

Article

Seismic Performance of a Masonry Structure with Large Openings and Equivalent Concrete Columns: An Experimental Investigation

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Abstract

In order to meet its need of functional improvement, the existing masonry structure generally adopts the method of replacing partial walls with concrete frame columns to expand the openings and reduce the number of the longitudinal walls. However, the partial removal of longitudinal masonry walls and the introduction of large openings may result in a nonuniform distribution of lateral stiffness in plan and consequently induce torsional response under horizontal seismic loading. In order to investigate the seismic performance of the existing masonry structure after replacement, a 1:4 scale four-story brick masonry–concrete structure model was designed and made. Based on the principle of stiffness equivalence, the partial walls on the side of the large openings of the model ground-level floor were replaced by frame columns and frame beams, and then the pseudo-static test was conducted on the model. Through the test, the failure patterns of each floor in the structure and the seismic performance indexes such as hysteresis curve, skeleton curve, displacement ductility, stiffness degradation, and energy dissipation capacity, were obtained. The results showed that the yield load of the ground-level floor with the equivalent frame columns is approximately 138% of that of the second and third floors, while its yield displacement is approximately 59% of that of them. That is, after the structure enters the yield stage, its ground-level floor has good bearing capacity and resistance to deformation. The ground-level floor of the structure consumes the least energy as compared to the second and third floors, while the second floor consumes the most energy and has stiffness mutation, and the damage to the walls in such layer is also the most serious. Hence, seismic strengthening of the second story should be considered to prevent the formation of a weak or soft story and the consequent risk of structural collapse. Although there is a significant difference in the material properties between reinforced concrete frames and masonry structures, it is feasible to use the replacement method based on the stiffness equivalence to solve the problem of structure torsion caused by the irregular plane arrangement.

Keywords: masonry structure; large openings at ground-level floor; equivalent reinforced concrete frame columns; pseudo static test; seismic performance

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1. Introduction

Earthquakes are among the most devastating natural disasters worldwide, causing severe casualties and substantial economic losses [1]. Consequently, the requirements for seismic design theory of buildings have become increasingly stringent. Despite the prevalence of concrete and steel structures in China, masonry structures continue to predominate in medium-sized and small cities for residential construction, a phenomenon attributable to the ease of material procurement and the reduced requirements for construction machinery. The most recent data indicates that as of 2020, this category of structure accounted for over 45% of urban housing in China [2]. Due to their advanced age, compromised overall integrity, and resistance to continuous collapse, these structures are particularly vulnerable to damage and typically experience the most severe damage during seismic events [3]. In the process of urban renewal, the functional use of ground-level floor spaces in masonry structures has become increasingly diverse, particularly with the common practice of converting ground-level floors into garages or street-front shops. In designs incorporating the ground-level floor garages to facilitate vehicle access, most designs adopt methods such as enlarging openings and reducing the number of longitudinal walls. However, this approach may cause both an abrupt change in vertical stiffness and a nonuniform distribution of lateral stiffness in plan. The abrupt change in vertical stiffness may result in the formation of a weak story at the ground level, whereas the nonuniform plan-stiffness distribution may produce an eccentricity between the centers of mass and stiffness and consequently induce torsional response under horizontal seismic loading. These adverse effects can further aggravate structural damage and reduce the overall seismic performance of the masonry structure [4,5]. Consequently, it is necessary to undertake a comprehensive study of the seismic performance of masonry structures with substantial ground-level-floor openings. Such study is imperative to ensure the safety of these structures and to promote urban renewal and standardization of the construction industry.

In order to achieve the seismic design objectives of “no damage from minor earthquakes, repairable after moderate earthquakes, and no collapse from major earthquakes” for masonry structures, “Code for Design of Masonry Structures” (GB 50003-2011) [6] and “Code for Seismic Design of Buildings” (GB 50011-2010) [7] enhance the seismic performance of masonry structures from two aspects: seismic measures and seismic load-bearing capacity verification. However, to meet the demand for improved functionality, the walls of existing masonry houses are often replaced with concrete frame columns to enlarge openings and reduce longitudinal walls. The design of these replacement structures can be based on bottom frame structures. Nevertheless, due to the limited relevant experimental research, the seismic performance of these structures requires further investigation. Jia et al. [8], Wu et al. [9] and Liu et al. [10] investigated earthquake damage to buildings in Wenchuan earthquake-stricken areas and found that, for buildings with a garage (or store) on the ground-level floor and residential accommodation above, the large difference in stiffness between the frame and masonry structures can lead to serious damage or even overall collapse of the bottom frame columns or load-bearing masonry walls. The bottom frame structure and the upper masonry structure do not deform coherently in seismic activity, which can result in serious damage or even the overall collapse of the bottom frame columns or load-bearing masonry walls.

Numerous studies on the seismic performance of perforated masonry walls have been carried out by scholars worldwide. These studies have used methods such as low cyclic loading tests and numerical simulations. Voon et al. [11] conducted pseudo-static tests on concrete block walls with eight different opening heights, numbers of openings, and reinforcement lengths. These tests showed that the bearing capacity of walls correlated with the height of openings: an increase in the height of an opening resulted in a

decrease in the walls' bearing capacity. Yañez et al. [12] designed 16 masonry walls with different types of openings. The test results showed that the stiffness of specimens with an opening area of 11% was similar to that of solid walls. However, as the size of the openings increased, the walls' ultimate bearing capacity decreased. Furthermore, experimental investigations conducted by Vera et al. [13] and Moeini et al. [14] have indicated that apertures constituting 20% to 25% of the panel area can result in a reduction in in-plane stiffness and strength by approximately 70% to 80% and 25% to 60%, respectively. Bagheri et al. [15] used Abaqus to model 20 masonry walls and analyzed the effect of the location of openings on seismic performance. The findings indicated that it is optimal for the center of the opening to align with the center of the wall, thereby ensuring the performance of perforated masonry walls in seismic conditions. Zhang et al. [16] conducted a study into the lateral stiffness of perforated masonry walls with code-compliant opening rates. The proposed correction method is based on two pillars: the first is the existing perforated wall calculation method with finite element method calculation and analysis to obtain the hole position change on the wall lateral stiffness of the influence of the quadratic curve change; the second is the existing open hole wall lateral stiffness calculation method and finite element calculation and analysis of the results of the error. Parisi et al. [17] studied the seismic performance of perforated irregular unreinforced masonry walls based on the "macro-unit" method and established an analytical model of seismic load bearing capacity of perforated irregular unreinforced masonry walls and a method for calculating wall deformation. Parisi et al. [17] conducted research into the seismic performance of irregular unreinforced masonry walls with openings, based on the 'macro-unit' method. The establishment of a seismic bearing capacity analysis model and wall deformation calculation method for irregular unreinforced masonry walls with openings was achieved. The research findings indicated that the in-plane seismic bearing capacity is considerably influenced by the irregularity of the wall, particularly in cases of variable opening heights.

Furthermore, scholars have conducted seismic performance studies on reinforced single-leaf masonry structures and scaled-down models of brick-concrete structures with openings on the ground-level floor. Zhao et al. [18] conducted pseudo-static tests on brick masonry wall specimens reinforced with reinforced concrete-masonry composite columns and reinforcing mesh cement mortar facing. The test results indicated that the force mechanism of the reinforced wall was analogous to a reinforced concrete shear wall. Furthermore, the ductility, energy dissipation characteristics, and load-bearing capacity of the reinforced wall were enhanced to a significant extent. Yang et al. [19] conducted pseudo-static tests on shear walls using modified reactive powder concrete as a partial replacement for concrete. The findings indicated that the utilization of reactive powder concrete at both sides of the wall led to a substantial enhancement in the load-bearing capacity, energy dissipation, and ductility of the shear walls. Liu et al. [20] conducted a comparative and analytical review of the seismic performance of bottom frame masonry structures with seismic walls (BFMSWs) and multi-story masonry houses from multiple vantage points, including floor story shear, inter-story drift, and crack patterns. The findings indicated that BFMSWs require the reinforcement of seismic structural measures on the second floor (transition floor). It is imperative that the seismic design incorporates a consideration of the adverse effects that could be induced by the excessive lateral movement on the ground-level floor of the BFMSW. Gao et al. [21] performed quasi-static tests on a 1:2 scale model of a seven-story bottom frame seismic masonry wall house. They compared the ultimate bearing capacity with the results of the experiment and evaluated the seismic capacity. The results showed that the second story of the BFMSW model was more complex and responsible for transmitting the seismic shear force and overturning moment of the upper story. Liu et al. [22], Liang et al. [23], and Zhou et al. [24] conducted

shaking table tests on a 1:5 scale model of a two-story masonry structural building with a ground-level floor intended for commercial use. The test results showed that the difference in stiffness between longitudinal and transverse walls caused an irregular distribution of seismic forces. These studies provide valuable insights for the seismic design of multi-story masonry buildings and the seismic strengthening of existing masonry structures.

In fact, solving the problem of asymmetry in plane stiffness and abrupt changes in vertical stiffness would resolve the negative impact of large openings on the ground-level floor and reduced longitudinal walls on the seismic performance of masonry structures. Liang et al. [25] conducted shaking table tests on standard ground-level floor multistory masonry houses constructed at a reduced scale of 1:5 and masonry house models with wing columns, respectively, to compare and analyze the damage process, acceleration amplification factor, relative displacement and typical positional strains and other parameters of the models. The test results showed that the collapse-resistant performance of the ground-level floor multistory masonry houses with wing columns was significantly improved. On this basis, installing reinforced concrete frame columns and frame beams at the intersections between the longitudinal and transverse walls on both sides of the large openings may be regarded as a retrofit strategy for improving the seismic performance of multistory masonry buildings with large openings at the ground-level floor. The necessity of this method lies in the fact that it can not only satisfy the functional requirement for an open ground-level floor space, but also restore the local vertical load-bearing and horizontal force-transfer paths by partially replacing the removed walls. This is conducive to improving the stiffness distribution in plan, mitigating the torsional response induced by nonuniform stiffness, and reducing the risk of forming a weak story at the ground-level floor. Different from previous studies that mainly focused on isolated perforated masonry walls, conventional bottom frame masonry structures, or overall strengthening measures such as wing columns and mortar facings, this method is a local replacement strategy for existing masonry structures after partial wall removal at the ground-level floor. Specifically, part of the exterior longitudinal wall is replaced by reinforced concrete frame columns and frame beams, and the cross-sectional dimensions of the replacement frame system are determined according to the stiffness equivalence principle, so that its lateral stiffness is compatible with that of the replaced masonry wall limbs. Compared with conventional bottom frame masonry structures in which the bottom frame acts as the main load-resisting system, this method places greater emphasis on maintaining the overall stiffness distribution of the original structure while satisfying the functional requirements of building renovation. However, there is currently no clear design code for this approach. It is not clear what the damage pattern of this type of structure would be under seismic action, how the structural elements would work together, or whether the “stiff on top, flexible on the bottom” problem could be solved.

Against the above background, the present paper has designed and constructed a scaled-down model of a masonry structure with a large opening at the ground-level floor with the scale of 1:4. A pseudo-static test has been conducted based on the stiffness equivalence method, involving the replacement of a section of the exterior wall with the reinforced concrete columns and beams on the side of the large opening at the ground-level floor. The purpose of this test was to assess the seismic performance of the model.

2. Description of the Investigated Structural System

The investigated system was a masonry structure in which portions of the exterior longitudinal walls at the ground story were removed to create large openings and replaced by reinforced concrete frame columns and frame beams. The specific structural

layout, geometric configuration, and design of the replacement frame adopted in the tested model are described in this section.

2.1. Configuration of the Investigated Structural Model

The test focused on a six-story masonry structure typical of medium and small cities. The scaled-down model was utilized due to its qualitative analyzability and ease of operation [26]. The test was designed and produced as a scaled-down model of a four-story masonry and concrete structure, with a scale of 1:4. The dimensions of the model were as follows: The dimensions of the structure were 3.71 m × 2.86 m × 2.85 m (the thickness of 60 mm for the external wall), and the height of the ground-level floor (garage floor) was 0.6 m. The remaining typical floor heights were 0.75 m. The configuration of the windows on the second, third, and fourth floors of the model had been meticulously designed to align with specific functional requirements. The substantial fenestrations along the A-grid line on the ground-level floor were strategically positioned to replicate the utilization demands of the designated space, which included the garage. In contrast, the ordinary door openings were situated along the D-grid line. The walls along the A-grid line and D-grid line were two longitudinal walls of the same structural model rather than two independent specimens. Although the masonry walls along the ground-floor A-grid and D-grid lines differed in opening arrangement and wall-limb dimensions, they belonged to the same integral specimen, and the overall reinforced concrete frame system remained unchanged. The employment of reinforced concrete frame columns and frame beams fulfilled a dual function. Firstly, it facilitated the replacement of a section of the external walls on the side of the large fenestrations on the ground-level floor of the model. Secondly, it compensated for the loss of stiffness induced by these large openings, thereby ensuring the structural integrity of the building remains intact. The schematic diagram of the structural model is shown in Figure 1. The reinforced concrete frame columns (hereinafter referred to as frame columns) are named KZ1 to KZ5, and the reinforced concrete frame beams (hereinafter referred to as frame beams) are named KL1 to KL4.

The thickness of the walls in the structural model was 60 mm. The concrete slab on the first and second floors of the model was 60 mm thick with two layers of bidirectional HRB400 reinforcement with a diameter of 4 mm at a spacing of 100 mm. The top slab on the third floor needs to bear a large horizontal load, and the top slab on the fourth floor needs to bear a large vertical load. Therefore, the concrete slab on the third and fourth floors was 80 mm thick with two layers of bidirectional HRB400 reinforcement with a diameter of 8 mm at a spacing of 100 mm.

