Seismic behavior of perforated brick walls with HPFL strips composite ring beam and constructional column

Lei Min¹, Shang Shou-ping²

¹ College of Civil Engineering, Central South University of Forestry and Technology, Changsha, Hunan, 410004, China
² College of Civil Engineering, Hunan University, Changsha, Hunan, 410082, China

*Corresponding author’s e-mail: 26788829@qq.com

Abstract. The existing perforated brick masonry structures are usually lack of ring beam and constructional column. Aiming at this problem, a new method of HPFL strips composite ring beam and constructional column is proposed. Six perforated brick walls with HPFL strips composite ring beam and constructional column and two comparative walls are tested under low cycle reversed loading. The failure mode, ultimate strength, energy dissipation capacity, and ductility of the strengthened walls and the comparative walls are compared. A calculation method is proposed for predicting the seismic shear capacity of perforated brick walls with HPFL strips composite ring beam and constructional column. The results show that using this method, the cracking load, ultimate load, energy dissipation capacity and deformation capacity are improved greatly. The calculated results are in good agreement with the measured results. The calculated method can provide the reference for seismic strengthening design of the perforated brick masonry.

1. Introduction

Perforated bricks have the advantages of energy saving, land saving, lightweight, good thermal insulation performance, etc., and it is widely used in rural areas of China. However, the performance of perforated bricks is more brittle than clay bricks, and most of the building structures in rural areas use self-built masonry structures without formal design, construction and supervision. Because of the general lack of seismic fortification measures, such structures are the most severely damaged engineering groups in the earthquake, causing the largest number of casualties [1].

The research shows that under the action of horizontal earthquakes, the masonry structure of the ring beam and the constructional column is improved, and the cracking load and shear capacity are improved. At the same time, due to the pulling action of the ring beams on the constructional columns at both ends, a weak frame is formed, which improves the seismic and collapse resistance of the wall [2-4]. The addition of concrete ring beams and constructional columns to perforated brick masonry has problems such as high cost, difficult construction, and large impact on the indoor and outdoor of the original building. Professor Shang Shouping from Hunan University proposed that the method of combining the HPFL (high-performance Ferro-cement laminate) strips with the original masonry to form the ring beam and the constructional column, and carried out a series of experimental studies [5-7]. In order to study the seismic performance of the perforated brick wall of the HPFL strips composite ring beam structure column, the pseudo-static tests of six perforated brick walls with HPFL strips composite ring beam constructional column and two contrast walls are carried out.
2. Specimens design and test plan

2.1. Specimen design

In the test, considering the difference in the number of longitudinal reinforcement of the composite constructional columns, 8 walls were designed and manufactured, of which 2 were unconstrained walls and 6 were walled with HPFL strips combined with ring beams. All specimens are 2000mm × 1500mm × 240mm (width × height × thickness). The masonry is made of KP1 type perforated clay brick with a size of 240 mm × 115 mm × 90 mm and the strength of brick is MU10. Masonry mortar strength is M2.5. C30 concrete top and bottom beams are placed above and below the wall for load application and wall fixing. The high-performance composite mortar has a strength grade of M40. The HPFL strips are arranged horizontally by placing on the upper edges of the two sides of the wall near the concrete top beam and forming a composite ring beam with the original wall body. The HPFL strips are vertically arranged on the left and right ends of the two sides of the wall and combined with the original wall to form a constructional column. The HPFL strip has a width of 240 mm and a thickness of 25 mm. The steel mesh is fixed to the wall by L-shaped shear pin, and the steel bar is made of cold-rolled ribbed steel with a diameter of φ6, tensile yield strength of 590 MPa, and a horizontal distribution of steel bar spacing 200mm. The specimen parameters are shown in Table 1.

