Experimental Research and Analysis on Seismic Behavior of Eccentric RC Beam-Column Joint

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Abstract. Four types of beam-column joints with eccentricity larger than one-fourth of the column width are investigated and analyzed experimentally. The stress concentration and potential cracks on the column are reflected by Finite-element analysis (FEA) and experimental research. This study presents the guidelines for the determination of the effective joint width and construction detail. We suggest design methods regarding the evaluation of the shear strength of the eccentric beam-column joint with haunch beam and construction requirements.

1. Introduction
In a reinforced concrete (RC) Frame structure, architects usually lean toward keeping the outer face of the beam flush with that of the column, leading to a large eccentricity between the beam and the column, especially in tall buildings with large size columns. Earthquake lessons had demonstrated the failure of beam-column joints with large eccentricity. The maximum eccentricity between the beam axis and column axis allowed by China’s current seismic design code is 1/4 of the column width. This research aims to determine the effective width of the joint core in connection with the shear strength and construction detail of the eccentric beam-column joint. Similar experimental studies have been conducted for a beam-column joint with haunch beams. The use of a haunch beam in effectively improving the seismic behavior of a beam-column joint with large eccentricity has been proven by research. Finally, design recommendations are suggested.

2. Program of experiment
2.1. Test specimens
Four types of specimens have been designed and tested. The specimens are divided into two groups: group 1 consists of PZD-1 and PZD-2, while group 2 contains PZD-3 and PZD-4. Group 1 specimens have an eccentricity of 3/8 of the column width. PZD-1 and PZD-2 are identical, with the exception that PZD-2 contains a slab with a thickness of 60 mm. Figure 1 shows the dimensions of the joints in group 1, while Figures 2 and 3 present the details of reinforcement of the beam and column. In group 2, haunch beams are used, and the eccentricity of the joints is also equal to 3/8 of the column width. The length and width of the haunch are 900 mm and 300 mm, respectively. Test load corresponding to the shear strength of the PZD-3 joint core is higher than the test load corresponding to the beam’s flexural strength, and that of the PZD-4 joint core is lower than the flexural strength of the beam. Figure 4 illustrates the dimensions of the group 2 joints in the plan, while Figures 5 and 6 depict the reinforcement of group 2 joints in detail. The reinforcement in the haunch beam is shown in Figure 7.
Figure 1. The plane of the beam-column joints.

Figure 2. The section of PZD-2.

Figure 3. The section of the column.

Figure 4. The joint with a haunch of beam-column.

Figure 5. The section of PZD-4.
2.2. Test setup and sequence of loading
Figure 8 shows the test setup. Hinge connections are used at the top and bottom. The height and length of the test specimen are 1.8 m and 4.5 m, respectively. Constant vertical load is exerted concentrically on the column. Cyclic loads are applied on the free ends of the beam. Before beam reinforcement yielding, all specimens are loaded under multiples of 20 kN, and then the specimens are loaded under multiples of beam yielding displacement, with each displacement cycle tripled.

3. Computational analysis
For finite-element analysis (FEA), the super-sap program is adopted. The computational model and boundary condition simulate those of the test specimens. There are two groups of ten models. The first group consists of CJ1–CJ8. The models’ eccentricities vary from 0 to 3/8 of the column width. The first group aims to analyze the seismic behavior of beam-column joints with different eccentricities. CJ8 includes a portion of a slab with 60 mm thickness and aims to study the effect of the slab. Figure 9 shows the dimensions of CJ1, and the dimensions of CJ2–CJ8 are shown in Figure 10.
The second group containing CJ9 and CJ10 aims to study the seismic behavior of joints with a haunch beam. The detail of CJ9 is depicted in Figure 11. The eccentric beam-column joint CJ10 in Figure 12 has a wide beam, and its eccentricity is 1/4 of the column width. CJ10 analysis result verifies whether the seismic behaviors of CJ9 and CJ10 are identical. Details of the computational models are presented in Tables 1 and 2.

