Investigation on Punching Shear Capacity of Hollow Slab-column Connection

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Abstract: To investigate the punching shear capacity of cast-in-situ reinforced concrete hollow slab-column connections under the unbalanced bending moments, a series of specimens were tested and analyzed in this paper. Hollow ratio, direction of moment, tube layout pattern and other factors are considered in the experiment. Performance of the specimens was evaluated in terms of cracking load, ultimate bearing capacity, fracture distribution and load-displacement curve. Moreover, the punching shear capacity formula of the hollow slab-column connections was provided. The experimental results show that the failure patterns of the specimens are punching failure; stiffness improved with the increase of thickness. Under one-direction unbalanced moment, the punching shear capacity decreases with the increase of hollow ratio; Under bi-direction unbalanced bending moment, the punching shear capacity increases with the increase in hollow ratio; the punching shear capacity is higher when bending moment along the tube compared to bending moment cross the tube, but it is not obvious; the specimens under bi-direction unbalanced moment have lower punching shear capacity and better deformation ability compared with specimens under one-direction unbalanced moment. Using the proposed punching shear capacity formula, the calculated values are in good agreement with measurements.

1. Information
In the slab-column system, the shear stress and normal stress at the connections are highly concentrated, and the connections are prone to punching shear failure. If designed improperly, the structure will undergo brittle and interlocking damage resulting in disastrous consequences. A large number of experimental studies and theoretical analyses of solid slab-column have been conducted by scholars from all over the world. Stasio[1] and Hawkins[2] have proposed a mathematical model for the slab-column connection under the combined action of vertical shear and unbalanced moments. Hawkins derived the calculation formula of bearing capacity for shear failure and bending failure. The theoretical value is in good agreement with the experimental value, but the formula is very complicated. Park[3][4] and Morrison[5] et al. studied slab-column connections under vertical shear and unbalanced bending moments by experiment. The results show that the ability of the connection under horizontal loads is not significantly improved by increasing the reinforcement of slab. Lu Xilin[6] et al. conducted a pseudo-static test study on six slab-column connections and deduced the calculation
method of the punched bearing capacity of connections under unbalanced bending moments using the plastic limit method.

Hollow slab-column structure is a type of structure using hollow slabs in the traditional slab-column structure. It is a new type of slab-column structure, which can save concrete, reduce the height of the slab, and has good thermal and sound insulation properties. It is more widely used in new projects than before. At present, there are few researches on punching shear performance of reinforced concrete hollow slabs in the world, and their stress performance and design theory are not yet clear. Therefore, the calculation formula of the punched bearing capacity of the hollow slab-column connection is proposed, which provides a basis for engineering application and theoretical research of this type of connection. It is an urgent problem to be studied at present.

In this paper, the hollow slab-column connections under different conditions are taken as the research object, and the ultimate bearing capacity, crack distribution, load-displacement curve and other factors of the connections under the effect of vertical loads and unbalanced moments are studied. The calculation formula of the punched bearing capacity of the hollow slab-column connection is proposed, which provides a basis for engineering application and theoretical research of this type of connection.

2. Experiment introduction

2.1 Specimen design

In this paper, the hollow slab-column connections are taken as the research object, and six specimens were designed and manufactured. The specimen is a cut-off body taken from the anti-bend line of the middle connection of a practical project, and is 1/2 of the original size considering the load factor of the laboratory. The size of the model loading column is 300 mm×300 mm×700 mm. The board size is 1400 mm×1400 mm×100 mm (125 mm), the main steel of the column is 8C16, the main steel of slab is C8@100 (double-layer bidirectional), and the concrete design strength grade is C30. The cross-section of the specimen is shown from Fig. 1 to Fig. 3, and the main steel is shown in Fig. 2. The detailed parameters of each specimen are shown in Tab. 1. The tensile strength of the ribs was 520 MPa, the ultimate strength was 607.5 MPa, and the compressive strength of concrete cubes was fc 32.22 MPa, and the axial tensile strength ft was 2.11 MPa.