Two sizes of large openings were provided in the wall along the A-grid line on the ground-level floor, with widths × heights of 592.5 mm × 490 mm and 935 mm × 490 mm, respectively. The opening dimensions in the wall along the D-grid line on the ground-level floor were 225 mm × 540 mm and 400 mm × 540 mm, respectively. The openings in the walls on the second to fourth floors had identical dimensions, with a width × height of 375 mm × 445 mm.

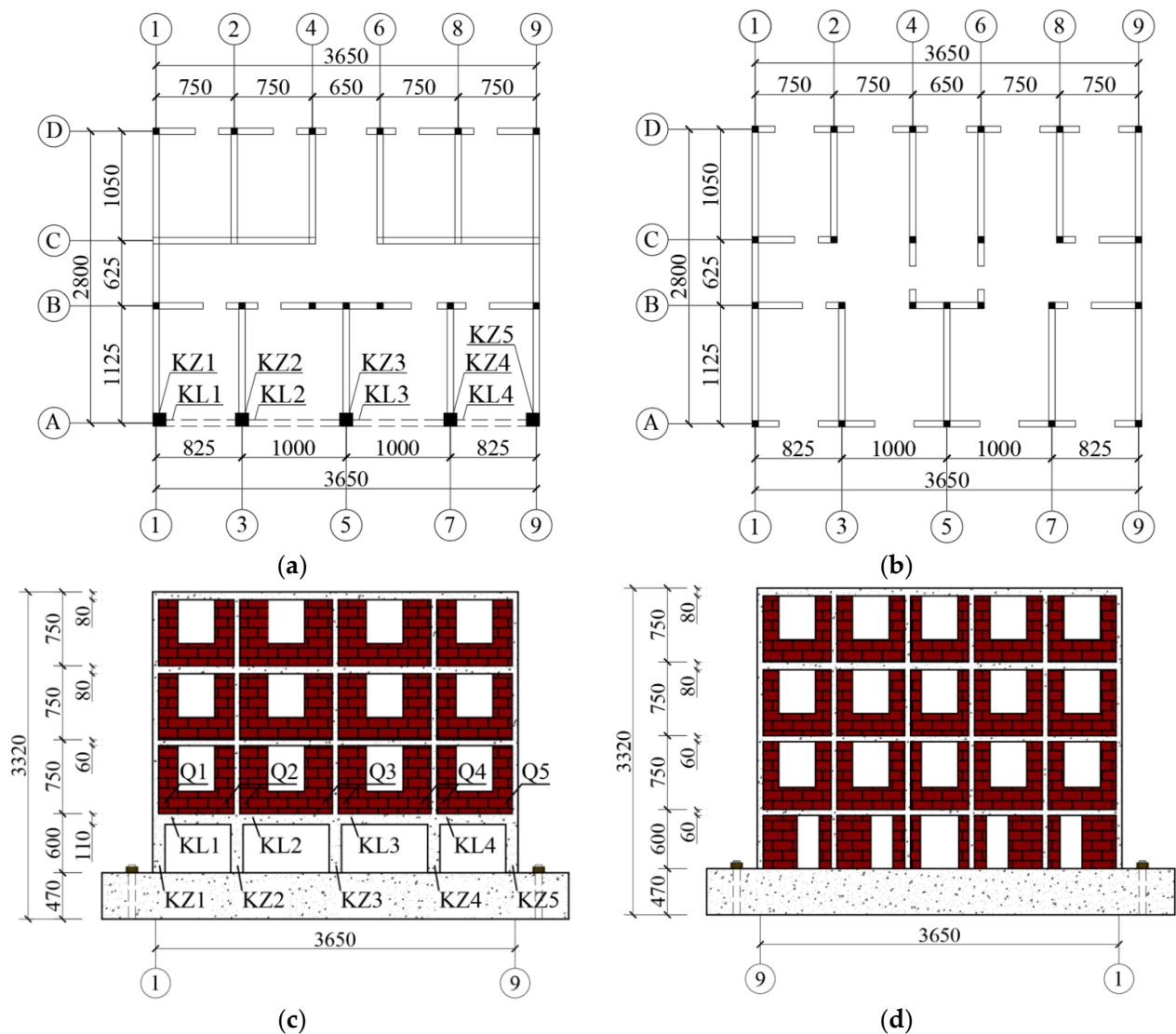


Figure 1. Schematic diagram of the structural model (unit: mm). (a) Ground-level floor. (b) Typical floor. (c) Side facade of the ground-level floor with large openings and partial masonry infill. (d) Side facade of the ground-level floor with door openings and partial masonry infill (Note: The circled numbers indicate the grid numbers; KZ1–KZ5 and KL1–KL4 denote the reinforced concrete frame columns and reinforced concrete frame beams, respectively; the red-filled areas represent ordinary sintered bricks; and the dashed lines indicate the ground anchor bolts).

2.2. Design of the Stiffness-Equivalent Replacement Frame

The frame columns on the side of the large openings on the ground-level floor of the model were square sections, and their dimensions were determined by the stiffness equivalence method. The stiffness-equivalence criterion was adopted because the primary purpose of the replacement frame was to compensate for the loss of lateral stiffness caused by the removal of the longitudinal masonry walls and to maintain, as far as possible, the original story-wise and in-plane stiffness distributions of the structure. These stiffness characteristics directly affect the distribution of seismic forces, the concentration of inter-story drift, the torsional response induced by stiffness eccentricity, and the potential formation of a weak ground story. By contrast, a strength-equivalence criterion could provide a comparable ultimate resistance but would not necessarily prevent a significant mismatch in the initial lateral stiffness between the reinforced concrete frame and the replaced masonry wall limbs. Similarly, a strain-energy-equivalence criterion would require the prior specification of a particular loading or deformation level and corresponding nonlin-

ear constitutive relationships, which cannot be uniquely defined for the present masonry–reinforced concrete composite system before testing. Therefore, stiffness equivalence was selected as the primary criterion for determining the column cross-sections, while the reinforcement details of the frame columns were specified in accordance with GB 50010-2010 [27]. Firstly, in Equation (1), we calculated the lateral stiffness (K_q) of each wall limb (Q1–Q5) in the external longitudinal wall at the typical floor shown in Figure 1c (The calculated results are shown in Table 1). Then, we added up the lateral stiffness of each wall limb to obtain the total lateral stiffness (K_1) of the typical floor on the side of the large openings, as shown in Equation (2).

To clarify the mechanical basis of Equation (1), each masonry wall pier was idealized as a homogeneous, linearly elastic rectangular member. The cross-sectional area and moment of inertia of the wall pier are $A = bt$ and $I = tb^3/12$, respectively. Under a horizontal shear force (V), the lateral displacement of the wall pier consists of shear and flexural components. Taking the shear-stress nonuniformity coefficient of the rectangular section as 1.2 and the shear modulus of masonry as $G = 0.4E$, the shear displacement of the wall pier can be expressed as $\Delta_s = 3V\rho/Et$. Under the rigid-floor assumption, the rotations at the upper and lower ends of the wall pier are restrained, and its flexural displacement is given by $\Delta_b = V\rho^3/Et$. The lateral stiffness of the wall pier is defined as $K_q = V/\Delta$. For a squat wall pier with $\rho < 1$, the flexural component is neglected in the simplified calculation, and $\Delta = \Delta_s$, giving $K_q = Et/3\rho$. For $1 \leq \rho \leq 4$, both shear and flexural deformations are considered. Therefore, $\Delta = \Delta_s + \Delta_b$ and the corresponding lateral stiffness is $K_q = Et/(3\rho + \rho^3)$. Therefore, the terms 3ρ and ρ^3 in Equation (1) represent the contributions of shear and flexural deformations to the lateral flexibility of the wall pier, respectively. The flexural contribution increases with the cube of the height-to-width ratio and therefore becomes increasingly significant as ρ increases. In the present model, the height-to-width ratios of wall piers Q1–Q5 ranged from 1.10 to 2.71; therefore, the second expression in Equation (1) was adopted for all wall piers.

Next, we calculated the lateral stiffness (D) of the frame columns (KZ1–KZ5) on the side of the ground-level floor with large openings, according to Equation (3). Finally, we added up the lateral stiffness of each reinforced concrete frame column to obtain the total lateral stiffness (K_2) of the ground-level floor with large openings, according to Equation (4). The cross-section size of the frame columns was then determined according to the principle that the total lateral stiffness (K_1) of the typical floor on the side of the large opening was equal to the total lateral stiffness (K_2) of the ground-level floor. In this study, the cross-section dimensions and reinforcement of the frame columns (KZ1 to KZ5) were identical to provide the same lateral stiffness. For constructional convenience, the cross-section size of the frame columns (KZ1 to KZ5) was determined as 125 mm $\times$ 125 mm. The frame columns were reinforced with eight 12 mm diameter HRB400 longitudinal rebars. HRB400 stirrups with a diameter of 2 mm were provided at spacings of 25 mm in the densified regions and 50 mm in the non-densified regions.

$$K_q = \begin{cases} \frac{Et}{3\rho} & \rho < 1 \\ \frac{Et}{3\rho + \rho^3} & 1 \leq \rho \leq 4 \end{cases} \quad (1)$$

$$K_1 = \sum_{i=1}^5 K_{qi} \quad (2)$$

$$D = \frac{12i_c}{h_c^2} = \frac{12E_c I_c}{h_c^3} = \frac{E_c B^4}{h_c^3} \quad (3)$$

$$K_2 = \sum_{i=1}^5 D_i \quad (4)$$

where ρ is the height-to-width ratio of the wall limb ($\rho = h/b$), h is the height of the wall limb, and b is the width of the wall limb; E is the elastic modulus of masonry, which is taken according to GB 50003-2011 [6]; t is the thickness of the wall limb; I_c is the moment of inertia of the reinforced concrete frame column section; E_c is the modulus of elasticity of concrete, which is taken according to GB 50010-2010 [27]; B is the section side length of the reinforced concrete frame column; h_c is the height of the reinforced concrete frame column; and i_c is the linear stiffness of the reinforced concrete frame column ($i_c = E_c I_c / h_c$).

Table 1. Lateral stiffness of each wall limb in the external longitudinal wall at the typical floor.

Wall Limb	Height h/mm	Width B/mm	h/b	Lateral Stiffness $K_q/\text{kN/mm}$
Q1, Q5	690	255	2.71	5.79
Q2, Q4	690	538	1.28	27.33
Q3	690	625	1.10	35.03

The risk of forming a weak or soft ground story was mitigated through the combined action of the stiffness-equivalent reinforced concrete frame columns and frame beams. The replacement frame was designed such that its total lateral stiffness (K_2) was approximately equal to the total lateral stiffness (K_1) of the corresponding masonry wall piers before removal. Therefore, the reinforced concrete frame compensated for the loss of lateral stiffness caused by the large openings and restored the local vertical load-bearing and horizontal force-transfer paths. Meanwhile, the frame beams connected the replacement columns with the remaining masonry walls, enabling them to participate jointly in resisting the horizontal action. This design was intended to reduce the abrupt loss of stiffness and load-bearing capacity at the ground story and thereby mitigate the concentration of inter-story drift and damage at this level. The cross-section dimensions of the frame beams were 60 mm × 110 mm. The cross-section size of the constructional columns was 60 mm × 60 mm. The frame beams were reinforced with two 10 mm diameter and two 8 mm diameter HRB400 longitudinal rebars. HRB400 stirrups with a diameter of 2 mm were provided at spacings of 25 mm in the densified regions and 50 mm in the non-densified regions. Their longitudinal reinforcement was configured as four HRB400 rebars with a diameter of 4 mm, and the hoop reinforcement was set up as an encrypted area according to GB 50010-2010 [27]. The encrypted range for corner columns was 150 mm above and below the floor, and for non-corner columns it was 100 mm. The dimension and reinforcement details of the reinforced concrete frame column, frame beam and constructional column are shown in Figure 2.

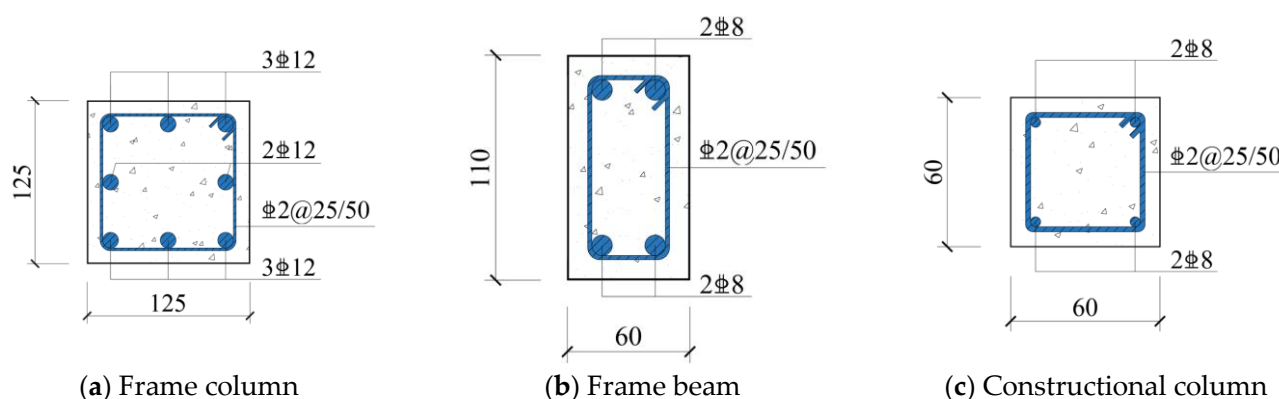


Figure 2. Dimension and reinforcement details of the reinforced concrete frame column, frame beam and constructional column (unit: mm) (Note: The reinforcement symbol “C” denotes HRB400 reinforcing bars, and the number following the symbol indicates the nominal bar diameter in millimeters. The symbol “@” denotes the center-to-center spacing between adjacent reinforcing bars. The blue-filled circles represent reinforcing bars, while the areas filled with triangular hatching represent concrete).