**Table 1. List of the Dimensions of Parts.**

| No. | e   | e/b₀ | h₀  | x   |
|-----|-----|------|-----|-----|
| CJ1 | 0   | 0    | 225 | 337.5 | --- |
|     |     |      |     |     |     |
| CJ2 | 112.5 | 1/8  | 225 | 225 | x>h₀/4 |

**Figure 9.** The plane of the beam-column joints.

**Figure 10.** The plane of the eccentric beam-column joints.

**Figure 11.** The plane of the haunch beam-column joints.

**Figure 12.** The plane of the wide beam-column joints.
| No. | e   | e/b<sub>c</sub> | b<sub>b</sub> | x          |
|-----|-----|----------------|-------------|------------|
| CJ3 | 187.5 | 1/5            | 225         | 150        |
| CJ4 | 225  | 1/4            | 225         | 87.5       |
| CJ5 | 262.5 | 5/17           | 225         | 75         |
| CJ6 | 281.5 | 5/16           | 225         | 56         |
| CJ7 | 337.5 | 3/8            | 225         | 0          |
| CJ8 | 337.5 | 3/8            | 225         | 0          |
| CJ9 | 337.5 | 3/8            | 225         | 300        |
| CJ10| 225  | 1/4            | 450         | 0          |

### Table 2. List of the Dimensions of Special Parts.

3.1. Boundary condition and type of loading
Translations in the X and Y direction at the top of the column are restricted, as are translations in the X, Y, and Z directions at the bottom of the column. The boundary conditions are identical to that of the test specimens.
The loading type is the same as the experiment, with a constant vertical load acting on the top of the column and cyclic loading acting on the beams’ free ends.
Figures 13 and 14 show the analysis model.

![Finite-element model of beam-column joints.](image)

**Figure 13.** Finite-element model of beam-column joints.
4. Experimental results of eccentric beam-column joint

Experimental results are in line with expectations. In PZD-1 and PZD-2, the test load corresponding to the beams’ flexural strength is less than the test load corresponding to the joint core shear strength. After the beam reinforcements have yielded, hoops in the joints core yield. PZD-1 and PZD-2 Joint cores reached their shear strength capacity, and at the same time, numerous vertical cracks appeared on both sides of the column.

4.1. Shear stress concentration in the cone

Seven models are used in the FEA of the eccentric beam-column joint with eccentricity ranging from 0 to 3/8 of the column width. Analysis reveals the shear stress distribution characteristics in the joint core. The shear stress is distributed symmetrically within the joint when the beam is in the middle of the column section. As the eccentricity increases, the peak shearing stress moves toward the column side where the beam is situated. When the eccentricity is less than 1/4 of the column width, the peak value of the shear stress does not exceed that of zero eccentricity.
The peak value of the shear stress increases when the eccentricity exceeds 1/4 of the column width. The peak shear stress at the face of the column reaches its maximum value, which is about twice the value corresponding to zero eccentricity when the face of the beam is flush with the face of the column. Figure 15 shows the shear stress distribution of the eccentric beam-column joint with different eccentricities.

![Image]

Figure 15. Shear stress distribution of the eccentric beam-column joint.

Figure 16 shows the tensile strain distribution of the hoops in the joint core during the experiment. The eccentricity of the specimen is 3/8 of the column width. It is found that an increase of loading leads to an increase of the tensile strain of the hoops within 300 mm width from the edge of the joint, whereas those outside this region have minor variation. As demonstrated by the experimental result, the peak value of the shear stress is located at the edge of the column when the beam face is flush with the column face.

The experimental result and FEA both lead to a shear stress concentration in the joint core of the eccentric beam-column joint. The shear stress distribution is unsymmetrical, and the maximum shearing stress is located at the beam side.

4.2. Torsion effect

The shear deformation distribution of the eccentric beam-column joint core is similar to that of the shear stress. The maximum value is situated at the beam side of the joint, and it decreases toward the other side. Figure 17 illustrates the shear deformation of CJ7. The maximum value is located at the beam’s outer side, and it gradually decreases within the beam’s width. The shear deformation slips down sharply once it is out of the beam width and approaches zero on the other side of the column. The effect of the differential shear deformation reflects the influence of torsion on the joint core. Both analysis and experimental results support the above effects.
4.3. Vertical cracks on column

According to FEA, the principal tensile stress distribution of the eccentric beam-column joint differs from that of the joint with a beam situated in the middle of the column. When the eccentricity is zero, the principal stress value decreases from both edges of the beam. The maximum value of the principal stress is situated at the edge of the column when the side of the beam of the eccentric beam-column joint is flush with the side of the column.

Figure 18-1. Distribution of shear deformation.

Figure 18-2. Principal tension stress.

Figure 18 demonstrates the principal tensile stress distribution of CJ1 and CJ7. The vertical cracks on the columns of PZD-1 and PZD-2 are shown in Figure 19. The location of the cracks matches the maximum stress location in FEA.
As demonstrated in the experiment, the vertical cracks develop upward on one side of the column and downward on the other side. The directions of vertical cracks are reversed with an alternative loading direction. The above phenomena explain that vertical cracks are caused by the combined effect of the differential shear deformation and the differential compressive deformation of the column section, as revealed by the experiment. Figure 20 shows the strain variation in the longitudinal reinforcement of the column under successive loading stages.

