- Technical Paper -

STRENGTH OF RC BEAM-COLUMN JOINT IN SOFT-FIRST STORY WHERE FIRST-STORY COLUMN EXTENDED INSIDE

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ABSTRACT

Strut-and-tie models were developed for beam-column joint in soft-first story where first-story columns were extended toward inside. The developed models were different from those for exterior joints in two points: (1) the sign of the shear force in the first-story column was opposite to that in the second-story column and (2) large struts were extended into the wall panel. The diagonal reinforcement was effective to the closing load because it prevented the compressive failure of the concrete in the wall panel. Based on the strut-and-tie models new design equations were proposed.

Keywords: beam-column joint, strut-and-tie model, soft-first story, exterior joint, wall panel

1. INTRODUCTION

Columns in soft first story are generally larger than those in upper stories, and usually these large columns are extended toward inside of the building as shown in Fig. 1a. Joint failure is concerned and the strength of the first story column might not be fully utilized.

Joint failures associated with yielding of reinforcement in the beam and column for usual exterior joints are investigated by Shiohara et al [1]. However, the existence of the wall panel and discontinuity of column depth makes the problem different.

Hanai et al [2] conducted an experiment on this type of joints for opening direction (the left side joint in Fig. 1a) to investigate the strength and failure mode of this kind of joint. Observed failures were different from usual exterior joints.

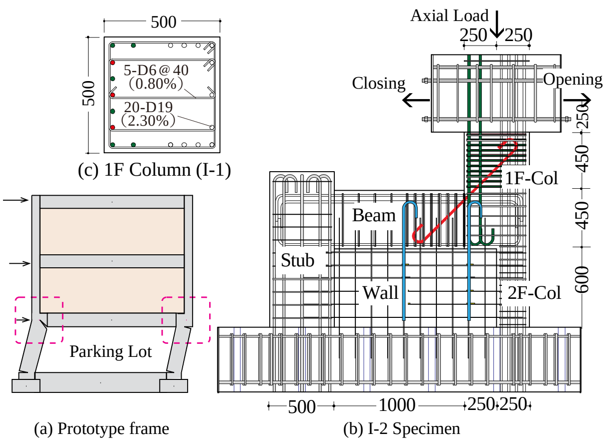

Fig.1 Elevation view of frame and specimen

Ogawa et al [3] conducted another test to investigate the joint behaviors in both directions of loading in Fig. 1a. In this paper, two specimens from the research are used as example for discussion of the strength of this kind of beam-column joint.

Strut-and–tie models (STMs) are developed for two specimens to investigate their strengths. STMs for usual exterior joint are also developed to understand the effects of the wall panel on the joint strength. Based on the STMs new design equations to compute the strength of such joints are proposed.

2. TEST PROGRAM AND RESULTS

Ogawa, et al [3] tested specimens depicting beam-column joints denoted by the dashed line in Fig. 1a. In the prototype building, the depths of the columns in the first story were assumed twice of those in the upper stories (Fig. 1a). A large boundary beam was assumed at the bottom of the wall to reduce the probability of joint failure shown in Fig. 1a. The specimens were constructed upside down to easily apply the loads with the scale of one-half as shown in Fig. 1b. The test parameter was the difference of reinforcement such as inclined bars shown by the red line in Fig. 1b. A stub (strong column) is attached assumed middle point of the span and serves the strength and rigidity of the other side of the span. The upper stub was located at the loading point, which represents mid-height of the first story. Longitudinal reinforcements in the innermost two layers of first story column (9-D19, green in Figs. 1b) were anchored with 180 degree hook at the joint while the remaining bars passed into second story column (Fig. 1b). In I-2 specimen, 3-D19 (red-colored bars in Fig. 1c) of the first story column is replaced with 5-D19 inclined bars.

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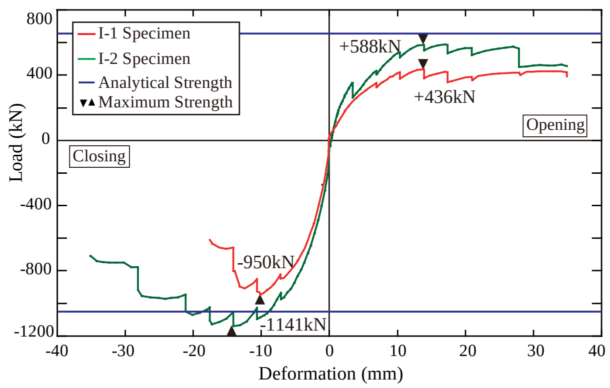

Fig. 2 Load-Displacement relationship

Contribution of the 5-D19 to the flexural strength of first story column is almost equal to 3-D19. More stirrups are provided in the beam over the length of inclined bars (6-D6@62.5), whereas I-1 specimen has fewer stirrups (4-D6@125) along the beam length. Additionally, 2-D19 bars (blue-colored bars in Fig. 1b) are provided and anchored in the wall panel. One is located in the beam-column joint and another is near the end of inclined bars.