![Fig. 1 Cross-section of the specimen](image1)
![Fig. 2 column reinforcement figure](image2)

(a) Test piece UM1          (b) Test piece UM2          (c) Test piece UM3
Tab. 1 Hollow plate column connection test piece parameters

| Specimen number | Thickness /mm | Pipe diameter /mm | Hollow ratio% | Board Rebar  | Bending moment direction | Line spacing /mm |
|-----------------|---------------|-------------------|---------------|--------------|--------------------------|------------------|
| UM1             | 100           | 50                | 26.2          | C8@100       | Administrative           | 25               |
| UM2             | 100           | 50                | 26.2          | C8@100       | Cross tube              | 25               |
| UM3             | 100           | 50                | 26.2          | C8@100       | Bidirectional           | 25               |
| UM4             | 125           | 75                | 35.3          | C8@100       | Administrative           | 25               |
| UM5             | 125           | 75                | 35.3          | C8@100       | Cross tube              | 25               |
| UM6             | 125           | 75                | 35.3          | C8@100       | Bidirectional           | 25               |

2.2 Test device

The test constraints are simply supported on four sides. The fixtures are rigidly framed and supported by 1m high steel buttresses that are reliably connected to the laboratory's rigid ground. Four sides of the support plate are set to prevent local damage to the slab, and the plate is overhead, which facilitates the observation of cracks at the bottom of the plate, loading device scene picture and schematic diagram shown in Figure 4. The manual jack is placed in the center of the loaded column head to simulate the vertical punching load, and the hydraulic jack is placed horizontally to simulate the horizontal load.
2.3 Loading plan and test content

After considering the loading conditions of the laboratory and referring to the relevant literature [7]–[9], we formulated the following loading system.

The test loader is divided into two stages: preloading and formal loading. Pre-loading is performed with 3 stages, each stage takes 10kN, and then grading is unloaded. Load horizontally at the time of formal loading, take 10kN per level, add it to 40kN and keep it unchanged. Then add a vertical load, take 10kN per stage, load until the component is destroyed. The test is carried out for 10 minutes after loading to facilitate observation and tracing of cracks. In the loading system, the value of the horizontal load is based on the bending moment of 21 kN·m, which is applied to the hollow slab-column connection during normal use.

One displacement meter is arranged at the center of the bottom surface of the board to measure the deflection of the bottom. Another displacement meter is arranged at the four corners of the board to measure the warping change of the four corners of the test board. Strain gauges were placed on the critical parts of the slab ribs to measure the deformation of the steel bars. Concrete strain gages were placed at a position 1 times the slab thickness from the top of the column to measure the concrete deformation around the column.

3. Test results

3.1 Destructive form

When the vertical load is applied to the cracking load, the slenderest crack that appears at the edge of the bottom edge of the slab is 45° to the edge of the slab. With the increase of the vertical load, the crack gradually develops from the sides of the column to the edge of the slab, and along the edge. The cracks in the direction of the pipe are also increasing, and the ring-shaped cracks are clearly developed during the cracking of the concrete. When the vertical load is applied to the yield load, the bottom of the plate gradually assumes a radial crack that radiates toward the edge of the plate along the edge of one side, a crack in the direction along the pipe, and a circumferential crack along one side of the column. The initial occurrence of cracks in the direction of the pipe is also becoming wider. When the vertical load is loaded to the ultimate load, the vertical load decreases, one side of the column head sinks, and the specimen is damaged. It can be known from the test phenomenon that due to the effect of monotonic horizontal load, the punching cone of the specimen is obviously asymmetric, and the damage of the specimen is a brittle punching failure. The damage pattern is shown in Figure 5.

