ORIGINAL ARTICLE

Latin American Journal of Solids and Structures

www.lajss.org

Collapse Resistance Model and Deformation Mechanism of Shear Wall Replacement

Lei Qi [a*], Xuansheng Cheng [b*], Shanglong Zhang [c]

[a] Gansu Province Gully Fixing and Tableland Protection Engineering Research Center, Longdong University, Qingyang 745000, China; Western China Technical Centre of Seismic Dissipation and Isolation, Lanzhou University of Technology, Lanzhou 730050, China. E-mail: qilei0314@sina.com

[b] Western Engineering Research Center of Disaster Mitigation in Civil Engineering of Ministry of Education, Lanzhou University of Technology, Lanzhou, 730050, PR China. E-mail: chengxslut@sina.com

[c] Key Laboratory of Disaster Prevention and Mitigation in Civil Engineering of Gansu Province, Lanzhou University of Technology, Lanzhou, 730050, PR China. E-mail: 347363807@qq.com

* Corresponding author

https://doi.org/10.1590/1679-78257189

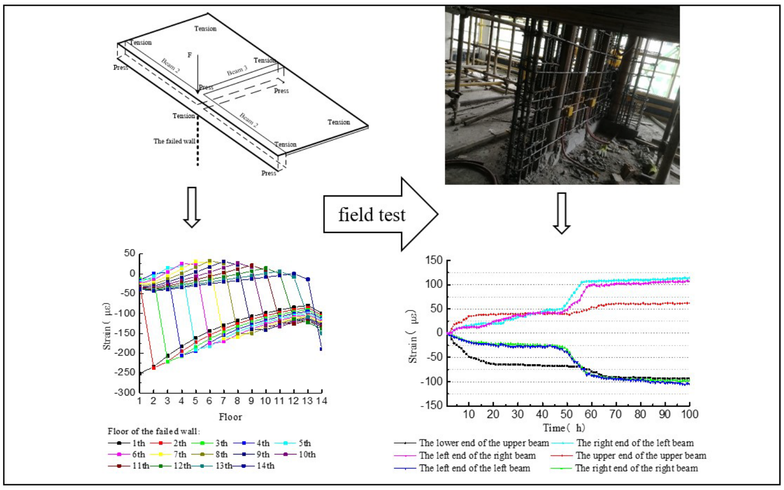
Abstract

Structural elements can be damaged or defective due to man-made and natural disasters, which will result in structural resistance below the design value. Component replacement can effectively improve the safety of the structure and does not affect its later use. However, this construction process must dismantle defective components. If the process is not properly operated, it will cause progressive collapse of the structure. There are few studies on progressive collapse resistance based on field tests. In this paper, the alternate load path method is used to study the deformation mechanism of reinforced concrete shear wall structures when partial members fail. The maximum vertical displacement and strain of the beam end near the failure walls are obtained, and load transfer laws are analyzed. Based on the component replacement construction of a high-rise shear wall structure, the progressive collapse resistance mechanism of the shear wall structure is verified, which provides theoretical guidance for component replacement construction.

Keywords

shear wall structure, component replacement, alternate analysis, beam mechanism, resistance of structure

Graphical Abstract



Received: July 17, 2022. In revised form: September 29, 2022. Accepted: October 07, 2022. Available online: October 11, 2022.
https://doi.org/10.1590/1679-78257189

1 INTRODUCTION

Reinforced concrete (RC) structures are often damaged by man-made and natural factors such as chloride erosion, freeze–thaw effects, carbonization, construction quality problems, earthquakes, fires, explosions, etc. (as shown in Figure 1). Furthermore, these defects affect the safety, applicability and durability of the structure and endanger the safety of people's lives and property. To improve the disaster resistance performance of the structure, it is necessary to strengthen those defective components. The component replacement method is a common method for structure strengthening; it must dismantle defective components and recast those components. This method is widely used in frame structures, masonry structures and shear wall structure strengthening. Studying the shear wall displacement mechanism, stress–strain behavior, and load transfer rule of relic structures has great significance for the safety of shear wall replacement. In addition, the study has important guiding significance for the shear wall structure collapse resistance study.

(a) Freeze-thaw damage

(b) Insufficient strength of concrete

(c) Concrete spalling

(d) Bottom wall damage

Figure 1 Instance of shear wall failure

RC frame structures are prone to progressive collapse when partial principal bearing elements are damaged. The progressive collapse resistance mechanisms of RC frame structures include beam mechanisms, arch compression mechanisms and catenary mechanisms (Li et al., 2011; Zhao et al., 2016). Mohammad et al. (2019) studied the relationship between the robustness and collapse resistance of structures and proposed several graphs to estimate the response of RC frame structures under progressive collapse. Structural robustness decreases with the increase in number of floors of the failed component (Mohsen and Mohammad, 2017). Chen et al. (2021) conducted a collapse resistance uncertainty analysis of RC frame structures and showed that the catenary mechanism was very important to the collapse resistance performance of RC frame structures. Studies have shown that progressive collapse resistance was closely related to the torsional irregularity (Hamed et al., 2019), failure paths, beam-column joints (Yu and Tan, 2017), etc.

Parisi et al. (2019) introduced five performance limit states associated with increasing levels of damage and quantified the corresponding load capacity under varying capacity model properties. Ghannoum (2007) conducted a progressive collapse model test on an RC frame structure, which does not satisfy the seismic requirements, and studied the performance of the structure when partial load-bearing columns fail. The floorslab membrane effect can increase the vertical progressive collapse capability in the catenary period. Transverse beams and floorslabs of RC frame structures can reduce the collapse vulnerability, and infilled walls improve the collapse resistance of RC frame structures (Ren et al., 2015; Qian et al., 2020). Yagob and Galal (2009) noted that although more research has been conducted on the progressive collapse phenomenon, it is necessary to develop a new methodology for structural design. Abbasnia et al. (2016) proposed a new method for progressive collapse; this method expressed the ultimate arching and catenary capacities of beams and can calculate the vertical displacement. The tension film effect of the plate can resist progressive collapse (Ma et al., 2020; Jiang et al., 2021). Yu et al. (2019) studied the influence of infilled walls on the progressive collapse resistance of structures. Zhang et al. (2020) studied the collapse resistance of an assembled monolithic beam-column structure and found that a prefabricated concrete structure had a lower progressive collapse resistance than a cast-in-place concrete structure.

In summary, many scholars have performed detailed studies on the collapse mechanism and structural robustness of RC frame structures and noted that the load path significantly influences the structural robustness, which affects the structural failure paths and progressive collapse performance. In addition, studies have shown that the ratio of longitudinal reinforcement and tensile deformation significantly impact the ultimate compressive capacity of reinforced concrete shear walls under earthquakes (Chrysanidis, 2019 and 2021). The ground motion intensity, longitudinal reinforcement ratio, floor weight, wall-to-floor area ratio and floor number have important effects on the wall thickness (Chai and Kunnath, 2005). However, only few studies have considered the influence of floorslabs on the collapse resistance of the structure; most studies are devoted to two-dimensional frame theory and experimental research, and those theories are rarely verified by actual engineering. In the existing structural system, floorslabs are integral parts of the vertical load-bearing component, and the contribution of floorslabs to the structural bearing capacity cannot be ignored. Considering the effects of floorslabs, the actual collapse reactance of a structure with partial shear walls failing has been analyzed, which is beneficial to the research and application of structural component replacement construction. In this paper, the alternate load path method is adopted to analyze the cause of the beam mechanism, stresses, strains and load transfer path of the shear wall structure when partial components fail. Combined with the field monitoring results, the strain responses by the finite element method (FEM) are verified.