Compressive strength of concrete is 26.4 N/mm^2 and yield strengths of reinforcements are 375 N/mm^2 and 369 N/mm^2 for D19 and D6 respectively.

Displacement was controlled at the loading point. Axial force equal to 30% (2250kN) of the first story column capacity was applied in the closing direction and no axial load was applied in the opening direction considering overturning mechanism of the structure.

Figure 2 shows the test results. The blue lines show analytical lateral strength based on flexural capacity of first story column at the beam bottom face. The strength is computed using Bernoulli-Euler assumption. For I-1 specimen, observed maximum strengths are smaller than analytical results both in the opening and closing directions. In the opening direction, vertical reinforcements in the first story column were almost yield. Beam bottom bars yielded in tension, stirrups near the joint yielded in early stage of loading and beam failure observed. In the closing direction, beam top bars yielded in tension, vertical reinforcements in the first story column were also close to yield. The wall panel above the joint crushed in compression.

Maximum observed strength of I-2 specimen agrees with the analytical result both in opening and closing directions. In the opening direction, first layer of vertical bars in the first story column, beam bottom bars, stirrups near the joint and inclined bars yielded in tension. The observed failure mode was joint failure. In the closing direction, first layer of vertical bars in the first story column and beam top bars yielded in tension. Inclined bars yielded in compression. Joint failure was observed in the test.

3. STRUT-AND-TIE MODELS

Strut-and-tie models (STM) are developed for the specimens to understand the failure modes and failure mechanism.

Figures 3 and 4 show the developed STMs for opening load of I-1 and I-2 specimens, respectively. Blue lines show bars in tension. Solid blue lines indicate tensile yielding while dashed lines indicate below yield.

STMs for the opening direction are determined assuming as follow:

(1) Moment by the forces acting on the stub is zero around node A.
(2) The three layers of the column main bars (AC and AD in Figs. 3 and 4) resist tension. The beam bottom bars, the inclined bars provided to I-2, vertical bars in wall panel and the stirrups also resist tension.
(3) Effective compression strength of concrete to determine strut's width is assumed to be 85% of concrete strength.
Note that the width of the wall panel was 100mm (1/4 of the beam width) and the strut widens at the boundary of the beam and the wall panel.
(4) Node B is located at centerline of the beam bottom bars such that the outer edge of the nodal zone is coincide with the outermost point of hooked beam bottom bar (see Fig.3).

In Fig. 3, the analytical strength of I-1 is determined by the yielding of tensile members (AC, AD, BF and EF). Tensile member EF represents stirrups in the beam and it yielded in the analysis. These stirrups also yielded in the test. Locations of nodes (E and F) are determined so as to get the strength as large as possible. The strut distribution agreed with the crack pattern appeared in the test as shown in Fig. 3.

In Fig. 4, tensile member A′G represents the diagonal reinforcements. The strength of I-2 is also determined by the yielding of tensile members (AC, AD and A′G). The strut distribution also agreed with the crack pattern appeared in the test as shown in Fig. 4.

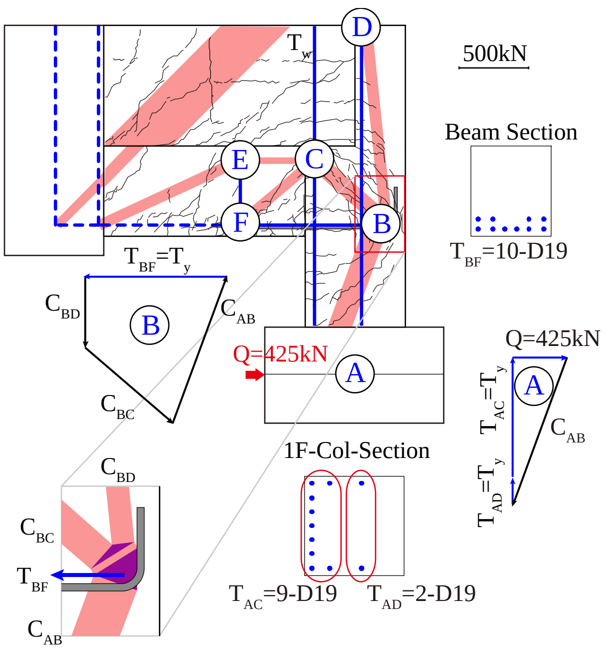

Fig.3 Strut and tie model of I-1 (Opening)