Fig. 5 Crack development and damage morphology

3.2 Load-displacement curve

As can be seen from Fig. 6, when the load is small, the load-displacement curve increases linearly, and the test piece is basically in the elastic working phase. As the vertical load continues to increase, the load-displacement curve begins to increase nonlinearly and the specimen enters the elasto-plastic working phase. When the load reaches the ultimate load, the load suddenly drops and the displacement continues to increase. When the load falls to 85% of the ultimate load, the specimen is destroyed. The
destruction of all test specimens is brittle punching shear failure, and its failure mode is similar to that of solid plates. The main load parameters of the test are shown in Tab. 2.

| Specimen number | Thickness (mm) | Cracking load /kN | Ultimate load /kN | Destructive form |
|-----------------|----------------|-------------------|-------------------|-----------------|
| UM 1            | 100            | 30                | 120               | Punching damage |
| UM 2            | 100            | 20                | 110               | Punching damage |
| UM 3            | 100            | 10                | 80                | Punching damage |
| UM 4            | 125            | 40                | 100               | Punching damage |
| UM 5            | 125            | 50                | 100               | Punching damage |
| UM 6            | 125            | 10                | 90                | Punching damage |

4. Analysis of test results

4.1 Thickness

The thickness of test piece UM1, UM2, UM3 is 100mm. The thickness of UM4, UM5, UM6 plate is 125mm. The diameter of hollow tube is 50mm and 75mm respectively. The thickness of solid area in the upper and lower layer are both 25mm, and the hollow ratio are 26.2% and 35.3% respectively. The test results show that when the plate thickness is increased from 100mm to 125mm, the rigidity of the test piece is significantly increased, and the ultimate displacement is reduced. The test pieces UM1 to UM3 are 2 times, 1.3 times and 3.4 times the UM4 to UM6 of the test piece, respectively. Comparing specimens UM1 with UM4 and comparing specimens UM2 with UM5, the ultimate bearing capacity of the connections in the hollow floor deck column decreases with the increase of the hollow ratio under the effect of one-way unbalanced bending moment. UM2 and UM5 decreased by 16.7% and 9.1%, respectively. For the specimens UM3 and UM6, the ultimate bearing capacity of the connections in the hollow floor slab increases with the increase of the hollow ratio under the bi-directional unbalanced bending moment, and the ultimate bearing capacity of UM6 increases by 12.5%.

4.2 The direction of the bending moment and the relative position of the pipe

UM1 and UM2, UM4 and UM5 are two identical test pieces, only the direction of the horizontal load is different. For UM1 and UM4, the horizontal load acts in the direction of the pipe. For UM2 and
UM5, the horizontal load acts in the direction of the horizontal pipe. As can be seen from Fig. 7, the ultimate bearing capacity of the specimens UM1 and UM4 is 1.09 and 1 times that of the specimens UM2 and UM5, respectively. When the unbalanced bending moment is small, the punched bearing capacity and the horizontal load of the connections in the hollow floor plate are not significantly related to the transverse pipe or the straight pipe. In the design, if the man-made direction is determined to be the main direction of force, the direction of the transverse pipe is determined as the direction of the secondary force, and even if the reinforcement is configured only in the direction according to the structure, the hollow floor will produce a crack along the pipe direction. When determining the arrangement direction of the hollow core tube cores, there is no need to have special requirement for the adjacent grid cores to be arranged perpendicular to each other, but the cores can be arranged in one direction according to the convenience of construction.

4.3 Bidirectional unbalanced bending moment

For the specimens UM1 to UM3 and UM4 to UM6, the construction of these two sets of specimens is identical, in which the UM1 and UM2 and the UM4 and UM5 are applied with unbalanced bending moments in one direction (coordination and cross tubes), UM3 and UM6 apply bidirectional unbalanced bending moments. As can be seen from Fig. 7, the ultimate bearing capacities of UM1 and UM2 are 1.5 and 1.38 times that of UM3, respectively, the ultimate bearing capacity of UM4 and UM5 is 1.11 times that of UM6. However, the displacements of UM3 and UM6 at the time of destruction are large, and the deformation capacity is increased by about 32% to 70% compared with the specimens subjected to unidirectional unbalanced moments. Therefore, under the combined effect of a vertical load and a bidirectional unbalanced bending moment, the connection's ultimate bearing capacity has a low safety margin, and the connection is prone to punching damage.