2 SHEAR WALL REPLACEMENT MECHANISM

Component replacement construction must dismantle defective components during the construction process. Incorrect manipulation will cause vertical collapse of the structure, which threatens the safety of people's lives and their property. It is of great significance to improve construction safety and reduce secondary damage to the structure by reasonably designing replacement measures. Because a high-rise structure means a large vertical load, a safer construction scheme should be made through theoretical analysis before the construction process. Based on theoretical analysis and combined with engineering practices, the shear wall replacement mechanism of a high-rise structure is adopted as follows:

1) Location and quantity of defective components. The position and quantity of the components to be replaced are ascertained, and the collapse resistance of the entire structure is evaluated to prevent collapse during construction.
2) Analysis of the internal force. Using an internal force analysis, the loads borne by those defective components, mechanical behavior of the substructure and collapse resistance mechanism of the structure are acquired.
3) Support layout scheme. According to the location and loads of those defective components, a support layout scheme should be made. For small section members, such as frame columns, it is necessary to adopt the construction method of support beams to replace columns and lay support for adjacent beams of the defective components. When the components bear a large load or the upper floor has heavy equipment, it is also necessary to set full space support. Large cross-section members such as large cross-section columns and shear walls can use the piecewise replacement method. The load capacity of each member should be analyzed, and the supports should be arranged in time. Beam replacement usually supports the slabs to replace the beams. Loads of the upper structure are directly transferred to the lower structure by full support.
4) Unloading. Loads of the defective components are obtained by calculating the support layout. The reaction force is applied to the superstructure through support jacking. The reaction force is generally half of the load of the

component (for the more important structures, it must be verified by finite element analysis), which reduces the vertical loads borne by failed components.

5) Monitoring system setting. A vertical displacement monitoring system is set up to monitor the displacement and deformation of the structure in the process of component replacement. A strain monitoring system is set up for adjacent beams and floors to monitor the structural deformation, which ensures the safety of structural component replacement construction.
6) Dismantle failed components. It is not suitable to use the blasting method, which will cause secondary damage to the structure, a sudden release of structural stress, and consequentially structural cracking. Moreover, mechanical vibration easily causes structural cracking. Therefore, the static breaking method should be adopted to dismantle failed components, and the dismantle time of components should be appropriately extended to avoid sudden unloading, which results in a sudden release of structural load, structural rebound and cracking of beams and slabs. A hydraulic tong is commonly used for this construction.
7) Permanent support. For vertical load-bearing components such as walls and columns, permanent vertical supports should be arranged in time after the failed components are dismantled. The support generally consists of a concrete-filled steel tube, and shear connectors should be installed on the periphery of the steel tube to increase the bond with the concrete. When the support is laid out, loose concrete on the connection surface shall be removed, and the gaps between upper and lower connection surfaces shall be filled with high-strength grouting materials and secured by counterforce bolts.
8) Reinforcement bar binding. The damaged reinforcement bar shall be removed, and reinforcement bars shall be rebound and properly connected according to design specifications and drawings.
9) Mold casting. Casting should be checked and accepted by the supervisor before it is performed. Pouring concrete has a higher designed strength than the original concrete, and the upper and lower connection surfaces should be cleaned and precoated with high-strength grouting material to ensure tight connection and integrity of the structure.
10) Follow-up. When curing is completed, formworks should be removed, and the strength and casting quality of the replaced components should be tested.

3 COLLAPSE RESISTANCE MODEL OF SHEAR WALL STRUCTURE

At the initial stage of vertical bearing component failure, the relic structure presents the collapse resistance mode of the beam mechanism first. The beam mechanism is that the bending resistance of the substructure is fully developed when the stiffness of adjacent beam joints is large and the shear resistance is good after the vertical load-bearing components have been removed. In the beam mechanism stage, the rotation angle θ and deflection ω at adjacent beam ends are smaller in the initial loading stage, and the resistance R and deflection ω of the substructure present a linear elastic relationship.

$$R = k_e \omega \tag{1}$$

$$k_e = 24EI_Z/L^3 \tag{2}$$

where, L is the span of a single-span beam and EI_Z is the bending stiffness of the beam section.

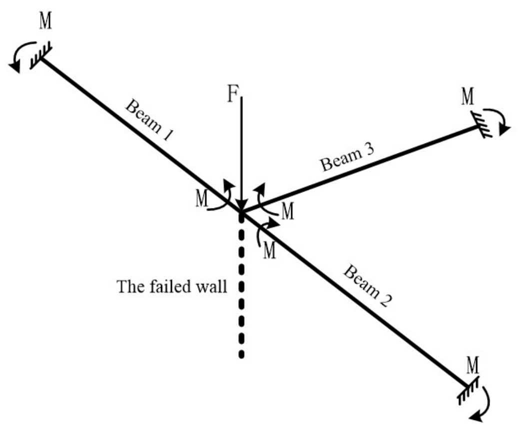


Figure 2 Stress mechanism of the structure when the edge wall is failed

The section strain at the end of the adjacent beam of the failed component is

$$\varepsilon = \frac{M}{EI_Z} y \tag{3}$$

where, I_Z is the inertia moment of the section, M is the bending moment on the section, y is the distance from the stress point to the central axis, and E is the elastic modulus of concrete.

For a single-layer structure, as shown in Figure 2, when a load-bearing component is removed, the structural resistance is shared by adjacent upper relic structures; in the beam mechanism stage, all three beams are subject to bending moments. The bearing moment of the beam is related to the span and torsional stiffness.

When considering the floor effect, the neutral axis moves up. For the beam mechanism, the strains of the beam ends are shown in Figure 3; i.e., when the load-bearing component fails, adjacent beam ends are subjected to tension strains at the bottom and compressive strains at the top. Tensile strains are shown at the top of the distal beam ends, and compressive strains are shown at the bottom.

