrt{1 - \frac{\mathrm{a}^2 \times \mathrm{a}_3}{(\mathrm{a}-\mathrm{a}_1)\times(\mathrm{a}-\mathrm{a}_2)\times(\mathrm{a}-\mathrm{a}_3)}\frac{1}{K}\beta}\right)\varepsilon_{c0} \tag{14}$$

$$\varepsilon_{c4}^{4} = \left(1 - \sqrt{1 - \frac{\mathrm{a}^3}{(\mathrm{a}-\mathrm{a}_1)\times(\mathrm{a}-\mathrm{a}_2)\times(\mathrm{a}-\mathrm{a}_3)}\frac{1}{K}\beta}\right)\varepsilon_{c0} \tag{15}$$

(5) The stress state analysis of the shear wall after damage.

In the ultimate state, when the concrete in the non-replaced area reaches a peak compressive strain, the increase in the strain of the non-replaced area is

$$\Delta\varepsilon = \varepsilon_{c0} - \varepsilon_{c4}^{4} = \varepsilon_{c0}\sqrt{1 - \frac{\mathrm{a}^3}{(\mathrm{a}-\mathrm{a}_1)\times(\mathrm{a}-\mathrm{a}_2)\times(\mathrm{a}-\mathrm{a}_3)}\frac{1}{K}\beta} \tag{16}$$

According to the assumption of a planar cross-section, after the replacement of replaced areas 1, 2, and 3 is completed, the strain produced by the concrete in the replaced areas and the non-replaced area is equal under the ultimate load. Therefore,

$$\varepsilon_{c1}^{5} = \Delta\varepsilon + \varepsilon_{c1}^{4},\ \varepsilon_{c2}^{5} = \Delta\varepsilon + \varepsilon_{c2}^{4},\ \varepsilon_{c3}^{5} = \Delta\varepsilon \tag{17}$$

After substituting the strain of replaced areas 1, 2, and 3 in the ultimate state into the constitutive relation of unconstrained concrete, the compressive stress of newly added concrete in the ultimate state of the bearing capacity can be obtained as follows:

$$\sigma_1^5 = f_c\left[2\left(\frac{\varepsilon_{c1}^5}{\varepsilon_0}\right) - \left(\frac{\varepsilon_{c1}^5}{\varepsilon_0}\right)^2\right] \tag{18}$$

$$\sigma_2^5 = f_c\left[2\left(\frac{\varepsilon_{c2}^5}{\varepsilon_0}\right) - \left(\frac{\varepsilon_{c2}^5}{\varepsilon_0}\right)^2\right] \tag{19}$$

$$\sigma_3^5 = f_c\left[2\left(\frac{\varepsilon_{c3}^5}{\varepsilon_0}\right) - \left(\frac{\varepsilon_{c3}^5}{\varepsilon_0}\right)^2\right] \tag{20}$$

here $\varepsilon_0 = 0.002$. The modified concrete strength utilization coefficients in replaced areas 1, 2, and 3 are defined as $\alpha_{c1} = \sigma_1^5/f_c$, $\alpha_{c2} = \sigma_2^5/f_c$, and $\alpha_{c3} = \sigma_3^5/f_c$, respectively. Then, we can obtain the modified strength utilization coefficient of the newly added concrete α_{c1}, α_{c2}, and α_{c3} using the following:

$$\alpha_{c1} = 2K\sqrt{\left(1-\frac{C_1}{K}\beta\right)\left(1-\frac{C_3}{K}\beta\right)} + K(C_1+C_3)\beta - K^2 \tag{21}$$

$$\alpha_{c2} = 2K\sqrt{\left(1-\frac{C_2}{K}\beta\right)\left(1-\frac{C_3}{K}\beta\right)} + K(C_2+C_3)\beta - K^2 \tag{22}$$

$$\alpha_{c3} = 2K\sqrt{1-\frac{C_3}{K}\beta} + KC_3\beta - K^2 \tag{23}$$

To sum up, for the shear wall under axial compression, the ultimate bearing capacity formula for the three-batch local strengthening replacement method with an unsupported roof is as follows:

$$N_u \leq \varphi\left[f_{c0}A_{c0} + \alpha_{c1}f_cA_{c1} + \alpha_{c2}f_cA_{c2} + \alpha_{c3}f_cA_{c3} + f'_{y0}A'_{s0}\right] \tag{24}$$