Table 1. The specimens parameters.

| Specimen group | Specimen number | Actual strengthen of brick (MPa) | Actual strengthen of masonry mortar(MPa) | Actual strengthen of high-performance composite mortar (MPa) | The size of the ring beam and constructional column (mm×mm) | Longitudinal bar number in one constructional column |
|----------------|----------------|--------------------------------|----------------------------------------|------------------------------------------------|------------------------------------------------|------------------------------------------------|
| JG0-0-0        | 2              | 6.99                          | 2.01                                   | —                                               | —                                               | —                                               |
| JG1-2-2        | 2              | 6.99                          | 1.95                                   | 44.13                                          | 240×290                                         | 4Φ6                                             |
| JG1-3-2        | 2              | 6.99                          | 2.06                                   | 41.57                                          | 240×290                                         | 6Φ6                                             |
| JG1-4-2        | 2              | 6.99                          | 1.95                                   | 42.36                                          | 240×290                                         | 8Φ6                                             |

2.2. Loading device and loading scheme

First, the vertical pressure is applied by the jack and the distribution beam, σ0 = 0.3 MPa. The vertical load is applied once to the specimen destroyed. The horizontal load is applied to the top beam step by step through a PLU-300 electro-hydraulic servo pulsation fatigue tester. Before the wall is cracked, it is loaded with a force of 10kN for one cycle. After the wall is cracked, the tests are controlled by deflection increments. The multiple of the displacement Δcr is incremented by the number of millimeters. According to the stability of the deformation, each cycle is repeated 1 or 2 times until the
specimen is broken. Dynamic displacement sensors are arranged in the middle of the top beam, in the middle of the wall and at the bottom of the wall. The test loading device is shown in Figure 2.

Figure 2. Test loading device.

3. Test results and analysis

3.1. Destruction process
From zero loads up to the destruction loads, the specimens experienced three stages of elasticity, elastic-plasticity, and failure. The failure modes of the specimens are shown in Figure 3.

The initial cracks of the specimen JG0-0-0 appear at the horizontal mortar joint between the first layer and the second layer bricks on the left edge of the wall. As the horizontal reciprocating loading increases, new cracks appear continuously, and the initial cracks extend upward, gradually forming a plurality of step-like cracks. When the load reaches more than 85% of the ultimate bearing capacity, the two main step-like cracks expand to form the X-shaped critical cracks. The load continues to increase to the ultimate load, the crack develops rapidly, and the wall deformation increases sharply.
After that, the displacement further increases, and the load gradually reduces until the oblique crack suddenly penetrates the wall. The main oblique crack width is more than 10 mm, and the bearing capacity reduces to less than 85% of the ultimate load. Under the reciprocating loading, the specimen undergoes shear failure in the form of X-shaped critical oblique crack.

The initial cracks of the specimen JG1-2-2 appear at the horizontal mortar joint between the third layer and the fourth layer bricks on the left constructional columns of the wall. As the horizontal load increases, a plurality of horizontal cracks gradually appear at a spacing of about 2 layers bricks on the constructional column. The four corners of the wall begin to have oblique cracks, and oblique cracks extend toward the middle of the wall. Then, there are several horizontal cracks in the middle of the wall. The load continues to increase, and the oblique crack at the corner is connected with the horizontal crack in the middle to form a critical crack of “>—<” shape. When the ultimate load is reached, the crack develops rapidly. With the displacement increased continuously, the horizontal load decreases. Due to the restraining effect of the composite ring beam and the constructional column, the cracks in the middle part of the wall continue to increase, and the mortar ash at the horizontal mortar joint of the critical crack is falling off, and the wall is destroyed.

The horizontal crack of the specimen JG1-3-2 firstly appears at the horizontal mortar joint between the second layer and the third layer bricks on the left constructional columns of the wall. As the load increases, the process of cracks appearance and development, the form of the critical crack and the failure mode are similar to the specimen JG1-2-2.