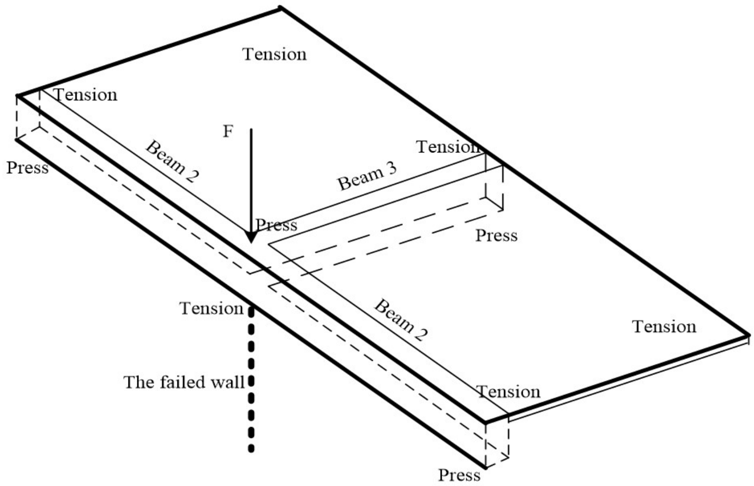


Figure 3 Strains when the edge wall fail

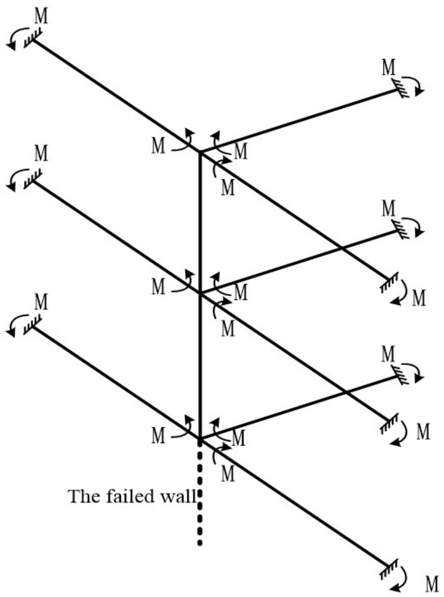


Figure 4 Stress of the substructure when the edge wall is removed

For multi-layer and high-rise structures, when partial load-bearing components of the structure are damaged, the original loads are borne by the overall upper structure, as shown in Figure 4. The upper substructure is considered a space truss, which bears the original loads. Structural deformation is mainly concentrated in two spans above the removed components, while the deformation of adjacent spans is minimal, which is considered the boundary condition of adjacent spans. The bottom beam ends have the strongest constraint, and the top beam ends have the weakest constraint. When the bottom components are removed, the constraint force of the beam ends gradually decreases when the floor increases, the stresses and strains of the beam ends also gradually decrease, and the minimum stress and strain are located in the top beam ends. The stresses and strains of the adjacent beam ends of the dismantled components gradually decrease with increasing floor where the failed components are located. However, when the top floor components are removed, the stresses and strains at the ends of the adjacent beam are different from those of other components. When the top-floor components are dismantled, the upper beams cannot form a space truss, and adjacent beams only bear the load by the bending moment, so the structural bearing capacity is different from those of other floors. Adjacent floorslabs have similar stresses and strains to the beams.

4 COLLAPSE RESISTANCE DESIGN OF THE SHEAR WALL REPLACEMENT

4.1 Project introduction

Take a shear wall structure in China (as shown in Figures 5 and 6) as an example. The structure has 14 floors on the ground, the story height is 2.9 meters, and its total height is 41.5 meters, which are used for residential and office property. The underground layer is an automatic garage, and its height is 3.6 meters. A raft foundation and a shear wall are adopted in the structure. When the major structure of this building was constructed to the 10th floor, the concrete strength of the partial shear walls on the 6th floor of the building failed to satisfy the design strength of 25 MPa, and replacement and reinforcement were proposed. The finite element analysis model is established to analyze the construction process of the structure. In this model, SHELL63 is used for the shear walls and floorslabs, and BEAM188 is used for beams. Based on the alternate load path method, stresses and strains were analyzed when key components were dismantled on different floors, and the results are shown in Figure 6.

Figure 5 Structural facade figure

4.2 Loads and key components

The alternative load path method is a common method to study the progressive collapse resistance performance of structures. By removing one or more vertical load-bearing components in the structure, the bearing capacity, internal force and structural deformation of the relic structure can be intuitively shown. The American GSA code (General Services Administration, 2003), DoD code (U.S. Department of Defense, 2009), Japan steel structure association code (Japanese Society of Steel Construction & Council on Tall Buildings and Urban Habitat, 2004), and Chinese code for anti-collapse design of building structures (China Engineering Construction Standardization Association, 2014) specified some parameters that should be considered in the analysis process, such as material characteristics, load, combination, and acceptable damage degree.

In this paper, load combination conditions of the static alternative load path method recommended by the GSA2003 code (General Services Administration, 2003) are used.

$$G_E = 2(D + 0.25L) \tag{4}$$

where D and L are the dead load and live load, respectively.

When key components of the structure are removed, it will cause insufficient vertical resistance of the relic structure and vertical collapse of the structure. It is helpful to study the vertical collapse of the structure and analyze the relic strains of the structure when partial components fail. The key components (such as ⑮/Ⓐ-Ⓑ,㉙-㉛/Ⓒ,㉛/Ⓐ-Ⓑ and ㉒-㉖/Ⓑ-Ⓒ in Figure 6) are selected according to the method in the GSA2003 code (General Services Administration, 2003), span and location are considered (number of beams in those figures are compiled according to the plane layout, and strains are obtained from the beam bottom).

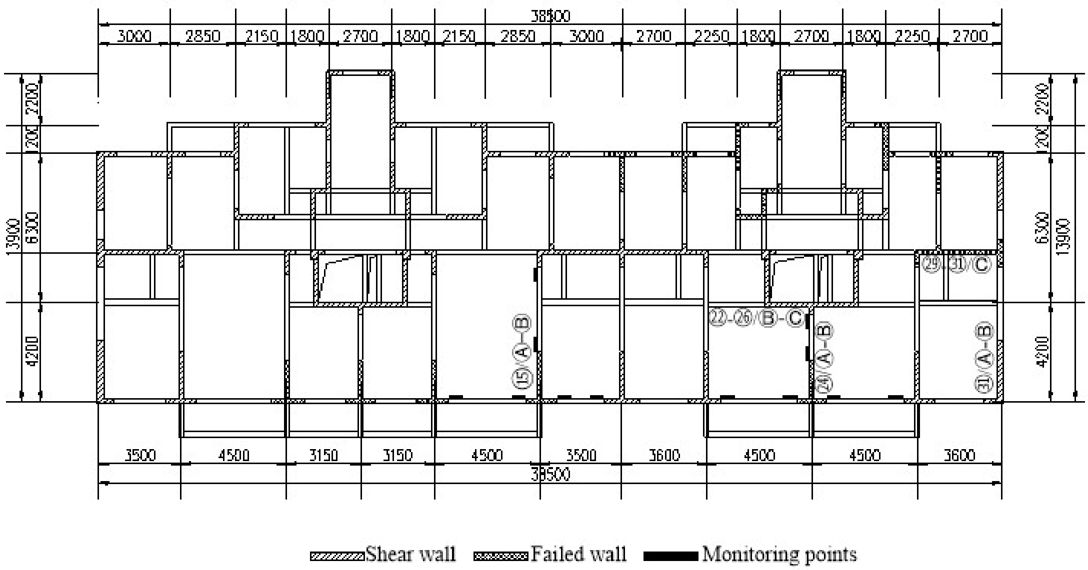


Figure 6 Shear wall structure layout and failure walls

4.3 Deformation of the relic structure

The static linear analysis method was used to analyze the deformation of the structure when key components on different floors are removed, and the maximum vertical deformations of the structure are shown in Figure 7.