Columns within the beam width are significantly more stressed than those outside the beam width. It can be seen that certain part of the column is more effective at resisting the vertical load and the flexural moment. The combined differential alters the formation of the principal stress and leads to the vertical cracks on the column.
4.4. Slab effect on the joint

Experimental result shows there is no apparent difference between the specimens with and without slab. In practice, the torsion effect of the eccentric beam-column joint on the column may be significantly reduced due to the large monolithic rigidity of the floor slab.

5. Design recommendation

The following design recommendation is suggested based on experimental research and FEA.

5.1. Effective joint width

The shear stress concentration effect in the joint core should be considered in the beam-column joint with \( b_b < b_c/2 \) or the eccentric beam-column joint with an eccentricity greater than \( b_b/4 \). It can be represented by the reduction of the shear area or the effective joint width. As stipulated in Refs. [1], [2], and [3], when the eccentricity is less than 1/4 of the column width, the effective joint width is taken to be the minimum value calculated from the following equations.

\[
\begin{align*}
b_j &= b_b + 0.5h_c \\
b_j &= b_c
\end{align*}
\]

(1)

\[
b_j = 0.5(b_b + b_c + 0.5h_c) - e
\]

The equations above are based on the assumption that the shear stress distribution on both sides of the beam is roughly \( 0.25h_c \). The effective joint width is equal to the sum of \( b_b, 0.25h_c \), and \( x \). (see Figure 21) when the distance between the beam face and the column face is less than \( 0.25h_c \),

\[
\begin{align*}
x < 0.25h_c, & \quad b_j = b_b + x \\
x \geq 0.25h_c, & \quad b_j = b_b + 0.25h_c
\end{align*}
\]

(2)

Table 3 lists the effective width of CJ1–CJ7, calculated using different formulas. Also listed are the ratios between the area of the shear stress diagram within the effective joint core section (\( Se \)) and the total area of the shear stress diagram within the whole joint core section (\( \Sigma S \)).

| Specimen | CJ1 | CJ2 | CJ3 | CJ4 | CJ5 | CJ6 | CJ7 |
|----------|-----|-----|-----|-----|-----|-----|-----|
| \( e/b_c \) | 0   | 1/8 | 1/5 | 1/4 | 5/17| 5/16| 3/8 |
| formula  | (1) | (1) | (1) | (1) | (2) | (2) | (2) |
| \( b_j(mm) \) | 375 | 375 | 375 | 375 | 300 | 281.5| 225 |
| \( Se/\Sigma S \) | 0.66 | 0.70 | 0.67 | 0.67 | 0.66 | 0.66 | 0.66 |
The values of \( Se/\Sigma S \) listed in Table 3 are almost identical, but the effective width calculated using Eq. (2) is less than that calculated using Eq. (1), indicating that the peak value of the shear stress is more dominant, which explains the characteristics of the beam-column joint with large eccentricity.

5.2. Shear strength of joint core

The combined action of shear and torsion stresses the beam-column joint with large eccentricity, and the effective joint width includes the effects of eccentricity and torsion. The formula in Ref. [1] can be employed to evaluate the shear strength of the joint core.

\[
V_j < 1.1\eta j f_d b j h_j + 0.05\eta j N b / b_c + f_{yP} A_{yj}(h_b - a_j) / s
\]  

Eq. (3)

Table 4 shows the computed values of the shear strength using Eq. (3) and the experimental result. The consistent results prove that Eq. (3) can be used in the design.

| No. | Experimental Value Loading | Shear Strength \( V_e \) | Computed Value \( V_C \) | \( V_e / V_C \) |
|-----|---------------------------|-----------------|-----------------|--------------|
| PZD-1 | 84 | 1080 | 1115 | 0.97 |
| PZD-3 | 83 | 1070 | 1110 | 0.96 |

5.3. Construction detail of beam-column joint with large eccentricity

The following construction details are suggested based on the performance of the beam-column joint with large eccentricity:

5.3.1. Special detailing of column reinforcement. As shown in Fig. 18, owing to the large eccentricity, the flexural moment resisted by the column section is concentrated in an effective width equal to the beam width plus a certain distance beyond both sides of the beam. Fig. 22 reveals that the flexural action of the column section can be considered in two steps; the effective width of the column section \( b_t \) can be taken as \( b_b + 2x \) or \( b_b + 0.5h_b \) in the case of \( x \leq 0.25h_b \) or \( x > 0.25h_b \), respectively. In the second step, the remaining part of the column section can be designed for the total action of the column section, i.e., \( b_t = b_c \). The total vertical load can be used in the design of the column section in both steps. The detail is shown in Fig. 22.