3. Experimental Program

3.1. Specimen Fabrication and Material Properties

The fabrication procedures of the structural model are illustrated in Figure 3, including the construction of ground beams, frame columns, masonry walls, slabs, and constructional columns. The ground beams, slabs, constructional columns, frame columns, and frame beams were cast with C30 concrete (Jinan Junhua Ready-Mixed Concrete Co., Ltd., Jinan, China), and according to GB/T 50081-2019 [28], the measured compressive strength of the concrete cube is 30.6 MPa. The masonry walls were made of ordinary sintered bricks (Shandong Juzhisheng New Materials Co., Ltd., Jinan, China), which were processed according to the similarity ratio, with the dimensions of 115 mm × 60 mm × 53 mm. The masonry bricks were tested for strength following GB/T 5101-2017 [29], and the measured strength is shown in Table 2. The mortar used for the joints between the bottom masonry wall and floor beams of the model was M10, while the rest of the masonry walls were made of M7.5. Strength tests on these two types of mortar were conducted by JGJ/T 70-2009 [30], and the measured strength is shown in Table 3. The masonry walls of the model were constructed using a single-leaf stretcher bond, with the vertical mortar joints staggered between adjacent courses. At wall intersections, the bricks were laid in an interlocking arrangement to ensure the integrity of the intersecting walls. During construction, the thicknesses of both the horizontal and vertical mortar (Jinan Shanshui Cement Co., Ltd., Jinan, China) joints were controlled at approximately 7 mm. Toothed joints were provided at the connections between the masonry walls and the constructional columns. Along the wall height, two 2 mm diameter horizontal tie bars were arranged at a vertical spacing of 125 mm and extended at least 250 mm into the masonry walls to improve the connection between the constructional columns and the masonry walls. The modeled rebar area was scaled down according to the similarity ratio (the area was reduced to 1/16 of the original size). The measured mechanical properties of the HRB400 rebar (Shandong Luzhao Iron and Steel Industrial Co., Ltd., Jinan, China) are shown in Table 4 in alignment with GB/T 228.1-2021 [31].



(a) Ground beam



(b) Frame column



(c) Masonry walls at ground-level floor



Figure 3. Fabrication procedures of the structural model.

Table 2. Strength of masonry bricks.

Average Compressive Strength/MPa	Standard Deviation of Compressive Strength/MPa	Standard Value of Strength/MPa	Strength Grade
20.4	7.5	6.8	MU10

Table 3. Strength of masonry mortar.

Type of Cement Mortar	Designed Strength Grade	Cubic Compressive Strength/MPa
Mortar between masonry wall at ground-level floor and ground beam	M10	15.0
Mortar used for masonry walls	M7.5	8.3

Table 4. Mechanical properties of HRB400 rebar.

Diameter/mm	Yield Strength/MPa	Ultimate Strength/MPa
2	589	791
4	531	743
8	449	607
10	400	590
12	413	570

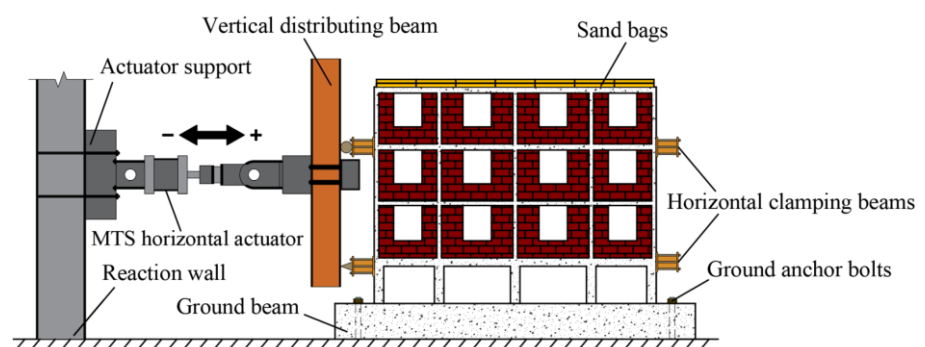
3.2. Loading Equipment and Regime

The model represented the lower four stories of the six-story prototype, while the gravity effects of the unbuilt fifth and sixth stories were simulated by additional uniform loads applied to the top surface of the model [32].

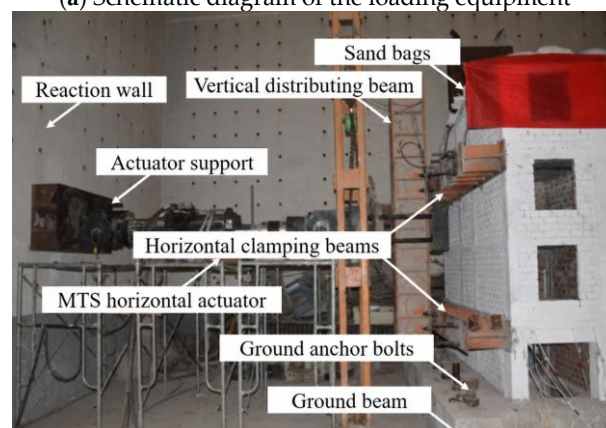
The pseudo-static test loading was divided into two parts: vertical and horizontal loading. Vertical loading was only considered for the effects of live loads and self-weight. According to GB 50009-2012 [33], the standard value of the uniform live load on the surfaces of residential buildings was 2.0 kN/m². In consideration of the model scaling and the constraints imposed by the test conditions, the uniform load was applied to each floor through the stacking of sand bags. Thus, the uniform load of 1.0 kN/m² was applied to the first to third floors of the model, while the uniform load of 3.0 kN/m² was applied to the top slab of the fourth floor.

Horizontal loads were applied through MTS electro-hydraulic servo actuators (MTS Systems (China) Co., Ltd., Shenzhen, China), and the bottom beams were connected to the rigid floor through the anchor bolts. The MTS horizontal actuator exerted horizontal reciprocating loads on the uppermost levels of the first and third floor via the vertical distributing beams and horizontal clamping beams. The positive and negative loads were thrust and tension respectively. The loading equipment and regime are illustrated in Figure 4. The horizontal seismic action calculated from the bottom shear method was utilized to ascertain the horizontal loads applied at the uppermost levels of the first and third

floors. It means that the loads applied to the first and third floors were as follows: $P_{\text{first floor}}:P_{\text{third floor}} = 1:2.27$, where $P_{\text{first floor}}$ and $P_{\text{third floor}}$ were the horizontal loads applied at the uppermost levels of the first and third floors, respectively. The loads used to construct the story-level hysteresis and skeleton curves represented the story shear forces rather than actuator forces measured independently at each story. Accordingly, the shear force of the first story was determined as the sum of $P_{\text{first floor}}$ and $P_{\text{third floor}}$, whereas the shear forces of the second and third stories were both equal to $P_{\text{third floor}}$. The second and fourth floors were therefore not connected to independent actuators. Instead, their contributions to the overall lateral-force distribution were represented by the two equivalent loading resultants. This two-point loading scheme was adopted to approximately reproduce the increasing trend of seismic forces along the structural height and the corresponding global base-shear and overturning effects. Nevertheless, as an equivalent pseudo-static loading scheme, it cannot reproduce the exact floor-by-floor inertia-force distribution or the dynamic effects occurring during an actual earthquake. The horizontal loading was controlled by absolute displacement rather than by multiples of a pre-assigned yield displacement. Because the masonry model did not have a clearly identifiable yield point, the yield displacement could not be reliably determined before testing and was therefore not used to define the loading history. It was subsequently identified from the measured skeleton curves, as described in Section 5.3. Horizontal loading was regulated by displacement, as illustrated in Figure 4. Initially, displacement was set at 0.2 mm. Subsequently, the displacement of each cycle was incremented by 0.2 mm, relative to the displacement of the preceding cycle. It was evident that as the displacement attained a value of 4 mm, an absolute increment of 0.4 mm was exerted to the next cycle. As displacement reached 8 mm, each cycle was augmented by 0.8 mm relative to the previous cycle. This process was continued until the maximum load was attained, at which point the loading was terminated.



(a) Schematic diagram of the loading equipment



(b) Photograph of the loading equipment

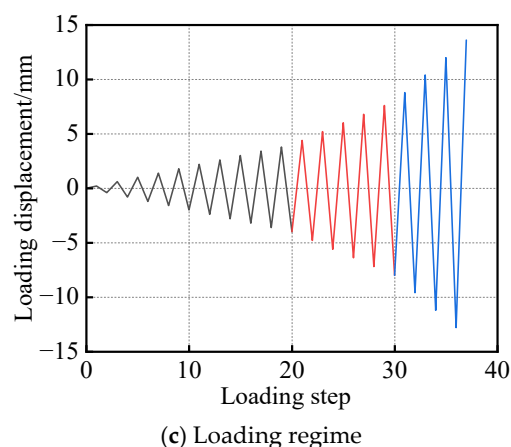
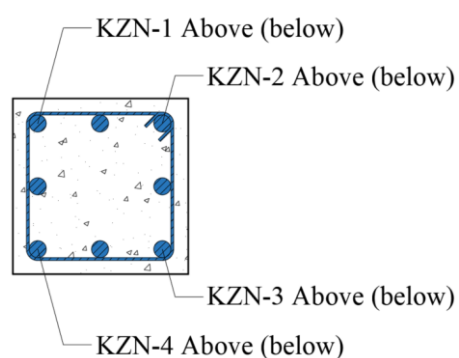


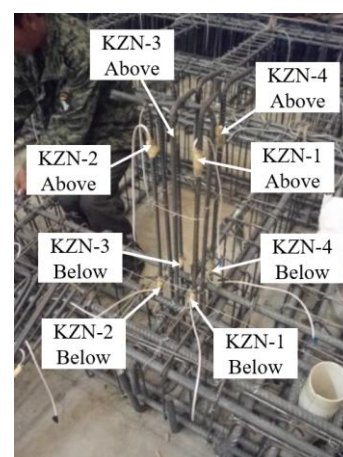
Figure 4. Loading equipment and regime.

3.3. Test Instrumentation

For this test, one strain gauge was positioned at the top and bottom of each of the four corner bars of the frame columns (KZ1-KZ5). Twelve strain gauges were arranged on the mid-span reinforcement and the negative reinforcement at the supports of the frame beams KL1-KL4, which were numbered from L1 to L12 in the direction from the nine-grid line to the one-grid line. The strain measuring point arrangement of the steel rebar in frame columns and frame beams is shown in Figure 5 and Figure 6, respectively. Forty-eight resistance strain gauges were arranged on the surfaces of the walls, concrete frame columns, and frame beams. The strain gauges on the masonry walls were mainly located at the corners of windows, while the concrete strain gauges were not only primarily positioned at the upper and lower ends of the concrete frame columns, but also at the mid-spans and supports of the concrete frame beams. The arrangement of the surface-mounted strain gauges on the walls, frame columns, and frame beams is shown in Figure 7 and Figure 8, respectively. All the strain data were collected by DH3816N static strain gauges (DongHua Testing Technology Co., Ltd., Taizhou, China). Seven displacement meters were arranged on the side of the model facing away from the loading equipment. One is positioned at the center of the top of the first, second, and fourth floors; one at the intersection of the A-grid line and D-grid line at the top of the third floor; and one at the intersection of the A-grid line and D-grid line on the floor beam. The arrangement of the displacement meters in the structural model is shown in Figure 9. The displacement data were collected by the TDS-530 static data acquisition instrument (Tokyo Measuring Instruments Laboratory Co., Ltd., Tokyo, Japan).



(a) Schematic diagram



(b) Photograph

Figure 5. Strain measuring point arrangement of the steel rebar in frame columns (Note: $N = 1-5$).

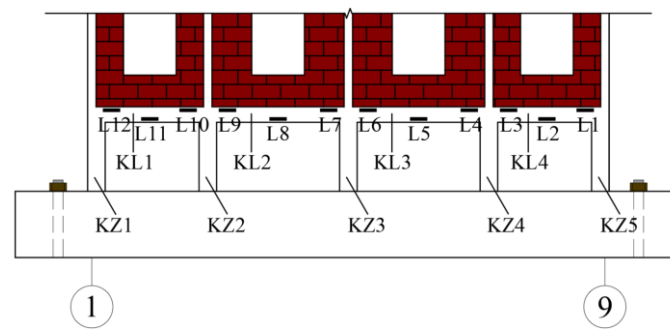


Figure 6. Strain measuring point arrangement of the steel rebar in frame beams (Note: L1–L12 denote the identification numbers of the strain gauges installed on the reinforcement of the frame beams).

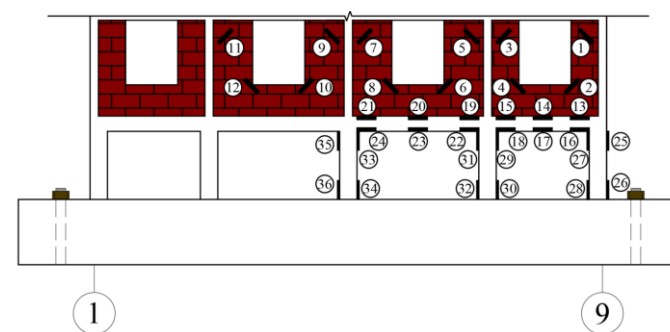


Figure 7. Arrangement of the surface-mounted strain gauges on the walls, frame columns, and frame beams along A-grid line (Note: the circled numbers indicate the identification numbers of the strain gauges).