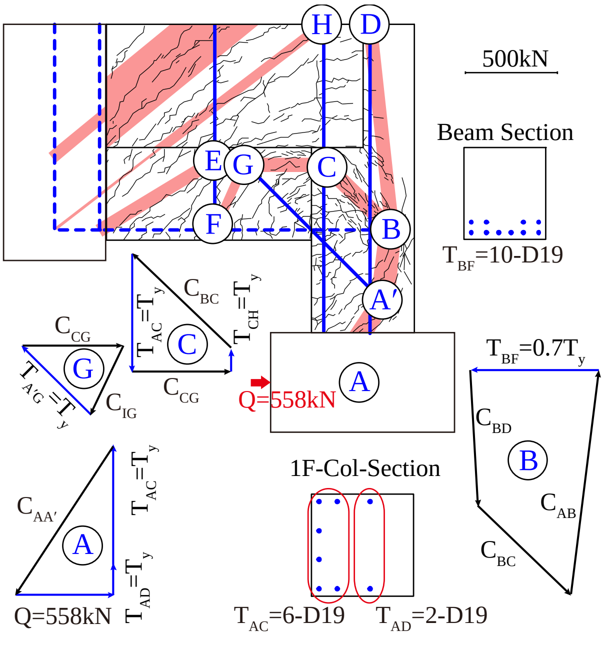

Fig.4 Strut and tie model of I-2 (Opening)

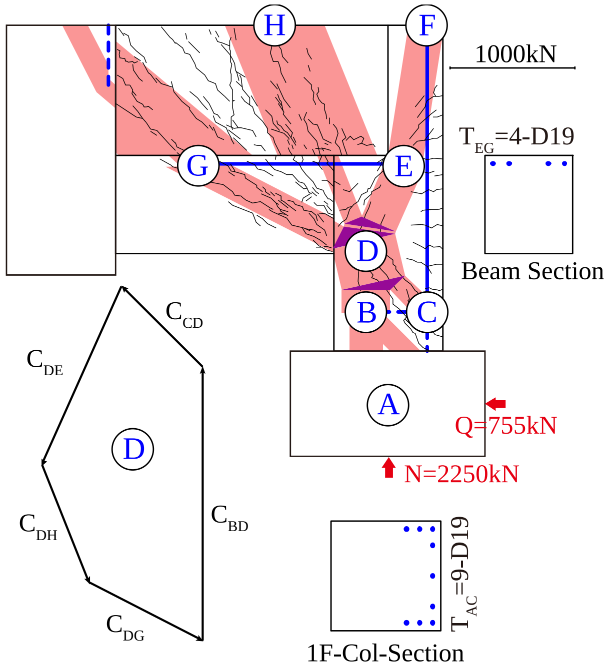

Fig.5 Strut and tie model of I-1 (Closing)

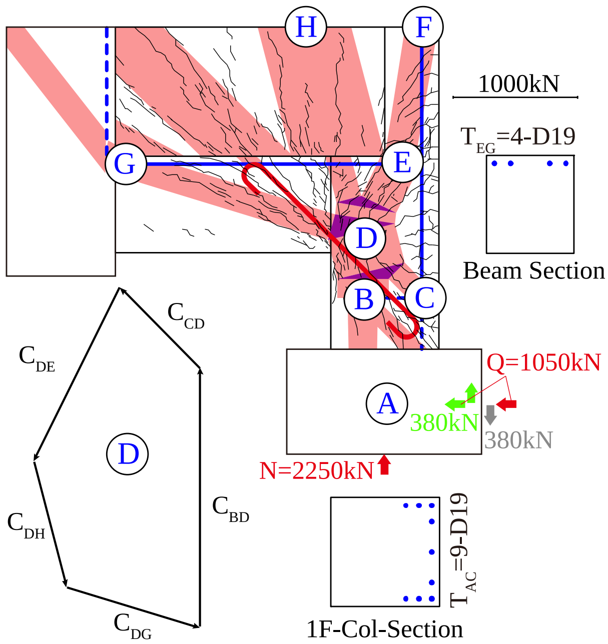

Fig.6 Strut and tie model of I-2 (Closing)

Computed tensile forces in the ties agreed with the observed strains in the reinforcement for both specimens. The observed strain of beam bottom bars for I-1 specimen was larger than that of I-2 specimen. In the STM, computed tensile force in beam bottom bars for I-1 specimen is larger than I-2 specimen. Strength based on STM also agreed with the observed strength of specimens with errors less than 10%. Sign of shear force in the second story column is opposite to that in the first story column (Figs. 3 and 4). This is different from usual beam-column joints. Wall panel combined the second story column with the beam, they resist to the bending moment together. In other words, the second story column helps the beam whereas it helps the first story column in usual beam-column joints.