The first crack of the specimen JG1-4-2 appears on the left side of the composite constructional column, corresponding to the horizontal mortar joint position between the first layer and the second layer bricks. As the horizontal load increases, a plurality of horizontal cracks gradually appear at a spacing of about 1 layer bricks along with the wall height at the constructional column. At the same time, oblique cracks appear in the middle of the wall and extend upwards and downwards. The load continues to increase and oblique cracks appear at the four corners of the wall. Different from JG1-2-2 and JG1-3-2, there is no horizontal crack in the middle of the wall, but an X-shaped critical oblique crack is formed through the wall when the ultimate load is reached. After that, the displacement is further increased. Due to a large number of longitudinal reinforcement in the combined constructional column, the restraining effect of the ring beam and the constructional column is obviously better than that of the previous specimen. The main crack does not expand sharply, but there are more and more diagonal cracks in the corners and middle of the wall, the much more the number of cracks than the previous specimen. When the displacement is reached, the brick in the middle of the wall is crushed and the specimen is destroyed.

3.2. Cracking load and Ultimate load

The test results of the specimen cracking load $V_c$ and the ultimate load $V_u$ are shown in Table 2.

Compared with the specimen group JG0-0-0, the cracking load increase range of the specimen group JG1-2-2, JG1-3-2 and JG1-4-2 with the composite ring beam and constructional column is 46.7%—53.3%. The increase of the cracking load is independent of the longitudinal reinforcement ratio in the composite constructional column and mainly depends on the synergistic shearing effect of the high-strength composite mortar strip.

The ultimate loads of the specimen groups JG1-2-2, JG1-3-2 and JG1-4-2 are 24%, 39.2%, and 49.4% higher than the ultimate load of the specimen group JG0-0-0. As the number of longitudinal reinforcement in the composite constructional column increases, the increase in the ultimate load increases.

| Specimen group | Specimen serial number | $V_c$(kN) | Average of $V_c$(kN) | Increased range (%) | $V_u$(kN) | Average of $V_u$(kN) | Increased range (%) |
|----------------|------------------------|-----------|----------------------|---------------------|-----------|---------------------|---------------------|
| JG0-0-0        | 1                      | 70        | 75                   | -                   | 102.85    | 106.53              | -                   |
|                | 2                      | 80        |                      |                     | 115.20    |                     |                     |
| JG1-2-2        | 1                      | 110       | 110                  | 46.7                | 125.60    | 132.05              | 24.0                |

Table 2. The crack load and shear capacity.
3.3. Hysteretic energy consumption

The hysteretic energy performance of the wall can be reflected by the hysteresis curve, and the hysteresis curve of each specimen group is shown in Fig. 4. The hysteresis loop of the specimen JG1-0-0 is long and narrow before cracking and is approximately anti-S-shaped after cracking. The shape of the hysteresis loop is not full, and the performance of absorbing seismic energy is poor. The hysteresis loop is arched and the enclosed area is significantly larger than JG1-0-0 after the specimen JG1-2-2 is cracked. It shows that after adding the composite ring beam and constructional column, the energy consumption performance of the wall is obviously improved. After the specimens JG1-3-2 and JG1-4-2 are cracked, the hysteresis loop is arcuate. Compared to JG1-2-2, the hysteresis loop is fuller. It shows that the energy consumption performance of the wall is better as the number of longitudinal reinforcements in the composite construction column increases.

The quantitative analysis of the energy consumption of the specimen is measured by the equivalent viscous damping coefficient $h_e$ [8]. Table 3 shows the equivalent viscous damping coefficients $h_{eu}$ and $h_{ef}$ for each specimen in the limit and failure state. It can be seen from Table 3 that the $h_{eu}$ of the specimen JG1-2-2 is increased by 45.8%, and the $h_{ef}$ of the specimen JG1-2-2 is reduced by 16.3%, compared with JG0-0-0. In general, the energy consumption of the composite ring beam and the constructional column against the wall is increased. The $h_{eu}$ and $h_{ef}$ of the specimen JG1-3-2 are 79.2% and 2.7% higher than JG0-0-0, and the $h_{eu}$ and $h_{ef}$ of the specimen JG1-4-2 are improved by 114.6% and 19.1% compared with JG0-0-0. It is shown that increasing the number of longitudinal reinforcement in the composite constructional column increases the energy consumption of the wall significantly.