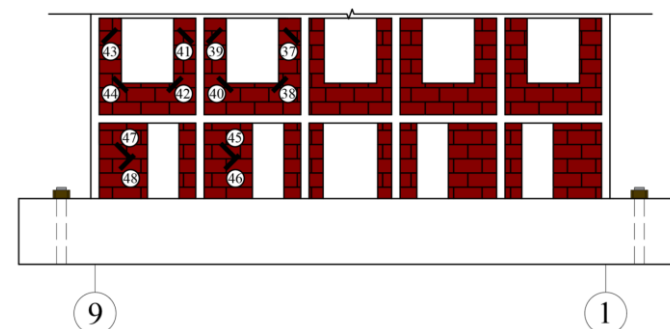


Figure 8. Arrangement of the surface-mounted strain gauges on the walls along D-grid line.

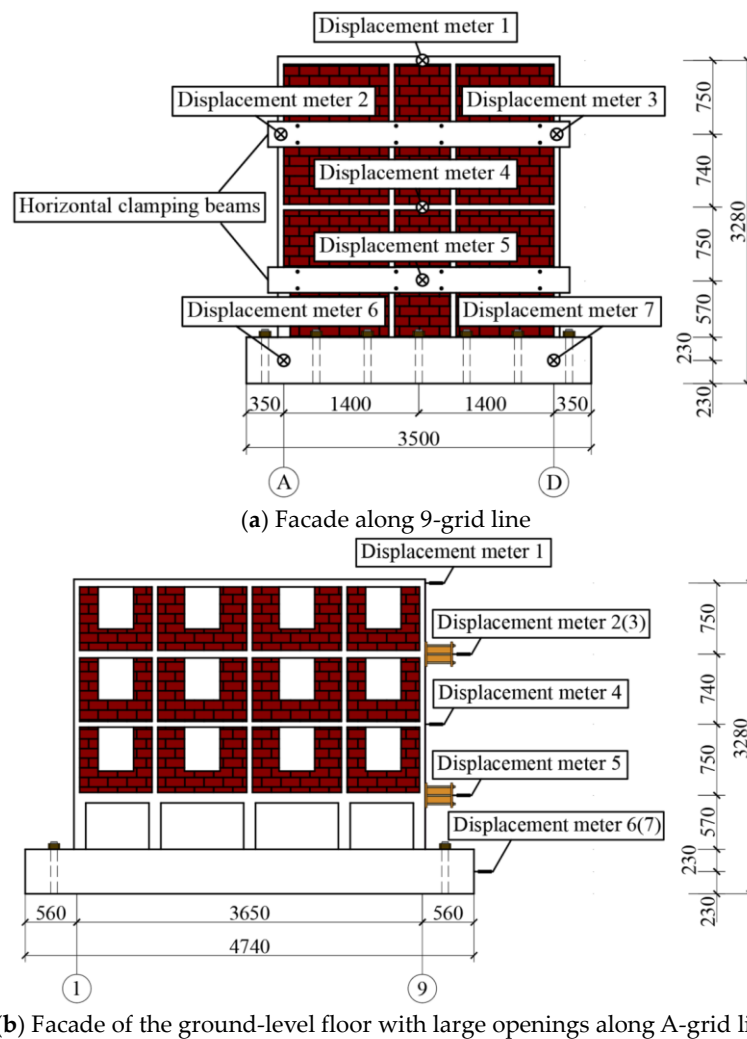


Figure 9. Arrangement of the displacement meters in the structural model (unit: mm) (Note: the numbers following “Displacement meter” denote the identification numbers of the displacement meters).

4. Experimental Phenomenon

For observational purposes, the displacements were all positive while the actuator was producing thrust. A black-colored marker was utilized to trace the cracks and record the location of their development. The displacements were all negative while the actuator was producing tension, and a red-colored marker was utilized for the purpose of tracing the cracks and recording the location of their development. The displacements were defined as the average displacements of the third-floor plate.

4.1. Longitudinal Wall Along A-grid Line

The crack pattern of the frame column on the ground-level floor of 1-grid line is shown in Figure 10. The longitudinal walls at all floors of the model A-grid line were found to be free of cracking when loaded to ± 0.2 mm. However, as displacement increased, horizontal cracks began to appear in the right wall limb along the 3-grid line on the second floor longitudinal wall of the A-grid line at the position of the lower left corner of the window opening, with a crack width of 0.05 mm. Furthermore, when loaded to ± 0.4 mm, a horizontal crack appeared at the top of the frame column at the intersection of the 1-grid line and A-grid line. Despite this, the column did not undergo significant damage, as shown in Figure 10. The crack pattern of the longitudinal wall and constructional column on the second floor along the A-grid line is shown in Figure 11. The results of the

experiment revealed that when the load was set to ± 0.6 mm, stepped diagonal cracks developed along the mortar joints in window corners of the longitudinal walls on the second and third floors of the A-grid line. These cracks increased in width and length with an increase in displacement, as shown in Figure 11. When the load was set to 3.2 mm, horizontal cracks appeared in the constructional columns at the 1-grid line and 9-grid line on the second floor along the A-grid line. These cracks eventually exhibited a distribution pattern with more at the lower end and fewer at the upper end. In the right longitudinal wall limb along the A-grid line on the second floor, the position of the upper left corner of the opening against the window at the 3-grid line, a crack width of 8 mm was observed. This crack was wider than the original crack and appeared in the region where the load had been set to -6.4 mm. The X-shaped cracks on the longitudinal walls along the A-grid line are shown in Figure 12. As can be seen in Figure 12, a load of $+6.8$ mm resulted in the emergence of a wider diagonal crack in the left wall limb at the 3-grid line of the second-floor longitudinal wall of the A-grid line. It occurred at the location of the upper right corner of the opening against the window, with the crack penetrating the structural column. As illustrated in Figure 12, the load of ± 9 mm resulted in the aforementioned crack extending further, penetrating the structural column. The 6 mm cracks in the longitudinal walls and constructional columns of the second and third floors of the A-grid line developed further. The existing cracks in the longitudinal walls and constructional columns of the second and third stories along the A-grid line continued to propagate with increasing displacement. When the loading displacement reached ± 12.0 mm, extensive cracking had developed in both stories, and the growth rate of the lateral load had decreased markedly. Considering the accumulated damage to the structural components and the safety of the test, the model was considered to have reached the test termination condition, and the loading was stopped.

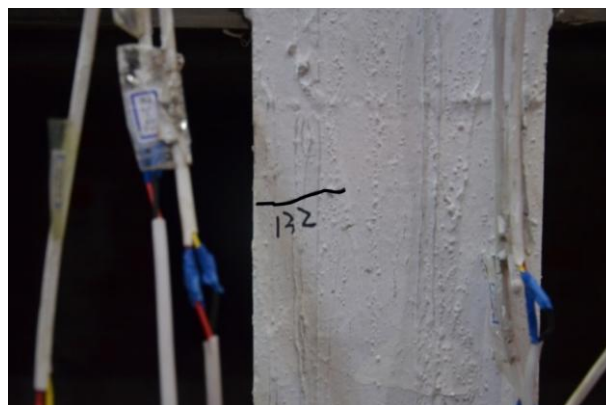
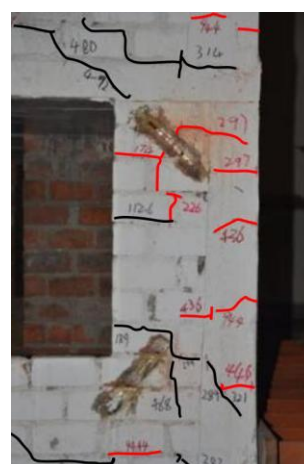
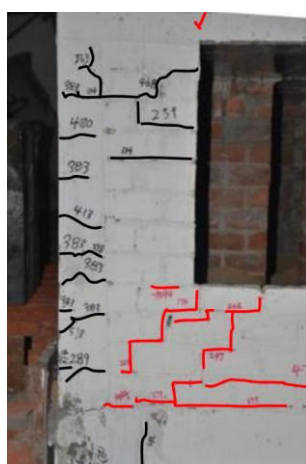


Figure 10. Crack pattern of the frame column on the ground-level floor of 1-grid line.



(a) Wall limb and constructional column at 1-grid line

(b) Wall limb and constructional column at 9-grid line

Figure 11. Crack pattern of the longitudinal wall and constructional column on the second floor along A-grid line (Note: the black color represents the cracks generated by pushing force, and the red color represents the cracks generated by pulling force).

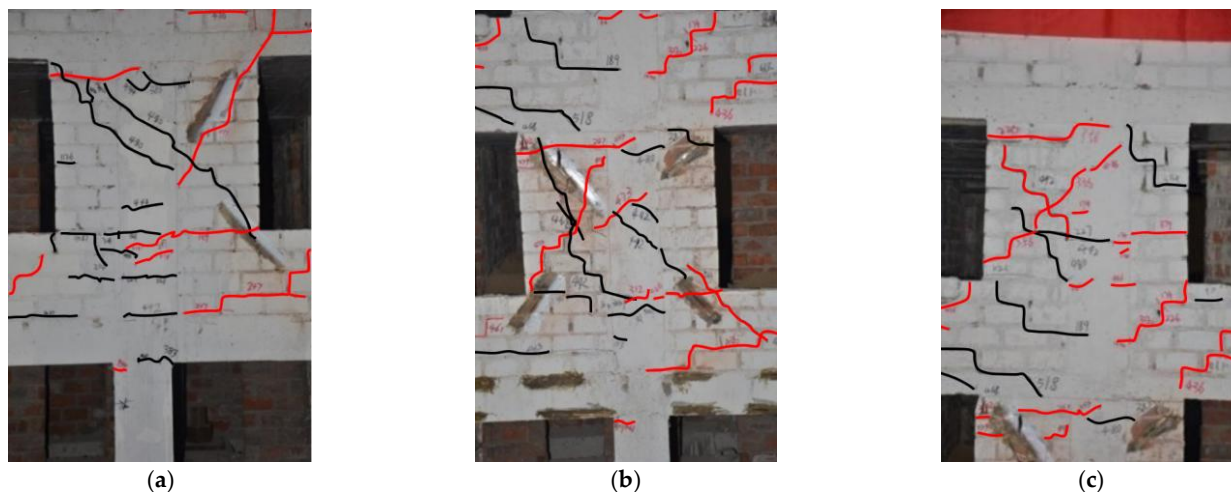


Figure 12. X-shaped cracks on the longitudinal walls along A-grid line. (a) Wall limb and constructional column on the second floor of 3-grid line. (b) Wall limb and constructional column on the second floor of 5-grid line. (c) Wall limb and constructional column on the third floor of 5-grid line.

Due to the poor integrity of the limb between openings, low bearing capacity, and stress concentration generated by the window and door openings [34], the diagonal cracks on the second and third floors between the openings on the A-grid line were distributed in an X pattern. A large number of stepped diagonal cracks also appeared under the window walls on the second and third floors, as shown in Figure 12. The ground-level floor of the A-grid line—consisting of reinforced concrete frame columns, beams, and slabs—exhibited effective composite action. The frame columns on the ground-level floor did not show obvious damage; the cracks were mainly horizontal. The crack pattern of the frame columns at the ground-level floor along the A-grid line is shown in Figure 13. Symmetrically arranged frame columns showed symmetrical crack patterns. The crack distribution on the longitudinal walls and constructional columns along the A-grid line is shown in Figure 14. The crack patterns shown in Figure 14 are used only to describe the observed damage distribution and are not intended to indicate that the second and third stories had the same degree of structural weakness.



Figure 13. Crack pattern of the frame columns at the ground-level floor along A-grid line.

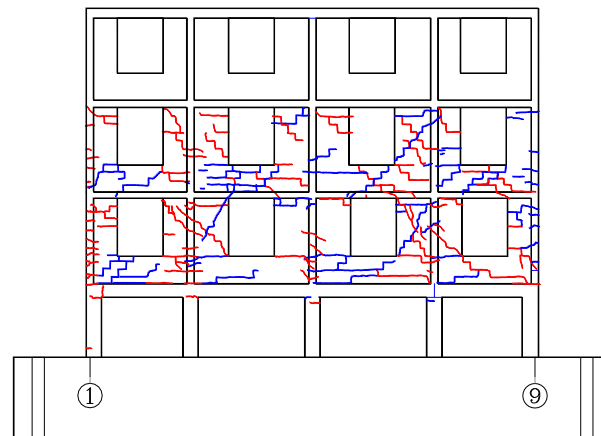


Figure 14. Crack distribution on the longitudinal walls and constructional columns along A-grid line (Note: the red color represents the cracks generated by pushing force, and the blue color represents the cracks generated by pulling force).