where C_1, C_2, and C_3 are the ratios of the length of the shear wall section, $C_1 = \frac{\mathrm{a}^2\times(\mathrm{a}_2+\mathrm{a}_3)-\mathrm{a}\times\mathrm{a}_2\times\mathrm{a}_3}{(\mathrm{a}-\mathrm{a}_1)\times(\mathrm{a}-\mathrm{a}_2)\times(\mathrm{a}-\mathrm{a}_3)}$, $C_2 = \frac{\mathrm{a}^2\times\mathrm{a}_3}{(\mathrm{a}-\mathrm{a}_1)\times(\mathrm{a}-\mathrm{a}_2)\times(\mathrm{a}-\mathrm{a}_3)}$, and $C_3 = \frac{\mathrm{a}^3}{(\mathrm{a}-\mathrm{a}_1)\times(\mathrm{a}-\mathrm{a}_2)\times(\mathrm{a}-\mathrm{a}_3)}$, where a is the total length of the shear wall, and a_1, a_2, and a_3 are, respectively, the lengths of replaced areas 1, 2, and 3; φ is the stability factor for the compression members.

For the shear wall under axial compression, the ultimate bearing capacity formula for the two-batch local strengthening replacement method with an unsupported roof is as follows:

$$N_u \leq \varphi\left[f_{c0}A_{c0} + \alpha_{c1}f_cA_{c1} + \alpha_{c2}f_cA_{c2} + f'_{y0}A'_{s0}\right] \tag{25}$$

where α_{c1} is the modified strength utilization coefficient of the newly added concrete in replaced area 1, $\alpha_{c1} = 2K\sqrt{\left(1 - \frac{C_1}{K}\beta\right)\left(1 - \frac{C_2}{K}\beta\right)} + K(C_1 + C_2)\beta - K^2$, $0 < \alpha_{c1} \leq 1$; α_{c2} is the modified strength utilization coefficient of the newly added concrete in replaced area 2, $\alpha_{c2} = 2K\sqrt{1 - \frac{C_2}{K}\beta} + KC_2\beta - K^2$, $0 < \alpha_{c2} \leq 1$; C_1, C_2 are the ratios of the length of the shear wall section, $C_1 = \frac{a \times a_2}{(a-a_1)\times(a-a_2)}$, $C_2 = \frac{a^2}{(a-a_1)\times(a-a_2)}$, where a is the total length of the shear wall, a_1 is the length of replaced area 1, and a_2 is the length of replaced area 2.

For the shear wall under axial compression, the ultimate bearing capacity formula for the one-batch local strengthening replacement method with an unsupported roof is as follows:

$$N_u \leq \varphi\left[f_{c0}A_{c0} + \alpha_{c1}f_cA_c + f'_{y0}A'_{s0}\right] \tag{26}$$

where α_{c1} is the modified strength utilization coefficient of the newly added concrete in the replaced area, $\alpha_{c1} = 2K\sqrt{1 - \frac{C}{K}\beta} + CK\beta - K^2$; C is the ratio of the length of the shear wall section, $C = a/a_3$, where a_3 is the length of the non-replaced area, and a is the total length of the shear wall.

4.1.2. Regression Analysis of the Formula for Bearing Capacity of N-Batch Replacements

Based on the bearing capacity formulas for the one-batch, two-batch, and three-batch locally reinforced replacements mentioned above, the ultimate bearing capacity formula for the N-batch locally reinforced replacements with an unsupported roof can be summarized as follows:

$$N_u \leq \varphi\left[f_{c0}A_{c0} + \alpha_{c1}f_cA_{c1} + \alpha_{c2}f_cA_{c2} + \ldots\ldots + \alpha_{cn}f_cA_{cn} + f'_{y0}A'_{s0}\right] \tag{27}$$

where α_{c1} is the modified strength utilization coefficient of the newly added concrete in replaced area 1, $\alpha_{c1} = 2K\sqrt{(1 - \frac{C_1}{K}\beta)(1 - \frac{C_n}{K}\beta)} + K(C_1 + C_n)\beta - K^2, 0 < \alpha_{c1} \leq 1$; $\alpha_{c(n-1)}$ is the modified strength utilization coefficient of the newly added concrete in replaced area (N−1), $\alpha_{c(n-1)} = 2K\sqrt{(1 - \frac{C_{(n-1)}}{K}\beta)(1 - \frac{C_n}{K}\beta)} + K(C_{(n-1)} + C_n)\beta - K^2, 0 < \alpha_{c(n-1)} \leq 1$; α_{cn} is the modified strength utilization coefficient of the newly added concrete in replaced area N, $\alpha_{cn} = 2K\sqrt{1 - \frac{C_n}{K}\beta} + KC_n\beta - K^2$, $0 < \alpha_{cn} \leq 1$; and C_n is the ratio of the length of the wall section in replaced area N, $C_n = \frac{a^n}{(a-a_1)(a-a_2)\cdots(a-a_n)}$.