When key components (as shown in Figure 7) are removed on different floors, it will cause a sudden change in vertical deformation. When the 4th floor shear wall ⑮/Ⓐ-Ⓑ is removed, the vertical deformation reaches 1.57 mm. When the floor of the shear wall location increases, the vertical deformations first increase and subsequently decrease. Vertical deformation is maximal when the 4th floor shear wall is removed. Wall ⑮/Ⓐ-Ⓑ adjacent beam span is the largest; when the wall is removed, the structural deformation changes the most. Wall ㉙-㉛/Ⓒ adjacent beam span is the smallest, so the vertical deformation hardly changes when the wall is removed. When the wall ㉒-㉖/Ⓑ-Ⓒ on the 14th floor is removed, it causes a sharp increase in vertical displacement because the location on the 15th floor is the elevator room. The wall removal causes insufficient bearing capacity, and the vertical deformation increases. When the corner shear wall ㉛/Ⓐ-Ⓑ on the 14th floor is removed, a sharp increase in vertical displacement occurs. The constraining force of the upper beam decreases, it transfers loads only through adjacent cantilever beams and cannot form a space truss, so the vertical bearing capacity is insufficient, which makes the vertical displacement of the structure increase.

Structural vertical deformations caused by the removal of the shear wall on the first floor are the largest. Except for elevator room walls and corner walls, deformations are minimal when the top floor wall is removed. Structural vertical deformations caused by the removal of walls in the same vertical position decrease with the increase of the floor. For shear walls with identical constraint conditions, the vertical deformations of the structure caused by the removed walls are larger when the span is large; conversely, the vertical deformations are smaller when the span is small.

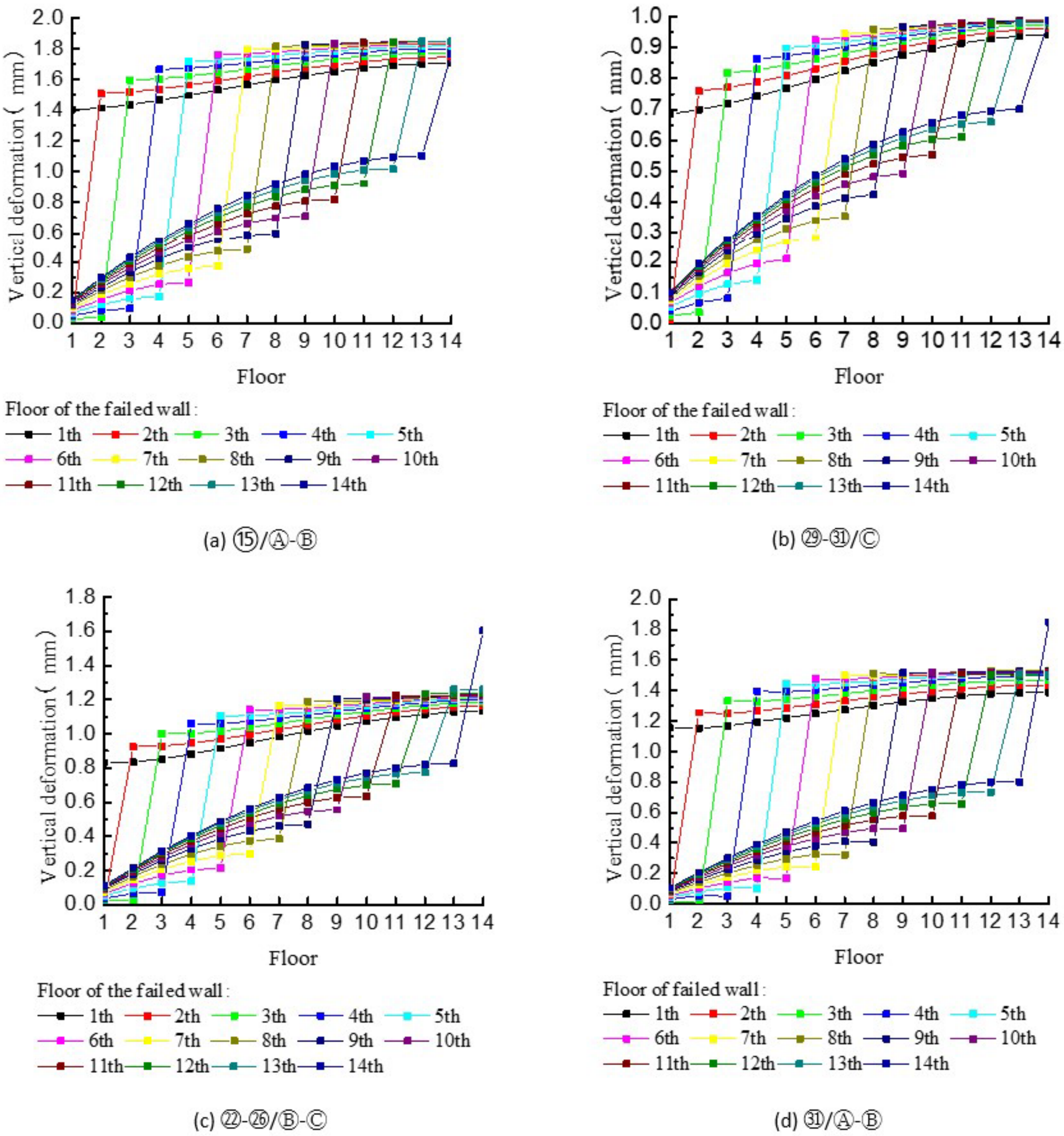


Figure 7 Vertical deformation of the structure

4.4 Strains of adjacent beam ends of relic structure

A vertical deformation analysis of the structure shows that vertical deformations caused by the wall ⑮/Ⓐ-Ⓑ removal are maximal and will lead to the maximum strain of adjacent beam ends. When other shear walls are removed, the structure has a similar strain to wall ⑮/Ⓐ-Ⓑ. Figure 8 shows the strain diagrams of each adjacent beam end when wall ⑮/Ⓐ-Ⓑ is removed on different floors.

When the wall is removed, it will cause stress mutation of adjacent beam ends, the removal of wall ⑮/Ⓐ-Ⓑ on the 1st floor makes the maximum strain of adjacent beam ends 330 με. The strains of the beam ends near the removed wall are positive and negative on the distal ends because all beams are subjected to bending moments when the key component is removed. When considering floorslabs, the neutral axis of the beam moves up, the floorslab is subject to tensile stress above the neutral axis, and the section below the neutral axis is subject to the compressive stress of the beam section below the neutral axis.

In general, the strains in the structure caused by the removal of bottom components greatly change. When the floor of the removed components increases, the corresponding beam end strains decrease. When the top wall is removed, the strains of the beam ends slightly change. The reason is that those beams are similar to cantilevers, and the vertical force transfer paths sharply decrease when the wall is removed. The structural strains are relatively small and far smaller than the elastic limit strain of concrete. Therefore, the shear wall structure remains in the elastic stage after one key component is removed.