| Specimen | $P_k$ (kN) | $\Delta$ (mm) | Energy Consumption |
|----------|------------|---------------|--------------------|
| JG1-3-2 | 110 | 53.3 | 138.50 |
|          | 120 |        | 144.50 |
|          | 115 |        | 152.00 |
| JG1-4-2 | 110 | 46.7 | 148.25 |
|          | 110 |        | 159.20 |
|          |    |        | 39.2    |
|          |    |        | 49.4    |

Figure 4. Hysteretic curves of the specimens.
capacity decreases gradually, and the deformation capacity gradually increases. The decline in bearing capacity and the lowest deformation capacity after the ultimate load. With the longitudinal stiffness and $JG_1$ parameters are shown in Table 5, and the normalized skeleton curve is shown in Figure 5(b).

The horizontal load and displacement are the failure point of the wall (the simulation. The characteristic points are normalized by taking the yield point, the limit point and the model composed of an elastic deformation ability of the wall.

Increasing the longitudinal reinforcement ratio of the composite constructional column can improve the wall deformability. Comparing $JG_1-2-2$, $JG_1-3-2$ and $JG_1-4-2$, $JG_0$, $JG_1$, $JG_2/JG_1$, and $JG_3/JG_1$ increase correspondingly as the number of longitudinal reinforcement in the composite constructional column increases, where $A_1/A_0$ increase is 12%, 38.7%, and 42.7%. Increasing the longitudinal reinforcement ratio of the composite constructional column improves the deformation ability of the wall.

### Table 3. Equivalent viscous damping coefficients.

| Specimen group | $h_{eq}$ | Increased range (%) | $h_{ct}$ | Increased range (%) |
|----------------|--------|---------------------|--------|---------------------|
| JG0-0-0        | 0.048  | —                   | 0.147  | —                   |
| JG1-2-2        | 0.070  | 45.8                | 0.123  | -16.3              |
| JG1-3-2        | 0.086  | 79.2                | 0.151  | 2.7                 |
| JG1-4-2        | 0.103  | 114.6               | 0.175  | 19.1                |

### 3.4. Deformability

To evaluate the deformability of a structure or component, the displacement ductility coefficient, the skeleton curve and the normalized skeleton curve in the test results are generally used [9].

Table 4 lists the average horizontal displacements $A_c$, $A_u$, $A_t$ of the test group corresponding to cracking, ultimate and failure loads, and the displacement ductility coefficient $A_d/A_c$ from the cracking load to the ultimate load, and displacement ductility coefficient $A_t/A_u$ from the ultimate load to the failure load. It can be seen that the $A_c$, $A_u$, $A_t$, $A_d/A_c$, and $A_t/A_u$ of the specimen $JG_1-2-2$ are improved compared with the comparative specimen $JG_0-0-0$. Adding the composite ring beam and the constructional column can improve the wall deformability. Comparing $JG_1-2-2$, $JG_1-3-2$ and $JG_1-4-2$, $A_c$, $A_u$, $A_t$, $A_d/A_c$, and $A_t/A_u$ increase correspondingly as the number of longitudinal reinforcement in the composite constructional column increases, where $A_t/A_u$ increase is 12%, 38.7%, and 42.7%. Increasing the longitudinal reinforcement ratio of the composite constructional column improves the deformation ability of the wall.

### Table 4. Ductility coefficients.

| Specimen group | $A_c$ | $A_u$ | $A_t$ | $A_d/A_c$ | $A_t/A_u$ | Increased range of $A_d/A_u$ (%) |
|----------------|------|------|------|-----------|-----------|----------------------------------|
| JG0-0-0        | 3.60 | 8.20 | 12.30| 2.28      | 1.50      | -                                 |
| JG1-2-2        | 4.75 | 9.65 | 16.20| 2.03      | 1.68      | 12.0                              |
| JG1-3-2        | 4.75 | 10.80| 22.50| 2.27      | 2.08      | 38.7                              |
| JG1-4-2        | 4.55 | 11.50| 24.65| 2.53      | 2.14      | 42.7                              |