5. Method for calculating the ultimate bearing capacity of a connection

Referring to the theoretical analysis of the plastic hinge, a method for calculating the ultimate bearing capacity of a cast-in-place hollow floor connection is deduced by applying the linear variation method to the analysis of the shear stress. The linear variation of shear stress assumes that the shear stress at a certain point around the critical section changes linearly with the distance from the centroid axis, and is caused by the shear force and partially unbalanced moments. The rest of the unbalanced bending moment is assumed by the bending of the plate.

According to study [10~12], the ratio of the bending moment generated by the shear force at the failure surface to the total moment \( \gamma \) is given by:

\[
\gamma_x = 1 - \frac{1}{1 + (2/3)\sqrt{I_y/I_x}}
\]

\[
\gamma_y = 1 - \frac{1}{1 + (2/3)\sqrt{I_x/I_y}}
\]

In this formula, \( I_x \) and \( I_y \) are the critical section lengths along the x and y directions, respectively. In this experiment, the testing square column has \( l_x = l_y \), therefore \( \gamma_x = 0.4 \).

When under the unbalanced moments \( \gamma_M \) delivered by punching, the entire punching failure surface can be linearly distributed according to its distance from the axis of the column. The error it brings is negligible. In order to simplify the calculation, the punching failure surface is simplified as a rectangle, and the position of the critical section is assumed to be at \( h_0/2 \) from the column edge. As shown in Fig. 7 and Fig. 8. Considering that bending moments may act in both directions, horizontal and vertical, the two situations are discussed separately.
5.1 Bending moment acting in the direction

As shown in Fig. 7, the moment of inertia of the small ribs between hollow tubes is,

\[ I_1 = \frac{1}{12} (s_i + 0.1d)\left(\frac{h_0}{2} - s_i\right)^3 \]  

The ribs perpendicular to the hollow tube and the moment of inertia of the core area of the column is,

\[ I_2 = \frac{1}{12} (c + 2s_i)(c + 2s_i)^3 = \frac{1}{12} (c + 2s_i)^4 \]  

The moment of inertia of the entire section is,

\[ I_3 = 2n I_1 + I_2 = 2 \cdot \frac{n}{12} (s_i + 0.1d)\left(\frac{h_0}{2} - s_i\right)^3 + \frac{1}{12} (c + 2s_i)^4 \]  

The stress of the slab edge is,

\[ \sigma = \frac{\gamma_v M}{W} = \frac{c + h_0}{n (s_i + 0.1d)\left(\frac{h_0}{2} - s_i\right)^3 + \frac{1}{6} (c + 2s_i)^4} \gamma_v M \]  

\( \gamma_v \)—The ratio of the bending moment generated by the shear force on the failure surface to the total bending moment is 0.4;

\( c \)—Column length;

\( s_i \)—Clearance between hollow tubes;

\( M \)—Bending moment due to external load;

\( h_0 \)—The effective height of the plate section.

It can be obtained:

\[ \nu_M = \sigma = \alpha \gamma_v M \]  

\[ \alpha = \frac{c + h_0}{n (s_i + 0.1d)\left(\frac{h_0}{2} - s_i\right)^3 + \frac{1}{24} (c + 2s_i)^4} \]  

When the shear stress on the punched out connection of the hollow slab column connection exceeds the strength limit, it can be used as a criterion for the punching failure of the hollow slab connection. That is, when the sum of the maximum shear stress \( \nu_M \) generated by the unbalanced bending moment and the shear stress \( \nu_p \) generated by the vertical load reaches the shear strength, or when the maximum shear stress \( \nu_{\text{max}} \) formed on the punching failure surface reaches the shear strength, the punching failure will occur.
Refer to 《Code for Design of Concrete Structures》 of China, it indicates that the equivalent $V_{eq}$ is generated under the equivalent vertical load $P_{eq}$. When the $V_{eq}$ is equal to the actual $V_{max}$, punching damage occurs, i.e.,

$$V_{eq} = V_{max} = u_y + u_M$$  \hspace{1cm} (9)$$

among them:
- $V_{eq}$ — Equivalent shear stress produced by equivalent load $P_{eq}$ on the punching failure surface;
- $u_y$ — Shear stress due to vertical load $P$ on the punched failure surface;
- $u_M$ — The maximum shear stress caused by the unbalanced bending moment $M$ at the punching failure surface;
- $V_{max}$ — The maximum shear stress on the punching failure surface under the combined action of $P$ and $M$.