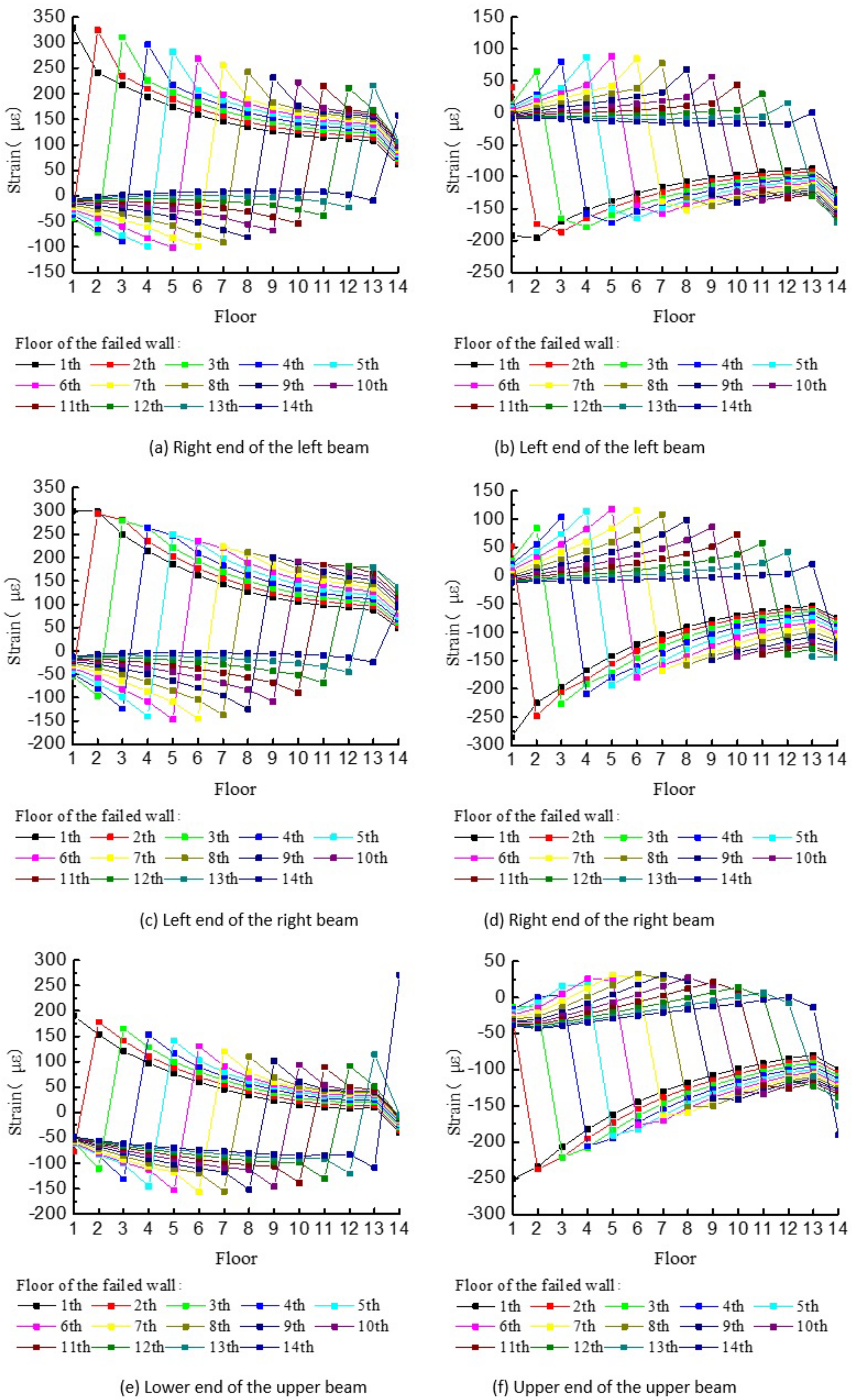


Figure 8 Strains at the ends of adjacent beams when shear wall ⑮/Ⓐ-Ⓑis removed

From the deformations and stresses of each beam when walls are removed on each floor, the substructure has the largest deformations when the wall on the first floor is removed. The strains at adjacent beam ends are shown in Figure 9 and Figure 10 when key components on the 1st and 6th floors are removed.

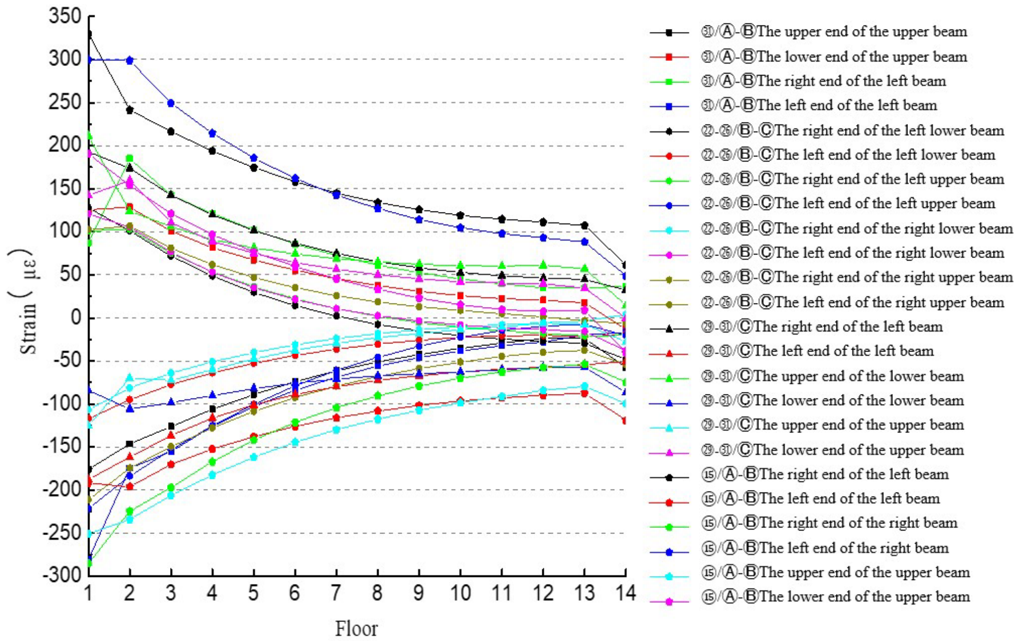


Figure 9 Strains at the ends of adjacent beams when key members are removed on the 1st floor

Figure 9 shows that when wall ⑮/Ⓐ-Ⓑ is removed, the strains of adjacent beams change the most because the adjacent beams have the largest spans and longer adjacent vertical force transfer paths, which leads to the largest bending moment in the beam ends. When wall ㉙-㉛/Ⓒ is removed, the minimum strains of the beam ends occur because the smallest span and shortest load transfer paths cause the minimum bending moment and strain. In summary, when the span increases, the strains at the beam ends of adjacent beams will increase; in contrast, when the span decreases, the strains at the beam ends will gradually decrease. When the wall is removed, the strains of each beam end on the upper structure change. Therefore, when the components are removed, the loads will be shared by the relic substructure.