![Figure 22. The plane of beam-column joints.](image)

5.3.2. Special hoops against splitting of column. As Fig. 17 shows, vertical cracks are likely to appear along the column due to the uneven flexural moment and torsion distribution in the column section. Enough horizontal hoops should be arranged for the above effects; the number of hoops should be sufficient to sustain the cracking moment of the column section with depth \( h_b \). Hoops should be spaced no more than 100 mm apart, and placed along with the whole height of the column, see Fig. 23.
Figure 23. Reinforcement of the column in the joint core.

The shear strength of the eccentric beam-column joint core should also satisfy two requirements: the first requirement is the effective joint width. Construction will be difficult due to the small effective area. The shearing stress level expressed by Eq.(4) is the other requirement.

\[ \frac{V_j}{f_k b_h} \leq 0.21 \]  \hspace{1cm} (4)

When vertical cracks appear on the column, the test values of PZD-1 and PZD-2 are 0.29 and 0.27, respectively. The employment of a haunch beam is an effective way to improve the seismic behavior of an eccentric beam-column joint.

6. Experimental result of beam-column joint with haunch beams

The failure mode of PZD-3 designed with a robust joint core and a weak beam is as follows. The joint core still remains in the elastic stage after the beams have yielded, and no vertical cracks appear on the column face. In contrast, the failure mode of PZD-4 designed with a strong beam and a weak joint core is that the hoops in the joint core yield, but the beam reinforcements remain in the elastic stage. The characteristics can be summarized as follows.

6.1. No stress concentration in joint core

Fig. 24 shows the shear stress distribution in the joint core of CJ9 and CJ10. The two curves are almost identical. It can be proven that the seismic behavior of the eccentric joint with the haunch beam is nearly identical to that of the joint with a beam whose width is equal to the width of the beam at the end of the haunch. Figure 25 illustrates the tensile strain distribution of the hoops in the joint core of PZD-3.
With increasing loading, the value of the hoop strain within the whole column section increases. It is quite different from the eccentric joint without a haunch beam. Experimental research and FEA reveal that stress concentration in the joint core has not been found in an eccentric beam-column joint with haunch beams, and shear stress is distributed evenly within the column section. The haunch beam restrains the joint core, which improves seismic behavior. Haunch beam works similarly to a beam with a uniform width equal to the width at the end of haunch.

6.2. Torsion effect
The experiment of PZD-3 with haunch beams shows the uniform distribution of shear stress on the whole column section. Experimental results and FEA prove that the torsion effect on the eccentric joint with haunch beam can be disregarded.

6.3. Vertical cracks on column
Figure 26 shows the longitudinal reinforcement strain distribution in the column under flexural moment due to loading on the beams. Longitudinal reinforcement within the column section is evenly stressed. The entire column section can be considered effective in terms of moment resistance. Simultaneously, the deferential shear deformation has been disregarded.
FEA illustrates the principal tensile stress curve of the eccentric beam-column joint with haunch beams simulated with that of the joint with uniform width beams; it is completely different from the curve of CJ7.

Figure 27. Distribution of maximum stress at haunch joint.

Figure 27 depicts the curves of CJ9 and CJ10. The curves’ characteristics are that the maximum value is situated at the edge of the beam (or haunch) and then drops sharply. The prediction was proven by the experimental result. Vertical cracks have not been found on the PZD-3 column. In the final stage of the experiment, very fine vertical cracks are found on the PZD-4 column.

7. Design recommendation for eccentric beam-column joint with haunch beams

Experimental study and FEA reveal that the seismic behavior of an eccentric beam-column joint with haunch beams is similar to that of a joint with beams of uniform width equal to the width at the end of the haunch.