4.2. Longitudinal Wall Along D-grid Line

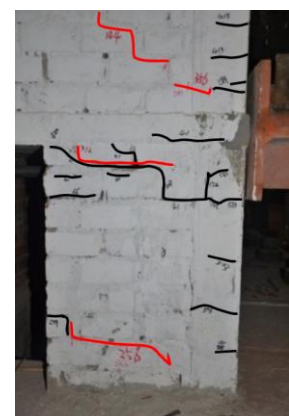
The longitudinal walls along the D-grid line at each floor of the model were found to be free of cracking when loaded to ± 0.2 mm. However, as the displacement increased, horizontal cracks began to appear in the upper mortar joints of the wall limbs at the 1-grid line of the first-floor longitudinal wall of model D-grid line, with a width of 0.05 mm. Concurrently, horizontal cracks emerged on the constructional column at the 2-grid line of the second-floor longitudinal wall of the model D-grid line. The crack pattern of the longitudinal wall and constructional column on the ground-level floor along the D-grid line is shown in Figure 15. As illustrated in Figure 15, 6 mm stepped diagonal cracks were observed to develop along the mortar joints in each wall limb of the ground-level floor longitudinal wall of the D-grid line, at the location of the corner of the door opening. The load was determined to be within the range of ± 6 . The location of the window corners on the second-floor longitudinal wall of the second-floor D-grid line exhibited 8 mm stepped diagonal cracks, which penetrated the constructional columns. The original cracks on the D-grid line longitudinal wall and constructional column were further developed when loaded to ± 9.6 mm. The test concluded when the load reached ± 12.0 mm. Following the conclusion of the test, it was observed that a number of the stepped diagonal cracks present in the first and second floors had penetrated the frame and constructional columns. The diagonal cracks on the second floor exhibited a V-shaped pattern, as illustrated in Figure 16. The crack distribution on the longitudinal wall and constructional column along the D-grid line is illustrated in Figure 17.



(a)

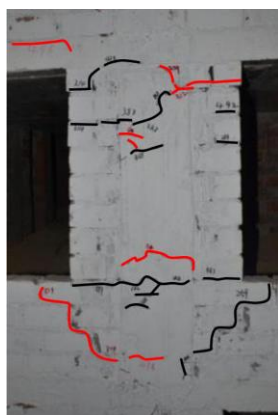


(b)



(c)

Figure 15. Crack pattern of the longitudinal wall and constructional column on the ground-level floor along D-grid line. (a) Wall limb and constructional column at 1-grid line. (b) Wall limb and constructional column at 2-grid line. (c) Wall limb and constructional column at 9-grid line.



(a) Wall limb and constructional column on 2-grid line



(b) Wall limb and constructional column at 8-grid line

Figure 16. V-shaped cracks on the longitudinal walls at the second floor along D-grid line.

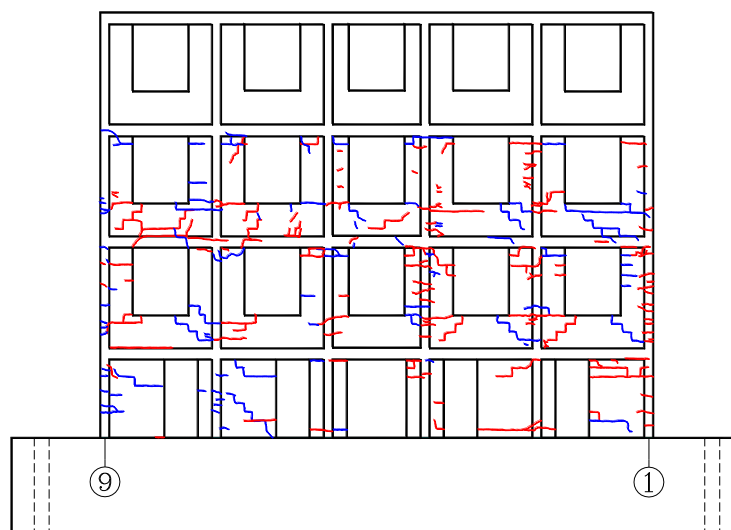


Figure 17. Crack distribution on the longitudinal wall and constructional column along D-grid line (Note: the red color represents the cracks generated by pushing force, and the blue color represents the cracks generated by pulling force).

4.3. Internal Longitudinal Wall

The crack pattern of the longitudinal walls and constructional columns along the B-grid line and C-grid line is shown in Figure 18 and Figure 19, respectively. The longitudinal walls of the B-grid line and C-grid line were arranged in an asymmetrical configuration, and the crack patterns exhibited by these axes were significantly divergent. The longitudinal wall of the B-grid line exhibited an incremental increase in the number of cracks, commencing from the base and culminating at the third floor. The longitudinal wall of the ground-level floor exhibited predominantly horizontal cracks, while the longitudinal walls of the second and third floors demonstrated stepped diagonal cracks. These diagonal cracks intersected with constructional columns in an X-shaped pattern, as illustrated in Figure 18. The development of stepped diagonal cracks was observed along the mortar joints and in the wall limbs at the location of the 1-grid line and 9-grid line on the bottom longitudinal wall of C-grid line. Furthermore, the horizontal cracks presented at the base

of the wall limbs at the 1-grid line and 9-grid line of the longitudinal wall on the second and third floors extended to the constructional columns, as illustrated in Figure 19.



Figure 18. Crack pattern of the longitudinal walls and constructional columns along B-grid line. (a) Wall limb and constructional column on the ground-level floor of 9-grid line. (b) Wall limb and constructional column on the third floor of 5-grid line.



Figure 19. Crack pattern of the longitudinal walls and constructional columns along C-grid line. (a) Wall limb and constructional column on the ground-level floor of 1-grid line. (b) Wall limb and constructional column on the second floor of 1-grid line.

4.4. External Transverse Wall

When the load was set to ± 1.8 mm, horizontal cracks became visible in the middle of the B-grid line and C-grid line constructional columns of the bottom transverse wall along the 1-grid line. Similarly, when the load was set to ± 3.4 mm, horizontal cracks appeared at the bottom of the transverse wall on the ground-level floor along the 1-grid line. Furthermore, it was observed that the number of horizontal cracks increased with increasing displacement. Cracks with a width of 2 mm appeared horizontally in the middle of the A-grid line and D-grid line constructional columns of the 1-grid line at the second floor, which were loaded to ± 5.2 mm. When loaded to ± 8.0 mm, the horizontal cracks continued to develop, and further cracks appeared in the middle of the 1-grid line second-floor transverse wall throughout the entire wall. When loaded to ± 10.4 mm, the original cracks in the transverse wall and constructional column of 1-grid line developed further. The loading process was completed at a magnitude of 12.0 mm. The transverse walls identified as the 1-grid line and 9-grid line were arranged in a symmetrical configuration, and the crack pattern exhibited symmetrical characteristics. Consequently, the development of cracks in the 9-grid line horizontal wall will not be further discussed in this analysis. The crack

pattern of the external transverse walls is shown in Figure 20. At the end of the test, the external transverse walls on both sides were most seriously damaged on the second floor, with dense penetrating cracks, while the first and third floors were less severely damaged, with sparser cracks.

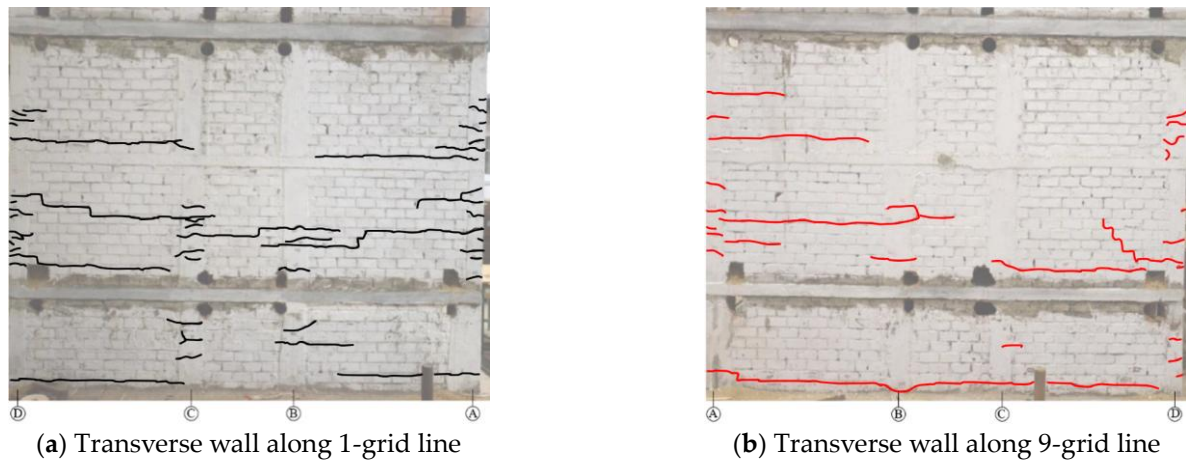


Figure 20. Crack pattern of the external transverse walls (Note: the black color represents the cracks generated by pushing force, and the red color represents the cracks generated by pulling force).

5. Experimental Result and Analysis

5.1. Strain

5.1.1. Strain of the Reinforcement in Frame Columns

The strain of the reinforcement in frame columns is shown in Figure 21, where positive values represent tensile strains and negative values represent compressive strains. As illustrated in Figure 21a, the application of thrust leads to the manifestation of compression in the upper region and tension in the lower region for KZ1-1 and KZ1-4. In contrast, KZ1-2 and KZ1-3 demonstrate a reverse configuration, with tension in the upper region and compression in the lower region. Tensile testing of the samples revealed that KZ1-1 and KZ1-4 exhibited tension in the upper part and compression in the lower part, while KZ1-2 and KZ1-3 demonstrated tension in both upper and lower parts. Furthermore, the frame column KZ1 exhibits higher stresses in the lower reinforcement and lower stresses in the upper reinforcement. This finding indicates that at the first-floor edge-span frame columns experience high bending moments in the lower part and low bending moments in the upper part. Additionally, the unilateral frame beams impose limited constraints on the edge-span frame columns.

As illustrated in Figure 21b,c, under the action of horizontal cyclic loading, the stresses in the lower and upper reinforcements of the middle frame column demonstrate closely analogous variation trends. It is evident that the upper reinforcement stress is marginally greater than the stress of the lower reinforcement. The finding suggests that the frame beams located on both sides of the middle frame column, the floor slabs, and the walls on the middle frame column collectively exert a restraining effect on the middle frame column.

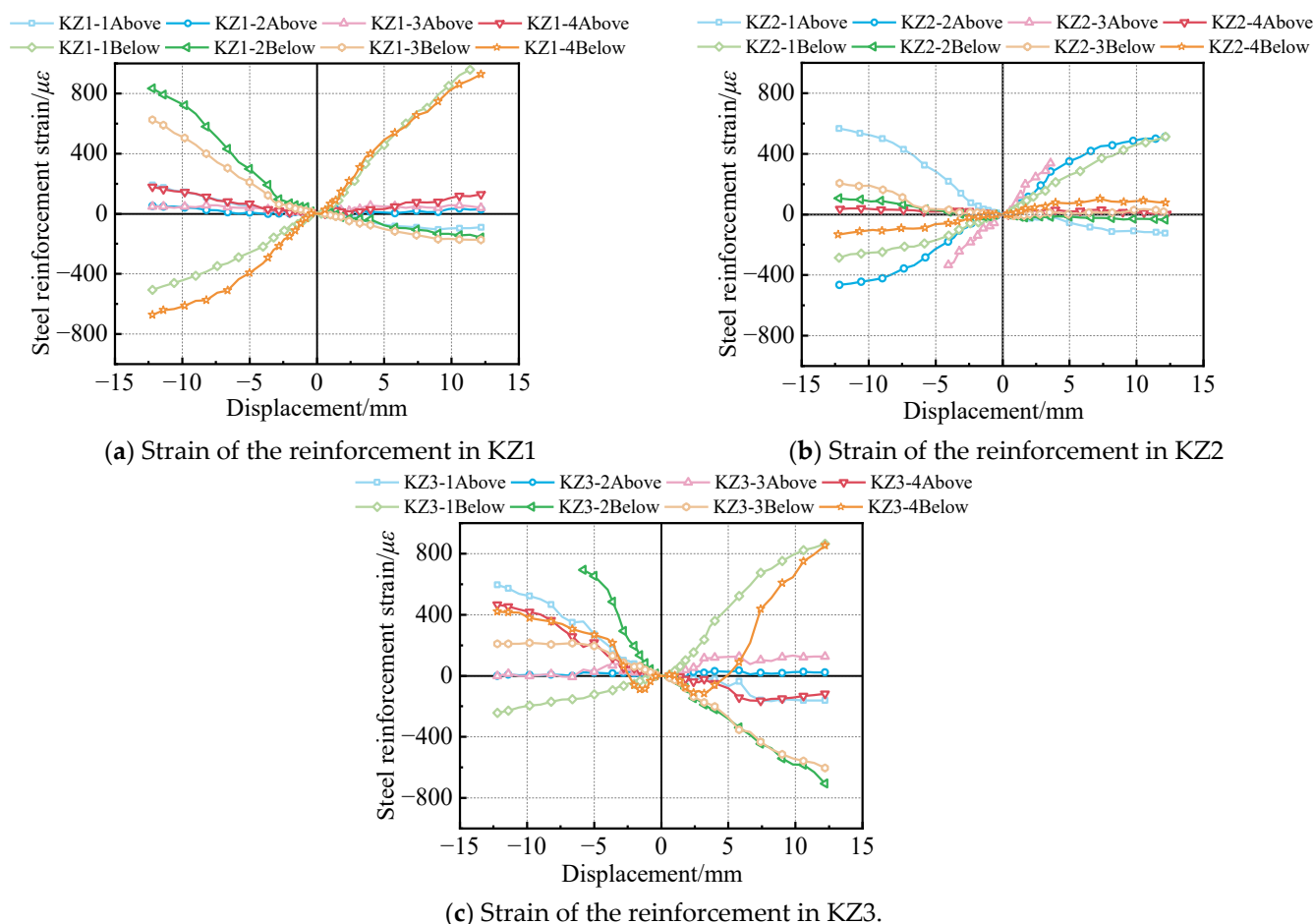
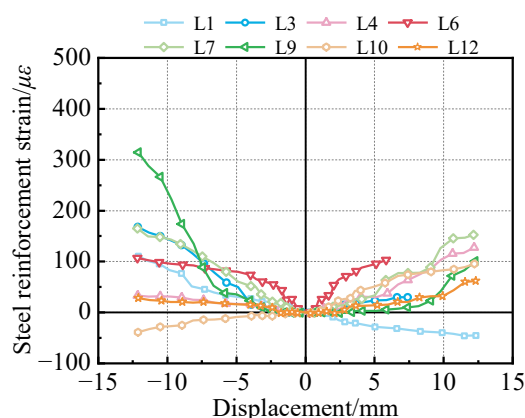


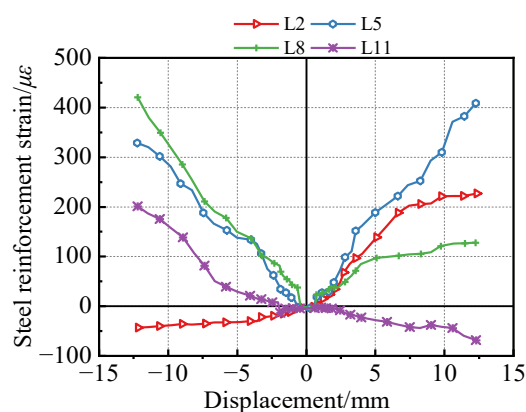
Figure 21. Strain of the reinforcement in frame columns.