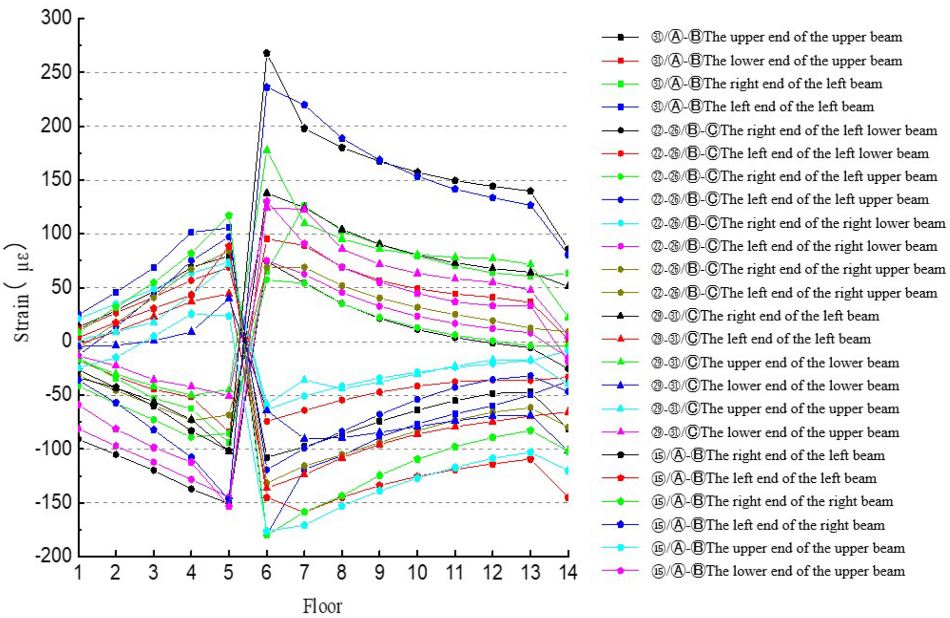


Figure 10 Strains at the ends of adjacent beams when key members are removed on the 6th floor

In Figure 10, the key components on the 6th floor are removed, which will cause sudden stress changes in adjacent beams on upper and lower floors and structural stress changes. When wall ⑮/Ⓐ-Ⓑ on the 6th floor is removed, the change in structural strain is maximal. When walls ㉒-㉖/Ⓑ-Ⓒ and ㉙-㉛/Ⓒ are removed, the strains slightly change because adjacent force transfer components are closer and the redundancy of the structure is large.

5 REPLACEMENT CONSTRUCTION OF SHEAR WALLS

5.1 Construction of shear wall replacement

As shown in Figures 5 and 6, the concrete strength of the partial walls on the 6th floor of a 14-story shear wall structure is insufficient due to construction defects. To satisfy seismic requirements and not affect later use, the defective walls are replaced and reconstructed. Since the superstructure remains under construction during shear wall replacement construction, the superstructure loads are designed on the entire structure.

Field investigation of this project shows that there are many defective shear walls. Because of the large plane size of shear walls, it is proposed to adopt a piecewise replacement method for construction. Full support is set up on the floor to ensure safety during construction. These shear walls are dismantled by hydraulic crushing and static cutting technology. These methods are characterized by no noise, no vibration, no flying stone, and no pollution and are easy to control. Because shear walls that must be replaced are in the middle floor, the overall disturbance of construction must be small. If shear walls are dismantled by violent demolition, it will cause great risks to the construction of the superstructure and damage structural components of adjacent layers. Permanent supports are made of concrete-filled steel tubes, as shown in Figures 11 and 12. Steels with a yield strength of 235 MPa are used for permanent supporting steel tubes and connecting steel plates. The strength of the grouting material is approximately 60 MPa, and the strength of the bolt rod is 800 MPa. Figure 13 shows the shear wall replacement construction site for this project.

5.2 Monitor system

The construction method of shear wall replacement is complicated, and the operation time is long. However, the construction process can be considered a special case of the analysis of the alternate load path method. The strain monitoring of local components can ensure the safety of construction, verify the alternate load path method, analyze the load transferring law, and verify the results of theoretical analysis.

When vertical load-bearing components are dismantled, adjacent beams are subjected to bending moments. Through strain monitoring results, the collapse resistance state of the structure can be effectively ascertained.

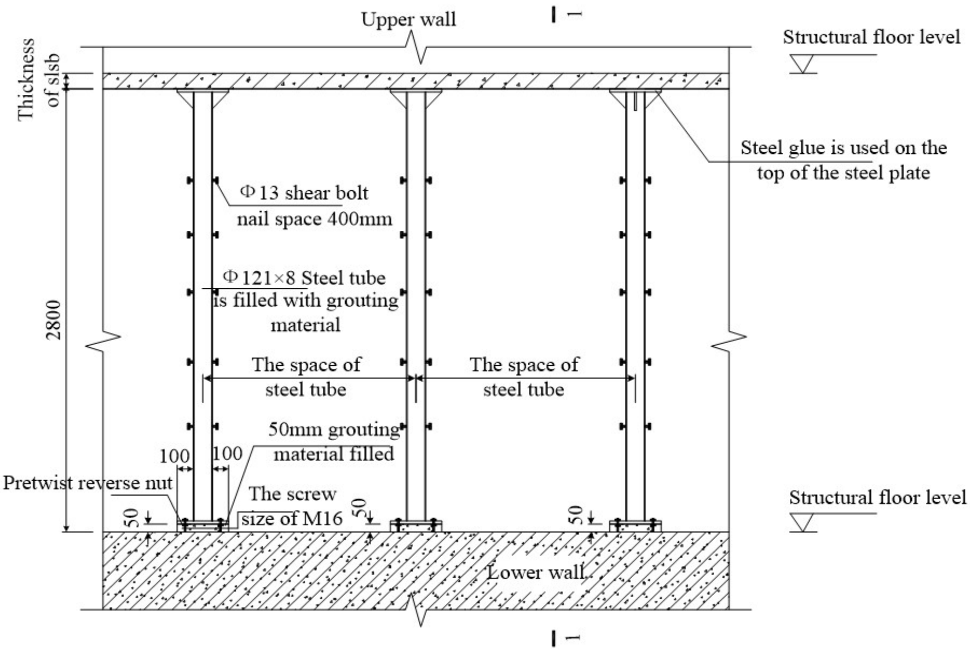
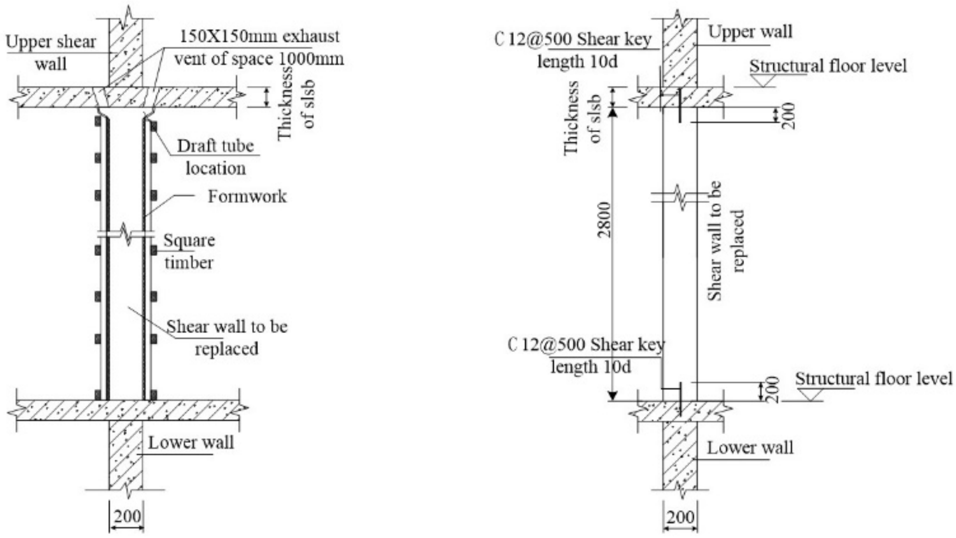