$$P_{eq} = P + \alpha(u_y h - \sum \frac{\pi d^2}{4} - \sum l d) \gamma \gamma M$$  \hspace{1cm} (10)$$

So you can get shear stress due to vertical load $P$ on the punched failure surface among them is,

$$u_y = 4(c + h_y)$$  \hspace{1cm} \text{among them: } d — \text{Hollow tube diameter};$$
- $h_y$ — Hollow plate thickness.

Among references [7], The punching shear bearing capacity of hollow slab column connection under vertical load is,

$$F_p = 0.9l(f,(24 \rho + 0.72)(\sigma_y h - \sum \frac{\pi d^2}{4} - \sum l d)$$  \hspace{1cm} (11)$$

With reference to our country's regulations, regardless of the influence of the ratio of reinforcement reinforcement, the load-bearing capacity of hollow-core slab connections under vertical load and unbalanced bending moment along the pipe direction is,

$$P = f,(u_y h - \sum \frac{\pi d^2}{4} - \sum l d) - \alpha(u_y h - \sum \frac{\pi d^2}{4} - \sum l d) \gamma \gamma M$$  \hspace{1cm} (12)$$

5.2 The moment acts in the transverse direction

Similarly, as shown in Fig. 4-5, the moment of inertia of the small ribs between hollow tubes is,

$$I_1 = \frac{1}{12} \left( \frac{h_y}{2} - s_i \right) (s_i + 0.1d)^3$$  \hspace{1cm} (13)$$

The ribs perpendicular to the hollow tube and the moment of inertia of the core area of the column is,

$$I_2 = \frac{1}{12} (c + 2s_i)(c + 2s_i)^3 = \frac{1}{12}(c + 2s_i)^4$$  \hspace{1cm} (14)$$

The moment of inertia of the entire section is,

$$I = 2nI_1 + I_2 = \frac{n}{6} \left( \frac{h_y}{2} - s_i \right) (s_i + 0.1d)^3 + \frac{1}{12}(c + 2s_i)^4$$  \hspace{1cm} (15)$$

The most unfavorable is,

$$\sigma = \frac{\gamma \gamma \gamma M}{W} = \frac{c + h_y}{\frac{1}{3} \left( \frac{h_y}{2} - s_i \right) (s_i + 0.1d)^3 + \frac{1}{6}(c + 2s_i)^4} \gamma \gamma \gamma M$$  \hspace{1cm} (16)$$

So you can get,
\[ v_{sl} = \beta \gamma_s M \]  

among them,

\[ \beta = \frac{c + h_0}{\frac{n}{3} \left( \frac{h_0}{2} - s_i \right) (s_i + 0.1d)^3 + \frac{1}{6} (c + 2s_i)^4} \]

\[ P_{e0} = P + \beta (u_n h - \sum \frac{\pi d^3}{4} - \sum l_d \gamma_s) M \]  

Thus, the impact-resisting bearing capacity of the hollow slab-column connection under the combined action of vertical load and unbalanced bending moment along the transverse pipe is,

\[ F_i = f_i (u_n h - \sum \frac{\pi d^3}{4} - \sum l_d) - \gamma_s (u_n h - \sum \frac{\pi d^3}{4} - \sum l_d) \gamma_s M \]  

among them,

\[ u_n = 4 (c + h_0) \]

The parameters of the test piece are brought into formula (12) and formula (19), respectively, and the theoretical calculation value of the punching shear bearing capacity of the cast-in-place hollow floor deck column connection can be obtained. Since the above formula does not consider the influence of the bidirectional unbalanced bending moment, the following calculation results do not include the test pieces UM3 and UM6. From Tab. 3, the theoretically calculated values of the test pieces are all less than the test values, the error is between 10.5% and 18%, and the ultimate bearing capacity of the connections is not related to the direction of the action of the unbalanced bending moments. The theoretically calculated value is in good agreement with the experimental value, and the theoretical value is smaller than the experimental value. It has a certain safety margin. This calculation method of the ultimate bearing capacity of the connection can be used for future design reference.