5.1.2. Strain of the Reinforcement in Frame Beams

The strain of the reinforcement in frame beams is shown in Figure 22, where positive values represent tensile strains and negative values represent compressive strains. As shown in Figure 22a, it is evident that there is compression in the reinforcement at the support of L1 and L10 of the edge-span beam, and the negative moment reinforcement of the supports in the other spans is in tension under the cyclic loading. As shown in Figure 22b, it can be seen that at mid-span locations L2 and L11 of the edge-span beams, compressive strain develops in the reinforcement, though its magnitude remains relatively low. The reinforcement in the middle span of the other spans is in tension under cyclic loading and has higher strains. Under the action of horizontal cyclic loading, the frame beams undergo internal force redistribution due to restraint from frame columns, resulting in distinct internal force distributions compared to general frame beams.



(a) At the upper part of the support of the frame beams

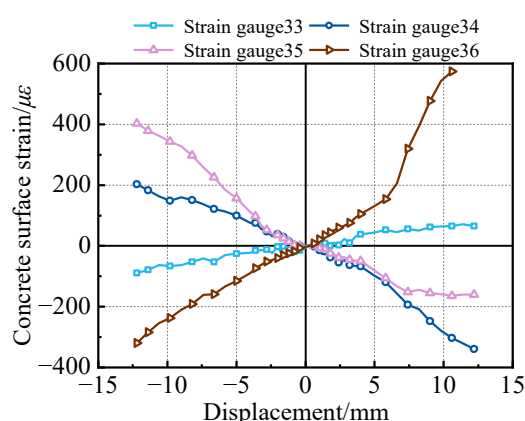


(b) At the bottom of the mid-span of the frame beams.

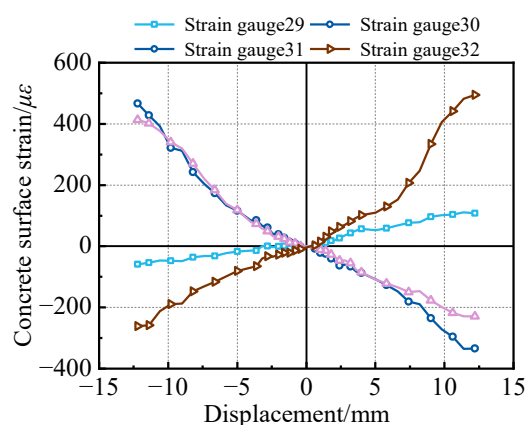
Figure 22. Strain of the reinforcement in frame beams.

5.1.3. Surface Strain of the Concrete in Frame Columns and Beams

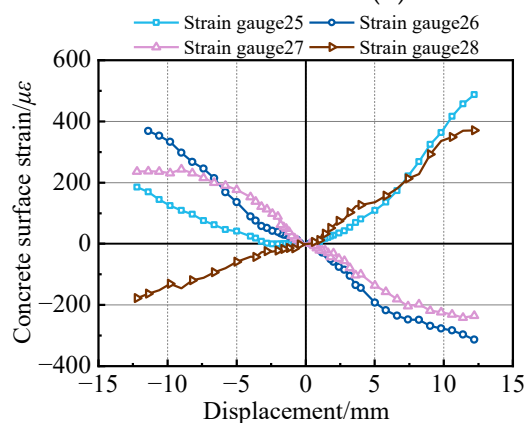
The surface strain of the concrete in frame columns is shown in Figure 23. It is evident that for frame columns KZ3 and KZ4, the surface strains on the upper and lower parts exhibit anti-symmetric behavior when the horizontal load is applied. This observation indicates that the distribution of the internal forces conforms to the stress pattern of frame columns with fixed nodes at both the top and bottom. For the boundary column KZ5, the outer strain at the upper end of the column is invariably a tensile strain, which is attributed to the presence of overall bending in the test model.



(a) Surface strain of the concrete in KZ3



(b) Surface strain of the concrete in KZ4



(c) Surface strain of the concrete in KZ5.

Figure 23. Surface strain of the concrete in frame columns.

The surface strain of the concrete in frame beams is shown in Figure 24. It is evident that there is tension and compression of the concrete at the support of the frame beam, which indicates that the frame beam exerts some restraint on the frame column under cyclic loading. This indicates that the concrete frame column and frame beam can work compositely. The concrete located at the center of the mid-span frame beam is invariably under tension. The concrete in the middle of the edge-span frame beam is under tensile and compressive stress, but the compressive stress is much smaller than the tensile stress. The stress pattern exhibited by the frame beams in this instance is distinctly different from that observed in typical frame beams. This deviation can be attributed to the fact that, under the effect of horizontal loading, the second-floor wall undergoes both shear and bending moments. The occurrence of horizontal or diagonal cracks in the wall results in a modification of the transfer path for the upper loads. The vertical load from each wall limb would be transmitted directly to the frame beam located at the upper part of the frame column. Concurrently, the bending moment generated by the wall limb due to the horizontal load would also result in the tensile force on the constructional column. It is noticeable that the bending moment would not be located at the node of the frame column or frame beam. Consequently, a positive bending moment would be generated at the center span of the beam.

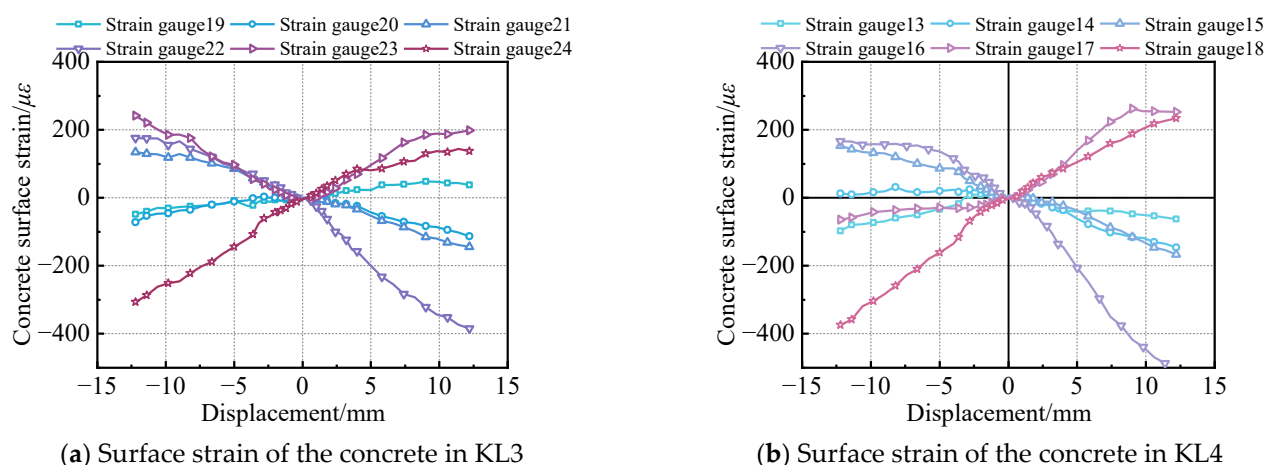


Figure 24. Surface strain of the concrete in frame beams.

5.1.4. Surface Strain in Masonry Walls

The surface strain of the masonry wall is shown in Figure 25. As illustrated in Figure 25, the strain values at the window corners changed faster and quit working earlier, which is consistent with the fact that the wall cracked first from the window corners during the test. The strain changes at the same location in each wall limb tend to be the same, from which it can be deduced that the force mode and damage form of each wall limb are the same.

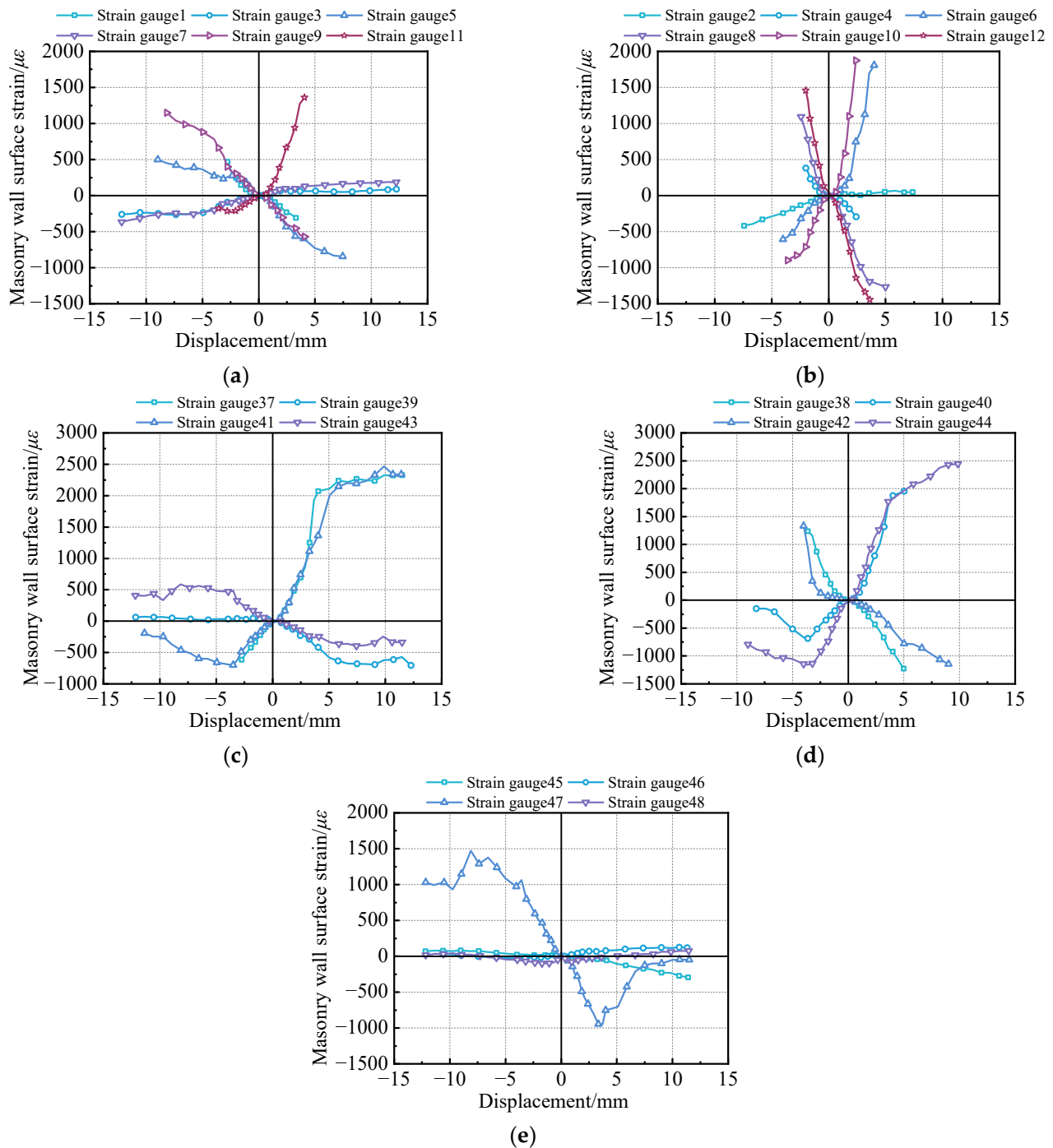


Figure 25. Surface strain of the masonry walls. (a) Surface strain at the window head corner of the masonry wall on the second floor of A-grid line. (b) Surface strain at the window sill corner of the masonry wall on the second floor of A-grid line. (c) Surface strain at the window head corner of the masonry wall on the second floor of D-grid line. (d) Surface strain at the window sill corner of the masonry wall on the second floor of D-grid line. (e) Surface strain at the window sill corner of the masonry wall on the first floor of D-grid line.

5.2. Displacement

The deformation characteristics of the structural model can be expressed in terms of lateral displacements of floors and inter-story displacement angles. The lateral displacement and inter-story displacement angle of each floor under different horizontal displacements are shown in Figure 26. When the applied horizontal displacement is minimal, the lateral displacement of each floor is distributed linearly, the inter-story displacement an-

gle between floors is comparatively slight, and the structural model undergoes overall bending deformation. With an increase in horizontal displacement, the longitudinal walls of each floor cracked, resulting in a decrease in lateral stiffness of the structure. The non-linear characteristics of lateral displacement of each floor became evident, and the inter-story displacement angle between floors increased. When loaded to 12.0 mm, the second story exhibits the most significant inter-story displacement angle of 0.0066, followed by the third story at 0.0059, and the ground-level floor demonstrating the least significant angle of 0.0024. Furthermore, the lateral stiffness of the first, second and third story is 866.0 kN/mm, 344.2 kN/mm and 359.3 kN/mm, respectively. This indicates a change in stiffness at the second story, and the second-story wall sustains the most severe damage under horizontal load. A comparison of the frame columns on the second floor with those on the ground-level floor reveals that the damage to the former is significantly more severe than that to the latter (Figure 26a). This is indicative of a structure with stiff lower stories and flexible upper stories.