7.1. Effective joint width

According to research and analysis, the effective beam width can be taken as the sum of the width of the beam and the width of the haunch, known as the total beam width. The effective joint width can be determined as follows:

When the eccentricity between the column width and the total beam width is not more than 1/4 of the column width, the effective joint width is taken as the minimum value computed by the following Eqs.

\[ b_j = (b_b + b_x) + 0.5h_c \]

\[ b_j = b_c \]

\[ b_j = (b_b + b_x) + 0.25h_c + x \]

When the eccentricity between the column width and the total beam width is more than 1/4 of the column width, the effective joint width can be determined by the following Eqs.

\[ e > b_c/4, x < 0.25h_c, b_j = (b_b + b_x) + x \]

\[ e > b_c/4, x \geq 0.25h_c, b_j = (b_b + b_x) + 0.25h_c \]
In the case of $e=0$, $b_b < b_c/2$. Symmetrical haunches can be arranged on both sides of the beam to satisfy $b_b + 2b_x > b_c/2$. The width of the haunch $b_x$ on each side of the beam is taken as:

$$b_x = b_c/4 - b_b/2$$

(8)

7.2. Evaluation of shear strength of joint core

Table 5 shows the experimental result of PZD-3, which is consistent with the value computed in Ref. [1]. The formula in Ref. [1] can also be used to evaluate the shear strength of the joint core for beams with haunch. The $n_j$ value can be considered according to the total beam width and the requirements stated in code GB50011-2001.

| Specimen | Experimental Value $V_e$ (kN) | Computed Value $V_c$ (kN) | $V_e / V_c$ |
|----------|-------------------------------|--------------------------|-----------|
| PZD-3    | 771                           | 768                      | 1.01      |

7.3. Construction detail of eccentric joint with haunch beam

7.3.1. Gradient of haunch. The haunch gradient is suggested to be 1:3. The width of the beam, including the haunch, should be half of the column width. (Figure 28).

7.3.2. Detailing of reinforcement in haunch. The plastic hinge can be relocated to the far end of the haunch. The flexural strength of the beam should be carefully considered to satisfy the requirement. Hoops should be closely spaced at the zone of plastic hinge and haunch. Double hoops should be placed at the intersection of the haunch and beam. (Fig. 29)

Figure 28. Strain distribution on hoops.

Figure 29. Reinforcement at haunch joint

8. Conclusions

The following conclusion has been drawn based on experimental research and FEA.

1. The seismic behavior of an eccentric beam-column joint with eccentricity limited to 1/4 of the column width is a critical margin of shear stress distribution over the entire area of a joint core. Away from this margin, the effective shear area of the joint core should be considered, and a more stringent limitation on the shear stress of the joint core should be taken into account to avoid column splitting
cracks.
2. The eccentric joint with haunch beams effectively improves the seismic behavior of beam-column joints with large eccentricity. Formulas for determining the effective joint width and shear strength of a joint core are presented based on experiment and analysis. The beam-column joint construction details are suggested.

9. Notations

e = eccentricity between beam and column axis.

\( b_c \) = column width

\( b_b \) = beam width

\( b_x \) = width of haunch

\( b_j \) = effective width of joint core

\( x \) = the shorter distance between the edges of beam and column

\( h_c \) = depth of column section

\( h_j \) = effective depth of joint

\( V_j \) = shear force acting on joint core (see Ref.1, Appendix D)

\( \eta_j \) = restraining coefficient of orthogonal beams, \( \eta=1 \) for eccentric joint

\( f_{tk} \) = characteristic value of axial tensile strength of concrete

\( f_{ck} \) = characteristic value of axial compressive strength of concrete

\( N \) = design value of minimum axial compressive force exerted on top of the joint, considering the disadvantage combination of earthquake action

when \( N>0.5f_c b_c h_c \) take \( N=0.5f_c b_c h_c \)

\( f_{yk} \) = characteristics value of tensile strength of hoop

\( A_{ovj} \) = total sectional area of legs of hoops in the same section

\( b_{bo} \) = effective height of beam section

\( a_s \) = distance from near extreme fiber of section to center of longitudinal reinforcement

\( s \) = spacing of hoops in joint core

\( \Sigma S \) = summation of area of shearing stress diagram distributed on the whole joint core section

\( S_e \) = summation of area of shearing stress diagram distributed on the effective joint core section

\( \sigma \) = stress value

\( S \) = strain value
References
[1] National Standard of the People’s Republic of China (NSPRC) 2001 Chinese Code for Seismic Design of Buildings (GB50011-2001) (Beijing, China: Ministry of Construction of Peoples Republic of China)
[2] NZS 3101 2006 Code of Practice for the Design of Concrete Structures (Standards Association of New Zealand)
[3] Eurocode 8 1993 Earthquake resistance design of structures
[4] ACI318M 1989 Building Code (American Concrete Institute)

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