For Equation (27) to have a reliable probability, a regression analysis was used to establish a practical calculation formula for a 95% guarantee. At the same time, considering randomness, the ultimate bearing capacity of the RC components strengthened using the local strengthening replacement method with an unsupported roof was regarded as a random variable. In order to clarify its probability distribution, the distribution law was verified with a probability density function in combination with mathematical statistics software. The results showed that the bearing capacity of the strengthened components conformed to the normal distribution law, so the following model was adopted:

$$\hat{N} = \sigma N$$

According to the results of the nonlinear analysis in the third section, the numerical values of random variables corresponding to the bearing capacity of each shear wall were obtained by using the above formula. Figure 16 shows a statistical histogram of the random variables, and it can be intuitively seen that they approximately obey the normal distribution. According to the mathematical analysis, the average value is 1.03, and the standard deviation is 0.03.

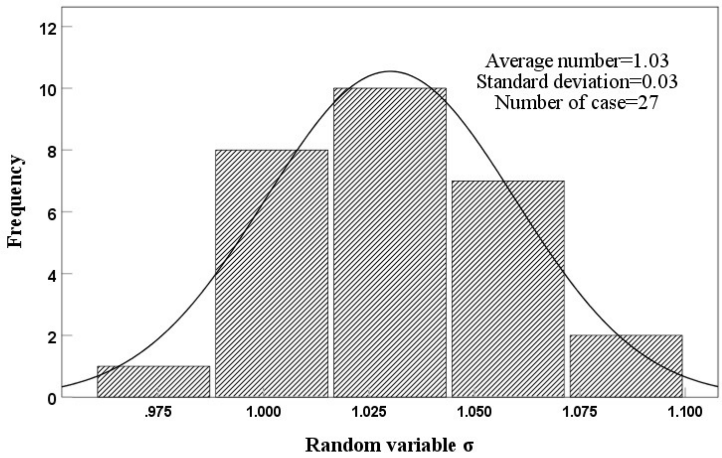


Figure 16. Histogram of the random variable.

Using the K–S method, the probability distribution of the hypothetical random variables was tested for goodness of fit. The sample size n was 27, the maximum observed $D_{27} = 0.03 < D_{27}^{0.05} = 0.26$ indicated that the test was accepted, and the random variable followed a normal distribution, with an average value of 1.03 and a standard deviation of 0.03. Therefore, the final formula for strengthening the bearing capacity of the RC members using the local strengthening replacement method without a supporting roof is

$$\hat{N} = (1.03 - 0.03 \times 4.325)\ N = 0.9N$$

Finally, after finishing, the bearing capacity formula for the RC members strengthened using the unsupported local strengthening replacement method is as follows:

$$N_u \leq 0.9\varphi\left[f_{c0}A_{c0} + \alpha_{c1}f_cA_{c1} + \alpha_{c2}f_cA_{c2} + \ldots\ldots + \alpha_{cn}f_cA_{cn} + f'_{y0}A'_{s0}\right] \tag{28}$$

4.2. Formula Reliability Test

According to the results of the simulation analysis and the collected test data, the ratios of the values calculated using the formula in this study to the values calculated using the formula in the Code were obtained by substituting the parameters of each specimen into each formula, as shown in Table 5. According to the data in Table 5, a comparison between this formula and the Code's formula could be drawn, as shown in Figures 17 and 18. As can be seen in the figures, the values calculated with this formula were all less than the experimental simulation values. When compared with the Code's formula, the formula in this study was more consistent with the experimental simulation results, and the dispersion is smaller. To sum up, the formula in this study is more reliable than the Code's formula.

Table 5. Correlated experimental data.