The skeleton curve of the specimen is shown in Figure 5(a). It can be seen that the perforated brick masonry wall has obvious strengthening stage and softening stage after yielding. As a result, a trilinear model composed of an elastic stage, a strengthening stage, and a softening stage can be used for simulation. The characteristic points are normalized by taking the yield point, the limit point and the failure point of the wall (the failure load takes 85% of the ultimate load), and the corresponding horizontal load and displacement are the feature points. The established normalized skeleton curve parameters are shown in Table 5, and the normalized skeleton curve is shown in Figure 5(b).

Compared with the specimen $JG0-0-0$, the elastic stiffness $K_0$ of the specimen $JG1-2-2$, $JG1-3-2$ and $JG1-4-2$ is increased from 1.59 to 1.69, 1.77 and 1.73. At the same time, the strengthening stiffness $K_1$ of the specimen $JG1-2-2$, $JG1-3-2$ and $JG1-4-2$ are reduced. But as the number of longitudinal reinforcement of the constructional column increases, $K_1$ increases. The softening stiffness $K_2$ of the specimen $JG0-0-0$ is the smallest, indicating that the specimen has the fastest decline in bearing capacity and the lowest deformation capacity after the ultimate load. With the increase of the longitudinal reinforcement ratio in the constructional column, the softening stiffness $K_2$ of the specimen $JG1-2-2$, $JG1-3-2$ and $JG1-4-2$ is reduced from -0.22 to -0.14, -0.13, the bearing capacity decreases gradually, and the deformation capacity gradually increases.

### Table 5. Parameters of the normalized skeleton curves.

| Specimen group | Yield point | Limit point | Failure point | Stiffness of each stage |
|----------------|-------------|-------------|---------------|-------------------------|
|                | $P_\text{/P}_u$ | $\Delta f/\Delta f_u$ | $P_\text{/P}_u$ | $\Delta f/\Delta f_u$ | $P_\text{/P}_u$ | $\Delta f/\Delta f_u$ | $K_0$ | $K_1$ | $K_2$ |
| JG0-0-0        | 0.70        | 0.44        | 1.0           | 1.0                     | 0.85          | 1.50                      | 1.59  | 0.54  | -0.30 |
| JG1-2-2        | 0.83        | 0.49        | 1.0           | 1.0                     | 0.85          | 1.68                      | 1.69  | 0.33  | -0.22 |
| JG1-3-2        | 0.78        | 0.44        | 1.0           | 1.0                     | 0.85          | 2.08                      | 1.77  | 0.39  | -0.14 |
| JG1-4-2        | 0.69        | 0.40        | 1.0           | 1.0                     | 0.85          | 2.14                      | 1.73  | 0.52  | -0.13 |
4. Shear bearing capacity calculation formula

When the brick wall is broken, its working diagram is shown in Figure 6. The lateral bearing capacity of the wall is provided by the friction of the brickwork and the shear capacity of the longitudinal reinforcement in the composite constructional column.

\[
V_u = (f_{vm} + \alpha \sigma_0)A_w + n \beta f_{ym} A_s
\]  

(1)

Where \( f_{vm} \) is the average value of the masonry shear strength; \( f_{ym} \) is the average tensile strength of the longitudinal reinforcement; \( \sigma_0 \) is the average compressive stress of the wall section; \( A_w \) is the brick masonry area; \( n \) is the number of composite constructional columns; \( A_s \) is the area of the longitudinal reinforcement in one constructional columns; \( \alpha \) is the correction coefficient, the perforated brick masonry is taken as 0.15; \( \beta \) is the reliability coefficient of the composite construction column and the masonry working together, and is taken as 0.27.