The initial lateral stiffness of the ground-level floor is greater than that of the upper floors because the height of the ground-level floor is smaller than that of the upper floors. Subsequent to the yielding of the structure, the stiffness of the upper floors experiences a more rapid degradation due to brittle damage to the walls. Conversely, the ductility of the bottom equivalent reinforced concrete framed columns and beams is relatively adequate, and the degradation of the bottom lateral stiffness is relatively slow. It suggests that the lateral stiffness of the ground-level floor can be enhanced following the yielding of the structure by the equivalent reinforced concrete framed columns. These results indicate that the stiffness-equivalent replacement frame effectively mitigated the concentration of deformation and damage at the ground-level floor of the tested model. At an applied horizontal displacement of 12.0 mm, the inter-story drift ratio of the ground-level floor was 0.0024, which was considerably lower than those of the second and third stories, namely 0.0066 and 0.0059, respectively. Combined with the relatively minor damage observed in the ground-story frame columns, these results demonstrate that the risk of forming a weak or soft ground-level floor was effectively reduced.

In terms of torsion, according to GB 50011-2010 [7]: A structural torsional irregularity is considered to occur when the maximum elastic horizontal displacement (or inter-story displacement) of the floor under the specified horizontal force is 1.2 times greater than the average of the elastic horizontal displacements (or inter-story displacement) at both ends of the floor [35]. Define the value of displacement meter 2 (Figure 9) as δ_2 , the value of displacement meter 3 as δ_3 , and $\delta_{\max}$ as the larger of δ_2 and δ_3 . According to the requirements of the above specification, the torsional irregularity can be negligible for this model if $2\delta_{\max}/(\delta_2 + \delta_3) \leq 1.2$ is satisfied. Conduct torsional irregularity assessment at selected displacement-controlled loading points; the calculation results are shown in Table 5. It is clear that the displacement loading control points at the determination of the standard calculation results are used to meet the requirements. During the elastic phase, the structure did not exhibit torsion, indicating that although there is a large difference in material properties between reinforced concrete frames and masonry structures, the replacement method based on stiffness equivalence is feasible for solving the structural torsion problem caused by irregularities in the floor plan arrangement. As displacement increases, the degree of damage and stiffness degradation varies between floors. These result in an offset center of stiffness for the structure, leading to a small degree of torsion. However, the torsion remains below the limit values specified in Table 5, thus rendering the torsion effect negligible.

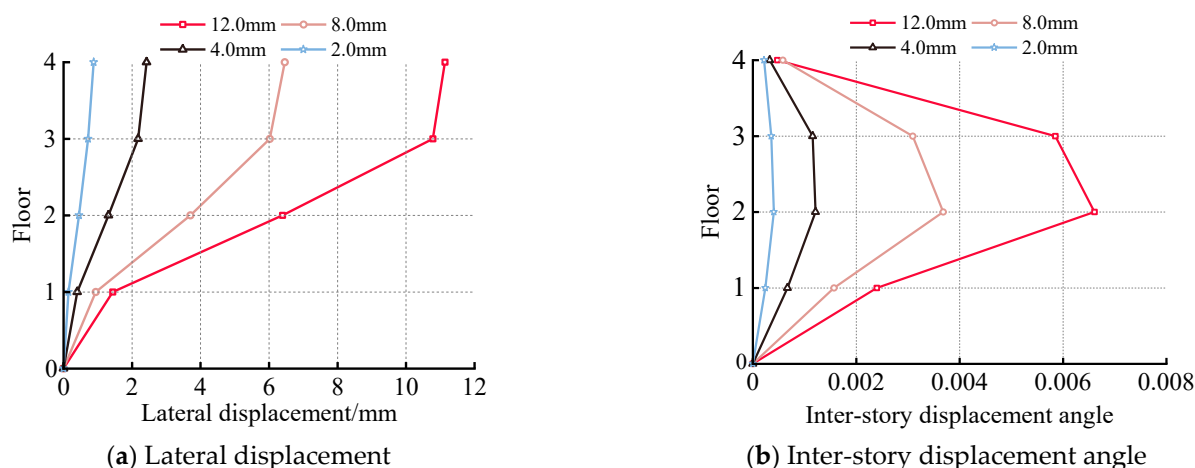


Figure 26. Lateral displacement and inter-story displacement angle of each floor under different horizontal displacements.

Table 5. Torsion irregularity criterion.

Criterion	Control Point for Displacement Loading/mm						
	0.4	0.6	0.8	1.8	2.8	6.4	12.0
$\frac{2\delta_{\max}}{(\delta_2 + \delta_3)}$	1.00	1.00	1.00	1.01	1.02	1.02	1.04

5.3. Hysteresis Curves and Skeleton Curves

The displacements used in the story-level hysteresis and skeleton curves were the relative inter-story displacements rather than the absolute lateral displacements of the floors. Specifically, the relative displacements of the second and third stories were calculated as $\Delta_2 = u_2 - u_1$ and $\Delta_3 = u_3 - u_2$, respectively, where u_1 , u_2 , and u_3 denote the measured lateral displacements at the tops of the first, second, and third stories, respectively. Therefore, the second-story displacement accounts for the lateral displacement at the top of the first story, while the third-story displacement accounts for the lateral displacement at the top of the second story. The hysteresis curve for each floor of the structural model is shown in Figure 27. As illustrated in Figure 27, the hysteresis curves for each floor are characterized by a significant two-stage process: During the initial loading phase, the hysteresis curve approximates a straight line passing through the origin, and the hysteresis loops exhibit a pinched shape with small enclosed areas. The model is not cracked and the floors are in the elastic phase. As the displacement increases, the hysteresis curve begins to bend, showing an inverted S-shape, and the hysteresis loops exhibit continuously expanding enclosed areas, progressively developing into fuller shapes. It indicates that the structure has entered the elastoplastic stage, with cracking occurring across structural members while simultaneously absorbing significant energy, and the crack develops rapidly with further increase in displacement.

Comparison of Figure 27a–c shows that compared with the hysteresis curves of the second and third floors, the hysteresis curves of the ground-level floor are more closely spaced, and the hysteresis loop area is smaller. The displacement of the ground-level floor is smaller than that of the second and third floors under the same loading. The hysteresis curves of the second and third floors exhibit a certain degree of the “pinching effect” phenomenon, with the hysteresis curve of the third floor revealing a fuller shape and a comparatively weaker degree of “pinching effect” in comparison to the second floor. The observation indicates that the third floor possesses a superior deformation capability and energy dissipation capacity in comparison to the second floor. Furthermore, an evident

asymmetry is observed in the two-floor hysteresis curve, attributable to the unequal dry friction effect during both forward and reverse loading. It results in asymmetric structural crack development and stiffness degradation [36]. From the asymmetry of the hysteresis curve, it is clear that the cracking and damage is more severe in the second floor of the structure.

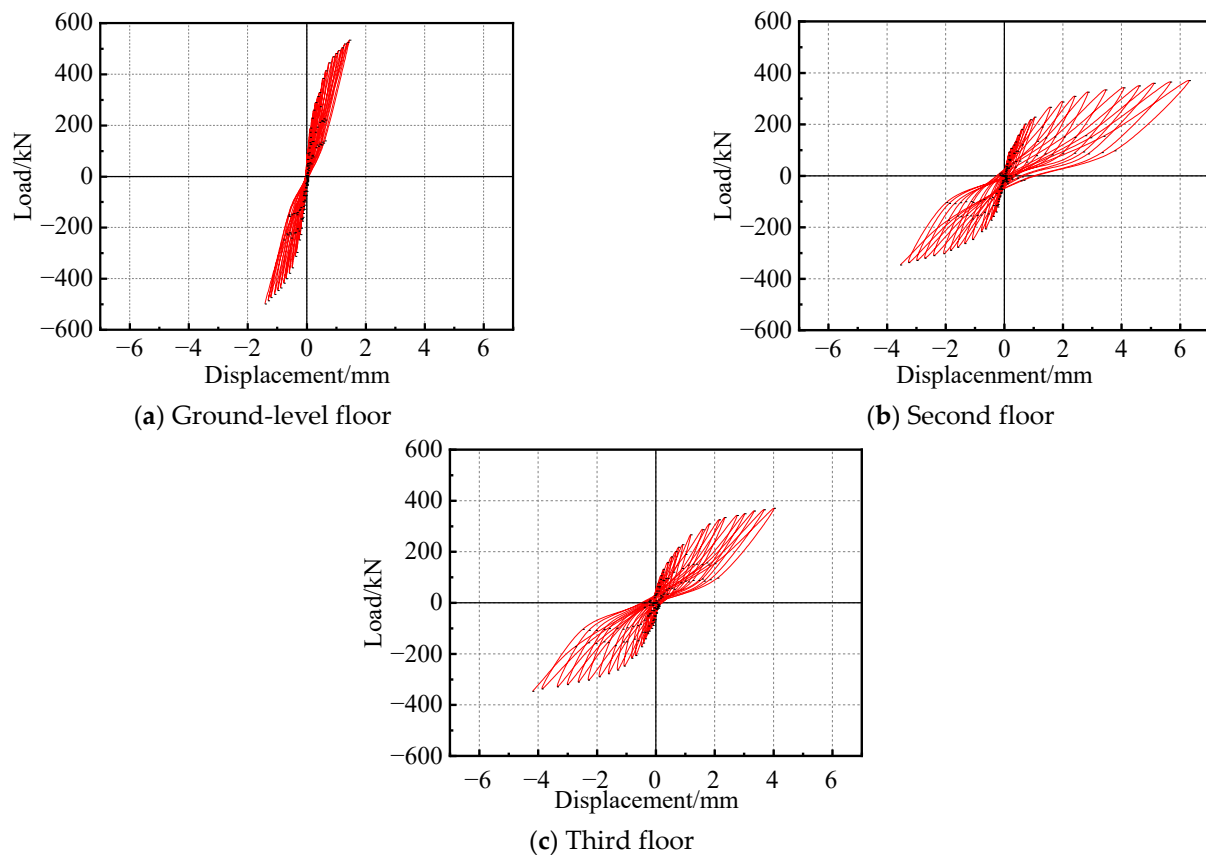


Figure 27. Hysteresis curve for each floor of the structural model.

A comparison of the skeleton curve for each floor of the structural model is shown in Figure 28. The load and displacement are basically linear during the initial loading phase, and the model is in the elastic stage. With increasing displacement, the slope of the skeleton curve decreases gradually, the curve plateaus, and the model enters the elastoplastic stage.

The skeleton curve of the ground-level floor is essentially a straight line with a slight slant, indicating that the damage to the bottom structure is minimal during the loading process, and the overall lateral stiffness of the ground-level floor remains relatively unchanged at the later stages of loading. The skeleton curves of the second and third floors show obvious “two-stage” characteristics—especially after the cracking of the wall—the overall lateral stiffness of the floor decreases significantly, and the skeleton curves gradually tend to be horizontal. Because the skeleton curves did not exhibit a clearly defined yield plateau, the yield point was determined independently using the “Drawing Method”, “Energy Method”, and “Park Method”. These methods identify the yield state from different perspectives, namely the change in curve slope, equivalent energy absorption, and reduced-stiffness-equivalent elastic–perfectly plastic idealization, respectively. The arithmetic mean of the three results was adopted as the representative yield load and yield displacement to reduce the method dependence and potential bias associated with any individual criterion. In the “Park Method”, the point corresponding to 75% of the maximum lateral load is first located on the ascending branch of the skeleton curve. The

secant line connecting this point to the origin is extended to intersect the horizontal line corresponding to the maximum lateral load. The displacement coordinate of this intersection is defined as the equivalent yield displacement. The yield load of the ground-level floor is 403.2 kN, while the yield loads of the second and third floors are 289.1 kN and 296.7 kN, respectively. Consequently, the yield load of the ground-level floor is about 138% of the second and third floors on average. The yield displacement of the ground-level floor of the model is 1.15 mm, while the yield displacements of the second and third floors are 1.70 mm and 2.25 mm, respectively. Consequently, the average yield displacement of the ground-level floor is about 59% of that of the second and third floors. The above results show that the ground-level floor still maintains sufficient bearing capacity and resistance to deformation after the structure enters the yielding stage.

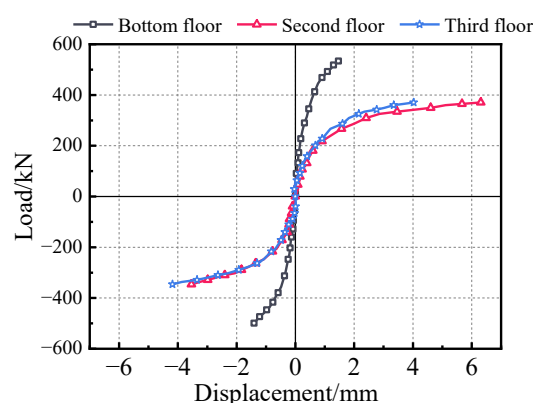


Figure 28. Comparison of the skeleton curve for each floor of the structural model.

5.4. Ductility

Ductility is a significant indicator of a structure's capacity to yield and deform under seismic loads. This capacity is typically quantified by the displacement ductility coefficient μ , calculated by Equation (5). A higher displacement ductility coefficient is indicative of greater plastic deformation capacity, thereby enabling the structure to dissipate more energy under seismic loads.

$$\mu = \frac{\Delta_u}{\Delta_y} \quad (5)$$

where Δ_u and Δ_y are the ultimate displacement and yield displacement, respectively.