| Specimen number | Test value /kN | Calculated /kN | error (%) |
|-----------------|----------------|----------------|-----------|
| UM1             | 120            | 98.4           | 18.0      |
| UM2             | 110            | 98.4           | 10.5      |
| UM4             | 100            | 88.8           | 11.2      |
| UM5             | 100            | 88.8           | 11.2      |

6. Conclusions

(1) The failure modes of cast-in-situ hollow floor connection columns under the action of unbalanced bending moments are punching shear failure, and the failure mode of connections is similar to that of solid plates.

(2) When the plate thickness increases, the stiffness of the test piece increases significantly. Under the effect of one-way unbalanced bending moment, the ultimate bearing capacity of the connections of a hollow floor plate decreases as the hollow ratio increases; Under the bi-directional unbalanced bending moment, the ultimate bearing capacity of the hollow floor connections increases as the hollow ratio increases.

(3) Compared with the unbalanced bending moment in the direction of the transverse tube, the ultimate bearing capacity of the unbalanced bending moment in the direction of the coplanar tube is greater, but the improvement is not obvious. The ultimate bearing capacity of the connection under the action of unidirectional unbalanced bending moment is higher, which is 1.11 to 1.50 times that of the connection where the one-way unbalanced bending moment is applied.

(4) Based on the calculation formula of the ultimate bearing capacity of solid slab connections, the ultimate bearing capacity calculation of the connections of cast-in-situ hollow floor slabs considering the effect of vertical load and one-way unbalanced bending moment was deduced. The theoretical
calculations are in good agreement with the experimental values and provide a reference for engineering applications and theoretical analysis of such connections.

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References
[1] Di Stasio, M. R. V. Buren. Transfer of Bending Moment between Flat Plate Floor and Column[J]. ACI Journal Proceedings.1960,57(3): 221-240.
[2] W. G. Corley, N. M. Hawkins. Shearhead Reinforcement for Slabs[J]. ACI Journal,1968,65(10): 1129-1138.
[3] S. Islam, R. Park. Test on Slab-Column Connections with Shear and Unbalanced Flexure[J]. ASCE, 1976, 102(3): 549-568.
[4] R. Park, S. Islam. Strength of Slab-Column Connections with Shear and Unbalanced Flexure[J]. ASCE, 1976,102(9).
[5] D. G .Morrison. Lateral-Load Tests of RC Slab-Column Connections[J]. Journal of Structural Engineering, ASCE, 1983,109(11): 2698-2714
[6] Ma Yunchang, Lu Xilin. Research on Seismic Behavior of Reinforced Concrete Slab Connections [J]. Journal of Architectural Structure. 2001, 22(4): 49-54.
[7] Zhu Liang. Experimental study on punching performance of cast-in-place hollow floor connections [D]. Nanjing: School of Civil Engineering, Southeast University, 2012.
[8] Shen Tao. Experimental Study on Mechanical Behavior of Reinforced Concrete Slab Connections [D]. Nanjing: Southeast University, 2002
[9] Yang Zhen, Lu Xilin. Analysis and Study on the Boundary Point of Concrete Slab Column under the Combined Action of Vertical Shear and Side Unbalanced Moments [J]. structural engineer, 1998, (4).
[10] Elgabry,A.,Ghali,A.,Moment Transger by Shear in Slab-Column Connections,ACI Structural Journa l,1,1996.2(pp.187-196)
[11] Elgabry, A.,et al, Transfer of Moment between Columns and Slabs: Proposed Code Revisions. ACI Structural Journal,1996.1(pp.56~61)
[12] Megally,S.,et al, Design Considerations for Slab-Column Connections in Seismic Zones, ACI Structural Journal,1994.3(pp.303~314)