Table 6 lists the ultimate load \( V_u \) test value and the results calculated according to formula (1). It can be seen that the two agree well.

| Specimen group | \( V_c \) (kN) | \( V_u \) (kN) | \( V_u^s \) (kN) | \( V_u^s/V_u \) |
|----------------|-----------|----------|-------------|-------------|
| JG0-0-0        | 75        | 106.53   | 106.56      | 1.000       |
| JG1-2-2        | 110       | 132.05   | 132.25      | 1.002       |
| JG1-3-2        | 115       | 148.30   | 147.79      | 0.997       |
| JG1-4-2        | 110       | 159.20   | 159.10      | 0.999       |
5. Conclusion
Through the pseudo-static test of six perforated brick walls with HPFL strips composite ring beam constructional column and two contrast walls, the whole process from cracking to failure is observed, and the cracking load, cracking displacement, ultimate load and limit are obtained. The test results of displacement, failure displacement, and hysteresis curve are analyzed and compared with the bearing capacity, energy dissipation capacity and deformation capacity of the specimen. The results show that the perforated brick wall with the composite ring beam constructional column has the higher shear capacity, energy dissipation performance and deformation capacity than the wall without the composite ring beam constructional column. Before the wall is cracked, the high-performance composite mortar strips contribute a lot, and cooperate with the wall; after the wall is cracked, the longitudinal reinforcement in the constructional column play a role. As the number of longitudinal reinforcement increases, the shear capacity, energy dissipation and deformation capacity of the wall increase. The addition of the HPFL strips combination ring beam constructional column is an effective method to improve the seismic performance of perforated brick masonry. On this basis, the calculation formula of shear wall capacity of the perforated brick wall of HPFL strips composite ring beam constructional column is proposed. The calculated results are in good agreement with the experimental results.

Acknowledgements
This research was financially supported by Natural Science Foundation of Hunan Province (Grant No. 2015JJ3172), the Research Foundation of Education Bureau of Hunan Province (Grant No. 16C1664), and Central South University of Forestry and Technology (Grant No. 2013RJ011).

References
[1] Lu, M., Xing, J.H., Yu, T., et al. (2013) The investigation and analysis of the damage to non-engineered buildings in Lushan earthquake with Ms7.0. Journal of Earthquake Engineering and Engineering Vibration, 33(6): 131–137.
[2] Dong, J.C., Wang, Y.W., Song, X.Z.. (1991) Experimental Study on Seismic Behaviour of Model KP1 Perforated Brick Masonry Buildings. Journal of Building Structures, 12(3): 34–43.
[3] Chen, B.W., Tang, C., Wu, Y.F., et al. (2016) Experimental studies on seismic behavior of autoclaved fly ash perforated brick walls with constructional columns. Earthquake Engineering and Engineering dynamics, 36(3):116–125.
[4] Wu, Y.T., Li, Y.M., Liu, L.P., et al. (2012) Earthquake-Induced Collapse Prevention Capacity of Brick Masonry Structures Confined with Tie Columns and Tie Beams. Industrial Construction, 42(3):25–32.
[5] Shang, S.P., Tang, W.H.. (2012) Experimental investigation on seismic behavior of perforated brick masonry strengthened with HPFL. Earthquake Engineering and Engineering dynamics, 32(5):73–80.
[6] Shang, S.P., Tang, Y.X., Li, L.. (2014) Experimental investigation on seismic behavior of HPFL-brick composite ring beam and constructional column. Earthquake Engineering and Engineering dynamics, 34(3):105–110.
[7] Shang, S.P., Li, L., Tang, Y.X. (2015) Investigation on Aseismic Performance of HPFL-brick Composite Constructional Concrete Column and Ring Beam. Journal of Disaster Prevention and Mitigation Engineering, 35(6): 822–827.
[8] MOHURD. (2015) JGJ/T 101-2015 Specification for the seismic test of buildings. China Architecture & Building Press, Beijing.
[9] Li, Y.M, Zheng, N.N, Xia H.L., et al. (2010) Pseudo-Static Test on Seismic Deformation Capacity of Masonry Wall Constrained by Core-Tie-Columns. Journal of Civil, Architectural& Environmental Engineering, 32(4):1–6.