Figure 11 Permanent support facade layout



(a) Wall replacement pouring schematic (b) Schematic of replacement connector

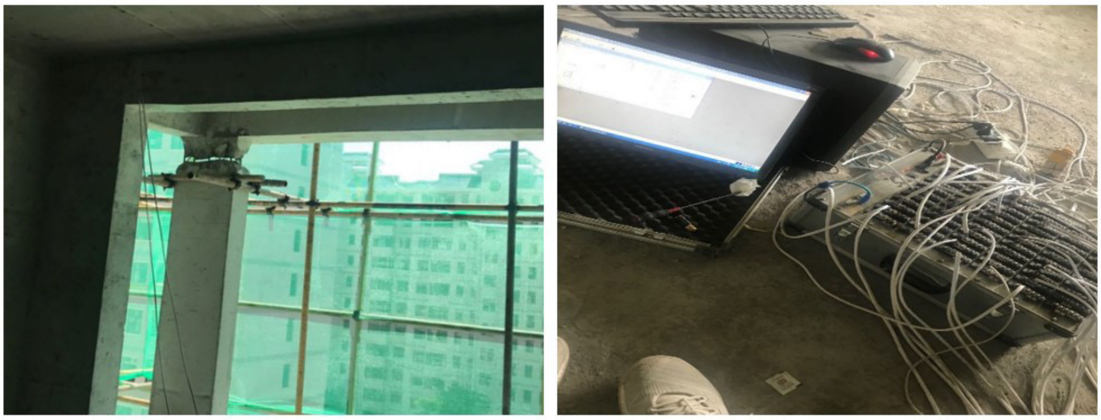
Figure 12 Permanent support cross-section (1-1 cross-section in Figure 11)

(a) Shear wall dismantle (b) Concrete filled steel tube support

(c) Reinforcement bars binding (d) Casting

Figure 13 Shear wall replacement construction

According to the previous analysis results of structural stresses and strains, the displacement of the structure changes little after partial components are removed, while the stresses and strains of the beam ends of adjacent beams change due to the action of the bending moment. Combined with the component replacement and reinforcement scheme, the strains of the upper beam ends are monitored where those defective components are located. Since the 6th floor of the structure is the working area, to prevent accidents and not affect the construction progress, the displacement monitoring system is only set at the bottom of the beam on the 6th floor, and the structural strain monitoring system is set on the 7th floor. Therefore, only monitored strains of the 7th beam ends of adjacent walls ⑮/Ⓐ-Ⓑ, ㉔/Ⓐ-Ⓑ and the arrangement of monitoring points are shown in Figure 6, and field monitoring is shown in Figure 14.



(a) Strain gauge (b)Data collection

Figure 14 Field Monitoring

5.3 Monitoring results and analysis

During the monitoring process, no obvious displacement changes occurred, so the structural displacements were not extracted. Data were taken every hour to monitor the strain results, and 100 hours of data results were extracted. The strain curves of the obtained components are shown in Figures 15 and 16.

In Figures 15 and 16, all strain curves present a two-step phenomenon because the construction was performed in two steps. In addition, for different spans of the left and right beams, the stress changes of the left and right beams are slightly different, and the beams with larger spans have larger strains. For beams with a large span, the strains at the ends of the beam are relatively large after the wall has been dismantled because the wall removal causes a bending moment to form in the beam. For beams with a larger span, their moment arms are also larger, which results in a greater bending moment in the beam. Therefore, the stresses of the beam are also large.

The field results show that all strains at beam ends near the dismantled wall are positive, i.e., the beam ends near the dismantled wall are subjected to tension. However, all strains at the beam ends far away from the dismantled wall are negative, which implies that the beam section is mainly under pressure. Because of the floor slab in the structure, the neutral axis of the T-section beam moves up, which makes the beam section near the dismantled wall below the floorslab experience tension, and the beam ends far away from the dismantled wall are compressed.

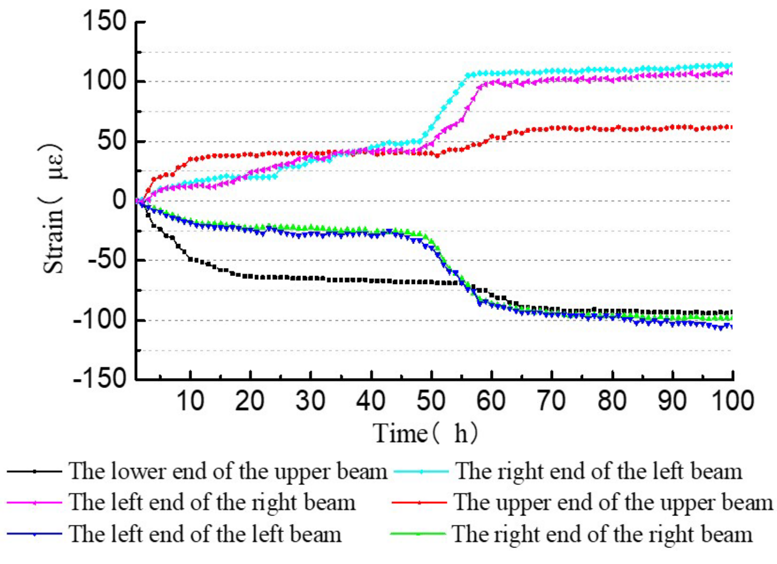


Figure 15 Strains at the ends of adjacent beams when the ⑮/Ⓐ-Ⓑ wall is removed on the 6th floor

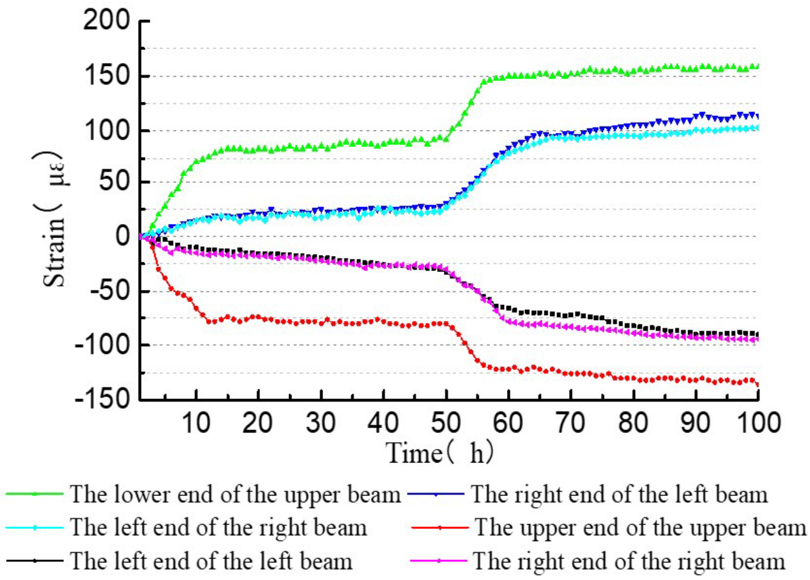


Figure 16 Strains at the ends of adjacent beams when the ㉔/Ⓐ-Ⓑ wall is removed on the 6th floor

The monitoring and FEM results of the strains of the beam ends adjacent to walls ⑮/Ⓐ-Ⓑ and ㉔/Ⓐ-Ⓑ are extracted and shown in Table 1.