Data Source	Specimen	Finite Element Simulation Value N_1 (KN)	The Calculated Value N_2 of the Formula in this Paper (KN)	Ratio of N_1 to N_2	The Calculated Value N_3 of the Code's Formula (KN)	Ratio of N_1 to N_3
VFEAP simulation in this study	YPZH-1	12,119	10,980	1.104	10,441	1.161
	YPZH-2	14,302	13,065	1.095	10,758	1.329
	YPZH-3	15,596	13,615	1.146	11,754	1.327
	YPZH-4	13,556	12,170	1.114	9578	1.415
	YPZH-5	13,214	12,190	1.084	10,846	1.218
	YPZH-6	14,097	13,263	1.063	12,529	1.125
	YPZH-7	11,830	11,121	1.064	10,353	1.143
	YPZH-8	11,561	11,552	1.001	11,387	1.015
	YPZH-9	14,000	13,184	1.062	11,213	1.249
	EPZH-10	11,335	10,660	1.063	10,441	1.086
	EPZH-11	11,456	11,353	1.009	10,758	1.065
	EPZH-12	12,620	11,978	1.054	11,754	1.074
	EPZH-13	10,374	10,218	1.015	9578	1.083
	EPZH-14	10,860	10,699	1.015	10,876	0.999
	EPZH-15	12,063	11,869	1.016	12,529	0.963
	EPZH-16	9995	9886	1.011	10,353	0.965
	EPZH-17	11,383	10,657	1.068	11,397	0.999
	EPZH-18	10,725	10,289	1.042	11,213	0.956
Yao Zexin [17]	ZH-1	9065	5922	1.531	5550	1.633
	ZH-2	9647	8348	1.156	7734	1.247
	ZH-3	10,055	10,010	1.004	9903	1.015
	ZH-4	8664	7471	1.160	6924	1.251
	ZH-5	8988	8483	1.060	8029	1.119
	ZH-6	9459	8522	1.110	8212	1.152
	ZH-7	8481	7399	1.146	7224	1.174
	ZH-8	8861	7125	1.244	7123	1.244
	ZH-9	9319	8914	1.045	8812	1.058

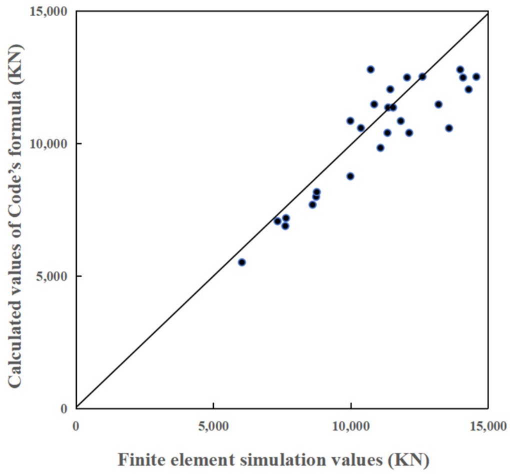


Figure 17. Comparison of the values calculated using the Code's formulas with the experimental simulation values.

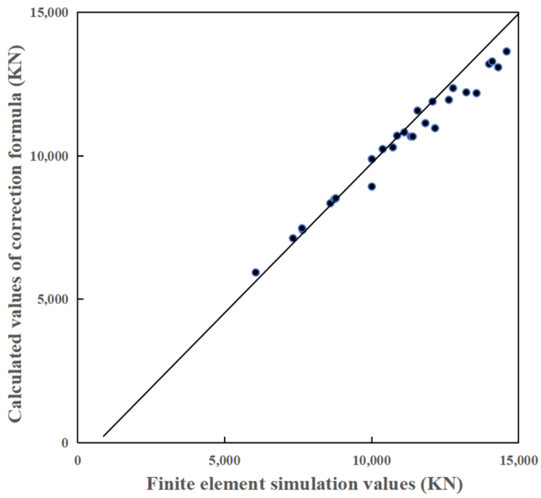


Figure 18. Comparison of the values calculated using the formulas in this study with the experimental simulation values.

5. Conclusions

(1) The three-dimensional solid degraded virtual laminated nonlinear finite element program VFEAP can simulate the actual failure processes and deformation of structures well. The simulated results were in good agreement with the test results, which indicated that the program is suitable for simulating the bearing performance of the shear walls after replacement.

(2) After shear wall replacement, the stress difference between the non-replaced area and replaced area increased with an increase in the initial stress level and the size of the replaced area; the larger the stress difference was, the more pronounced the phenomenon of stress hysteresis became. When the replacement had just been completed, the stress of the shear wall was not the same; the stress of shear wall in the non-replaced area was the greatest, while the stress of the shear wall in the replaced area was related to the replacement order, that is, the earlier the shear wall was replaced, the greater the stress was.

(3) The replacement concrete strength, the initial stress level, and the size of the replaced area had a significant influence on the bearing capacity of the shear wall after the replacement. The ultimate bearing capacity increased with an increase in the replacement concrete strength; it decreased with an increase in the initial stress level. At low stress levels, it increased with an increase in the size of the replaced area, while at high stress levels, it decreased with an increase in the size of the replaced area.

(4) Considering the influence of the replacement concrete strength, the initial stress level, and the size of the replaced area, then introducing a strength improvement coefficient considering the constraints of the stirrups, the modified strength utilization coefficient of the new concrete in the replaced area was established. Finally, a bearing capacity formula for a shear wall reinforced using a multi-batch local reinforcement replacement method was derived, and it had a higher degree of coincidence with the test simulation results, as well as a smaller degree of dispersion, and a higher degree of reliability than the Code's formula.

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