The displacement characteristic points for each layer of the structural model and the displacement ductility calculation results are shown in Table 6.

Table 6. Comparison of the displacement characteristic points and ductility of each floor in the structural model.

Floor	Yield Displacement Δ_y/mm	Ultimate Displacement Δ_u/mm	Ductility Coefficient $\mu = \Delta_u / \Delta_y$
Ground-level floor	1.15	1.44	1.25
Second floor	1.70	6.40	3.76
Third floor	2.25	10.79	4.80

The displacement ductility coefficient of general masonry structures ranges from 2.16 to 7.80 [37]. As demonstrated in Table 6, the calculated displacement ductility coefficients for the second and third floors are within the normal range, while the ground-level floor exhibits a comparatively low displacement ductility coefficient due to not reaching the ultimate load-bearing capacity (or ultimate displacement). The ductility of the ground-

level floor of the masonry structure with large openings, where equivalent concrete frame columns replace part of the exterior walls, is similar to that of the conventional multi-story masonry structure.

5.5. Stiffness Degradation

In circumstances of low cyclic loads, the cumulative structural damage to the structure is the cause of degradation of its overall stiffness. The stiffness degradation of each floor of the structure reflects the degree of damage to the structure as a whole during the loading process, as well as the differences in the seismic performance of each floor. According to JGJ/T 101-2015 [38], the wall stiffness uses the secant stiffness. The formula is shown in Equation (6).

$$K_i = \frac{|+F_i| + |-F_i|}{|+X_i| + |-X_i|} \quad (6)$$

where $+F_i$ and $-F_i$ are the load values at the peak positive and negative points of the i th cyclic, respectively; and $+X_i$ and $-X_i$ are the displacement values at the peak positive and negative points of the i th cycle, respectively.

The stiffness degradation curve reflects the changing law of stiffness in the process of cracking, damage, and failure of the structure, as well as the difficulty of resisting deformation in each stage [39]. A comparison of the stiffness degradation curve for each floor of the structural model is shown in Figure 29. As illustrated in Figure 29, the stiffness degradation curves of each layer of the model are characterized by “two stages”. Firstly, after the structure crosses the elastic phase, new cracks continue to appear, leading to significant stiffness degradation in all floors and a steeply decreasing segment in the stiffness degradation curve. Secondly, after the structure enters the elastoplastic phase, the original cracks have been fully developed, the new cracks are reduced, and the stiffness degradation curves of each floor gradually plateau. The rate of stiffness degradation of the first and third floors is significantly greater than that of the second floor, since the loading locations were located in the first and third floors of the structural model, resulting in faster crack development in these two floors. Furthermore, the initial stiffness of the ground-level floor is considerably higher than that of the second and third floors. Consequently, despite the ground-level floor exhibiting a significantly faster stiffness degradation rate than that of the second and third floors, its residual stiffness in the late loading stage is still higher (Figure 29). This indicates that the ground-level floor exhibits greater resistance to deformation after horizontal reciprocating loading.

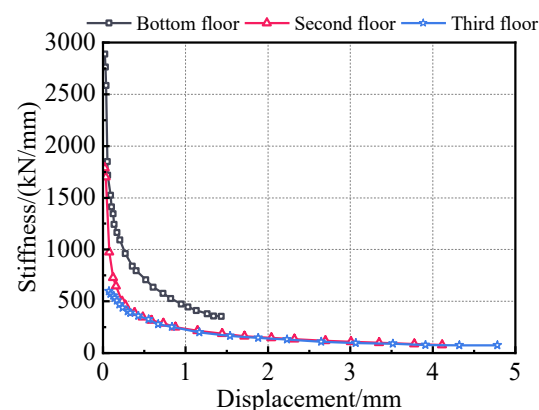


Figure 29. Comparison of the stiffness degradation curve for each floor of the structural model.

5.6. Energy Dissipation

The energy dissipation capacity of a structure is usually defined by the area of the hysteresis loop, characterizing the seismic performance of the structure in the elastic–plastic state [39]. In this study, the equivalent viscous damping ratio ζ_{eq} is used to express the energy dissipation capacity of each layer of the structural model. The schematic diagram for calculating the equivalent viscous damping ratio is shown in Figure 30, and the calculation formula is shown in Equation (7) [40].

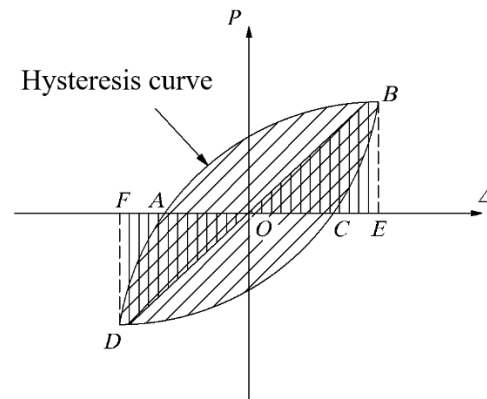


Figure 30. Schematic diagram for calculating equivalent viscous damping ratio (Note: B and D denote the positive and negative peak points of the hysteresis loop, respectively; A and C denote the intersections of the hysteresis loop with the displacement axis; E and F are the vertical projections of B and D onto the displacement axis, respectively; and O denotes the origin. The solid lines form the hysteresis loop, while the dashed lines BE and DF are auxiliary projection lines used to define the reference energy areas).

$$\zeta_{eq} = \frac{1}{2\pi} \times \frac{S_{ABCD}}{S_{(\triangle OBE + \triangle ODF)}} \quad (7)$$

where S_{ABCD} is the area enclosed by the hysteresis curve ABCD in Figure 30, and $S_{(\triangle OBE + \triangle ODF)}$ is the sum of the areas of triangle OBE and triangle ODF in Figure 30.

A comparison of the equivalent viscous damping ratios and energy dissipation for each floor of the structural model at different horizontal displacements is shown in Figure 31, where the energy dissipation is the hysteresis loop area of the first cycle of each displacement loading step. As shown in Figure 31a, when the horizontal displacement is minimal, the trend of the equivalent viscous damping ratio curves of the first and third floors is generally consistent with a decreasing trend, and the equivalent viscous damping ratios of the second floor show an increasing trend in the elastic phase. In circumstances where the horizontal displacement is substantial, the equivalent viscous damping ratio curves of all floors show a decreasing trend. As shown in Figure 31b, the energy dissipation in each floor of the model increases with displacement. In the early stages of loading (horizontal displacements up to 3 mm), the third floor consumes more energy than the first and second floors. It is evident that as the horizontal displacement exceeds 4 mm, there is a rapid rise in the energy consumption of the second floor. The increase in energy consumption is significantly greater than that observed on the first and third floors, which consume the most energy and result in the most substantial damage to the wall. Compared to the second and third floors, the ground-level floor consumes the least amount of energy due to the presence of equivalent reinforced concrete frame columns that allow for less deformation, crack development, and damage accumulation in the structural components of the ground-level floor.

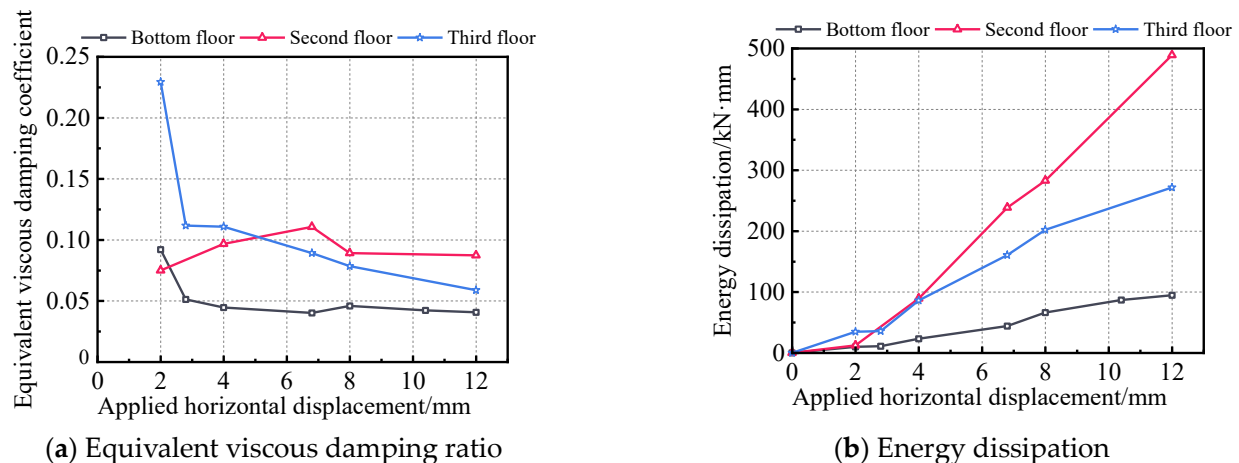


Figure 31. Comparison of the equivalent viscous damping ratio and energy dissipation for each floor of the structural model under different horizontal displacements.

5.7. Lateral Stiffness Ratio

The seismic performance of masonry structures based on the frame column and frame beam replacement is strongly influenced by the lateral stiffness ratio of the second floor to the ground-level floor. In this paper, the lateral stiffness ratio of the second floor and the ground-level floor is about 0.4. The phenomenon is indicative of an abrupt change in stiffness, which occurs in the second floor. Consequently, the structure forms a soft layer, and the damage to its second-floor wall is the most severe after the cessation of loading. This indicates that the stiffness-equivalent replacement frame mitigated the soft-story risk at the ground story; however, the relatively large stiffness difference between the replacement frame story and the adjacent masonry story shifted the major stiffness discontinuity to the second story. Han et al. [41] indicated that an increase in lateral stiffness ratio resulted in a gradual shift of the structural soft floor from the second masonry floor to the bottom frame floor. When the lateral stiffness ratio exceeded 1.25, the masonry structure predominantly relied on the bottom frame floor for energy consumption, while the upper masonry floor consumed less energy and exhibited a lighter degree of damage. Consequently, from the perspective of ductility, the design of the replaced masonry structure should prioritize the utilization of the bottom frame floor for energy dissipation. This is to be achieved by ensuring that the lateral stiffness ratio of the second floor to the ground-level floor does not fall below 1, thus averting the occurrence of brittle damage to the structure. In addition, in cases where it is not possible to change the height of the ground-level floor, it is necessary to take seismic reinforcement measures for the whole masonry floor of the second floor to avoid the formation of a seismically soft floor that could result in structural collapse.

6. Conclusions

In this paper, a four-story brick structure scale model was designed and produced according to the scale of 1:4. Based on the principle of stiffness equivalence, the reinforced concrete framed columns and framed beams were used to replace part of the external wall on the side of the large openings on the ground-level floor. The seismic performance test under low cyclic loading test was carried out, and the conclusions are as follows.

(1) The equivalent concrete frame columns replace the ground-floor large openings in the masonry structure of the exterior walls. The exterior longitudinal walls primarily exhibit horizontal cracks and shear diagonal cracks. Horizontal cracks are primarily distributed along the edge-bay structural columns, the upper portions of wall sections, and the intersections between window-to-window walls and walls below windows. Shear di-

agonal fractures are predominantly observed on the second and third stories of window-to-window walls and at window corners, forming X-shaped or V-shaped patterns. Cracks in the structural exterior transverse walls (gable walls) are primarily horizontal. The distribution of wall cracks is as follows: a concentration of cracks is observed on the second floor, specifically in the middle of the wall. In contrast, the first and third floors exhibit a distribution of wall cracks at the bottom of the wall.

(2) In the case of horizontal reciprocating loading, the structural bottom edge-span frame columns experience significant bending moments in the lower portion and comparatively negligible bending moments in the upper portion. Consequently, the restraining effect of the single-sided frame beams on the edge-span frame columns is found to be limited. The difference between the upper and lower bending moments of the middle frame columns on the ground-level floor is not significant. The middle frame columns are constrained by various elements, including the frame beams on both sides, the floor slabs, and the walls.

(3) Prior to the manifestation of structural cracking, the structure is in the elastic stage and no torsion has occurred. As the degree of horizontal displacement increases, the varying degrees of damage and stiffness degradation across different layers cause the stiffness center of the structure to shift, resulting in some torsion. However, this torsion remains below the limits specified by the standards. Despite the notable disparities in material properties inherent to reinforced concrete frames and masonry structures, the displacement method, founded upon stiffness equivalence, has been demonstrated to be a viable approach for addressing structural torsion issues arising from irregular planar layouts.

(4) The hysteresis curves of each floor manifest a reverse S-shape. It is evident that among these floors, the second floor exhibits the largest hysteresis loop area and presents a higher degree of asymmetry, manifesting in more severe cracking and damage. The average yield load of the ground-level floor of the stiffness-equivalent frame columns is approximately 138% of that of the second and third floors, and the average yield displacement is approximately 59% of that of the second and third floors. The finding suggests that, upon entering the yield stage, the ground-level floor maintains its load-bearing capacity and resistance to deformation.

(5) The ductility of the ground-level floor large-opening masonry structure, with equivalent concrete frame columns replacing part of the exterior walls, is similar to that of ordinary multi-story masonry structures. It has been illustrated that the energy dissipation of each structural layer increases in proportion to the displacement load. A comparison of the ground-level floor with the second and third floors reveals that the former experiences the lowest energy dissipation, while the latter experiences the highest energy dissipation and exhibits a sudden change in stiffness, resulting in the most severe wall damage. In order to prevent the formation of a seismic weak layer that could lead to structural collapse, it is advisable to reinforce the second floor of the structure.

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