Table 1 Strains at the ends of the adjacent beam of the 7th floor after the partial wall has been removed on the 6th floor

No.	Beam section	Right end of left beam	Left end of left beam	Right end of right beam	Left end of right beam	Upper end of upper beam	Lower end of upper beam
⑮/Ⓐ-Ⓑ	Numerical simulation	198	-159	-158	220	-171	91
	Field monitoring	107	-105	-98	114	-93	62
㉔/Ⓐ-Ⓑ	Numerical simulation	146	-124	-116	148	-150	202
	Field monitoring	114	-94	-90	103	-136	158

A comparison of the numerical analysis results with field monitoring results shows that the numerical analysis results are basically consistent with the field monitoring results. In the actual construction process, the strains of the beam ends are smaller than the simulated value because in the construction process, full supports are arranged on the 6th floor to prevent accidents. Therefore, the layout of full support can reduce the vertical deformation of the structure and provide necessary protection for safe construction.

6 CONCLUSIONS

After one shear wall has been removed, all adjacent beams on the upper structure of the dismantled component are involved in load transfer, the structure is in the beam mechanism stage, and the beams mainly bear the moment. When the floorslab is considered, the strains at the ends of the adjacent beam are small, which will not cause large deformation of the structure.

In the beam mechanism stage, the removal of load-bearing components causes stress mutation at the ends of adjacent upper beams, and the loads that were borne by the removed components are borne by the overall upper structure. With the increase in number of structural floors, the strains at the ends of the beam decrease, the load transfer of the superstructure decreases gradually.

The shear wall structure has sufficient bearing capacity by setting structural support when partial component replacement and reinforcement are used. Through a reasonable arrangement of stress and strain monitoring systems in the process of component replacement, accidents can be effectively prevented. Numerical simulation can be used to analyze the structural component replacement process and provide a theoretical reference for the replacement construction of important structures and large space structures.

ACKNOWLEDGMENTS

The research described in this paper was financially supported by the Natural Science Foundation of China (Grant number: 51368039). Higher Education Innovation Fund Project of Gansu Province, China (Grant number: 2022B-215). Science and Technology Project of Gansu Province, China (Grant number: 22JR5RM211).

Author's Contribuitions: Conceptualization, X S Cheng; Methodology, X S Cheng; Investigation, S L Zhang and L Qi; Writing - original draft, L Qi, X S Cheng and S L Zhang; Writing - review & editing, L Qi; Funding acquisition, X S Cheng and L Qi; Resources, X S Cheng and L Qi; Supervision, X S Cheng.

Editor: Pablo Andrés Muñoz Rojas

References

Abbasina, R., Nav, F. M., Usefi, N., and Rashidian, Q., (2016) A new method for progressive collapse analysis of RC frames, Structural Engineering & Mechanics, 60(1), 31-50.

Chai, Y.H. and Kunnath, S.K., (2005) Minimum thickness for ductile RC structural walls, Engineering Structures, 27, 1052–1063.

Chen, Z. F., Lin, K. Q., Lu, X. Z., and Li, Y., (2021) Uncertainty analysis on progressive collapse resistance of RC beam-column substructures, Engineering Mechanics, 38(6), 72-80+90.

China Engineering Construction Standardization Association (2014) Code for anti-collapse design of building structures, China engineering construction standardization association, Beijing, China.

Chrysanidis, T., (2019) Influence of elongation degree on transverse buckling of confined boundary regions of R/C seismic walls, Construction and Building Materials, 211, 703-720.

Chrysanidis, T., (2021) The effect of longitudinal reinforcement ratio on the lateral buckling behavior of R/C walls modelled using prism elements, Journal of Building Engineering, 42, 102456.

General Services Administration (GSA) (2003) Progressive collapse analysis and design guidelines for new federal office buildings and major modernization projects, The US General Services Administration, Washington DC, USA.

Ghannoum, W., (2007) Experimental and analytical dynamic collapse study of a reinforced concrete frame with light transverse reinforcements, University of California, Berkeley, USA.

Hamed, Y., Mohammad, S. G., and Mansoor, Y., (2019) Progressive collapse potential of different types of irregular buildings located in diverse seismic sites, Heliyon, 5(1), e01137.

Japanese Society of Steel Construction & Council on Tall Buildings and Urban Habitat (2004) Construction of steel buildings with high redundancy, Guidelines for Collapse Control Design, Tokyo, Japan.

Jiang, J., Lv, D. G., Lu, X. Z., Li, G. Q., and Ye, J. H., (2021) Research progress and development trends on progressive collapse resistance of building structures, Journal of Building Structures, 43(1), 1-28.

Li, Y., Ye, L. P., and Lu, X. Z., (2011) Progressive collapse resistance demand of RC frame structures based on energy method I: beam mechanism, Journal of Building Structures, 32(11), 1-8.

Ma, F., Gilbert, B. P., Guan, H., and Li, Y., (2020) Experimental study on the progressive collapse behaviour of RC flat plate substructures subjected to edge-column and edge-interior-column removal scenarios, Engineering Structures, 209, 110299.

Mohammad, R. A., Ali, M., and Hassan, M., (2019) Effect of Structural Redundancy on Progressive Collapse Resistance Enhancement in RC Frame Structures, Journal of Performance of Constructed Facilities, 33(1), 1-12.

Mohsen, A. S. and Mohammad, M. J., (2017) Progressive Collapse-Resisting Mechanisms and Robustness of RC Frame-Shear Wall Structures, Journal of Performance of Constructed Facilities, 31(5), 04017045:1-12.

Parisi, F., Scalvenzi, M., and Brunesi, E., (2019) Performance limit states for progressive collapse analysis of reinforced concrete framed buildings, Structural Concrete, 20, 68-84.

Qian, K., Huang, Z. Q., Li, H. H., and Yu, X. H., (2020) Numerical study on progressive collapse performance of infilled RC frame, Journal of Building Structures, 41(S1), 221-229.

Qian, K., Li, B., and Zhang, Z. W., (2015) Testing and simulation of 3D effects on progressive collapse resistance of RC buildings, Magazine of Concrete Research, 67(4), 163-178.

Ren, P. Q., Li, Y., Guan, H., and Lu, X. Z. (2015) Progressive Collapse Resistance of Two Typical High-Rise RC Frame Shear Wall Structures, Journal of Performance of Constructed Facilities, 29(3), 1-10.

U.S. Department of Defense (DoD) (2009) Design of Buildings to Resist Progressive Collapse, Unified Facility Criteria (UFC), UFC 4-023-03, Washington DC, USA.

Yagob, O. and Galal, K., (2009) Progressive collapse of reinforced concrete structures, Structural Engineering & Mechanics, 32(6), 771-786.

Yu, J. and Tan, K. H., (2017) Structural Behavior of Reinforced Concrete Frames Subjected to Progressive Collapse, ACI Structural Journal, 114(06), 63-74.

Yu, J., Gan, Y. P., and Li, S., (2019) Analysis of progressive collapse performance of reinforced concrete frames with full-height infill walls, Journal of Building Structures, 40(11), 112-121.

Zhang, W. X., Wu, H., Zhang, J. Y., Wang, X., and Yi, W. J., (2020) Experimental test on progressive collapse resistance of the spatial behavior of integrated precast concrete frame substructures, China Civil Engineering Journal, 53(5): 42-56.

Zhao, H. L., Zhang, L., Wang, T. C., and Chen, Q. W., (2016) Progressive collapse resistance of reinforced-concrete frames with specially shaped columns under loss of a corner column, Magazine of Concrete Research, 68(9), 435-449.