Figures 5 and 6 show the developed STMs for closing load of I-1 and I-2 specimens, respectively. In addition to 1st, 3rd and 4th assumptions for opening direction (note that node B in 4th assumption is replaced with node E in the case of closing direction, but the location of the node is decided similarly), the following assumptions are made in the closing direction:

(5) The three layers of the column main bars (AF in Figs. 5 and 6) resist tension. The beam top bars (EG in Figs. 5 and 6) also resist tension.
(6) Vertical bars in the compressive area of the first story column do not carry any compression force because they are terminated in the joint (green in Fig. 1b).
(7) Inclined bars are assumed to resist compression; applying compressive force onto them separately and then superimposing with the remaining system of forces.

In Fig. 5, strut widths are much wider than those in opening direction because large axial force was applied in closing direction. The strength of I-1 in closing direction is determined by the yielding of bars (CF and EG in Fig. 5). Computed strength is almost 80 % of the maximum strength in the test.

STM for I-2 specimen was developed as sum of two independent models. First model is consisted of only diagonal reinforcements. Diagonal force (it is decomposed into vertical and horizontal forces shown in Fig. 6 by green arrows) is applied to this model and the diagonal bars yield to satisfy the 7th assumption. Compressive force goes into the wall panel as shown in Fig. 6. Second model is specimen I-2 without diagonal bars. In the analysis of the second model, virtual vertical force (gray arrow in Fig. 6) is applied to cancel the vertical component in the first model. The strength of I-2 is computed as sum of vertical forces in these two models (1050 kN in Fig. 6).

The computed strength of I-2 in closing direction is larger than that of I-1 and close to the maximum strength of I-2 in the test. The inclined bars in I-2 specimen were more effective especially in the closing direction. Forces computed in the reinforcements also have good agreement with the strains observed in the test. In both specimens, big shear force is transferred to the wall panel in the second story. Shear force in the second story column is also opposite to that in the first story column in closing direction.

4. STM FOR EXTERIOR JOINT

STMs are developed for usual exterior joints to compare stress transfer mechanism of exterior joints with that of the joint in soft-first story. Procedure, assumptions and reinforcement details same as to specimens are used. In addition, it is assumed half of the applied shear force is resisted by second-story column. Axial force equal to that of the test specimens (2250kN) is applied only in the closing direction. Beam flexural failure was predicted both in opening and closing directions.

STM for the opening load is shown in Fig. 7. The predicted strength (281 kN) is approximately 60% of those of the test specimens. This difference is attributable to the location of the strut and tie in the second story column: In Figs. 3 and 4, there is struts in the wall panel which produces bending moment opposite to that of usual joint (Fig. 7).

STM for the closing load is shown in Fig. 8. The predicted strength (175 kN) is approximately 20% of those of the test specimens. This difference is attributable to the following three reasons:

(a) The amount of the top beam reinforcement (4-D19) is smaller than that of the bottom reinforcement (10-D19).

(b) In Figs. 5 and 6, several struts such as DH transfer shear forces into the upper story.

(c) In Figs. 5 and 6, tie CF is located to the right of strut EF, which produces bending moment opposite to that of usual joint (Fig. 8).

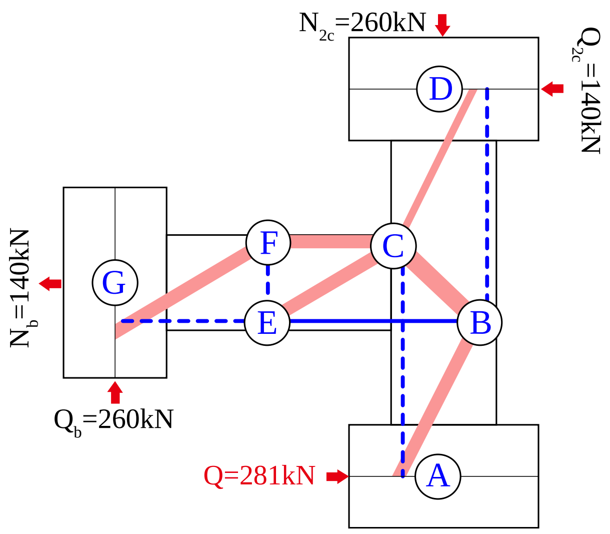

Fig.7 Strut and tie model of exterior joint (Opening)

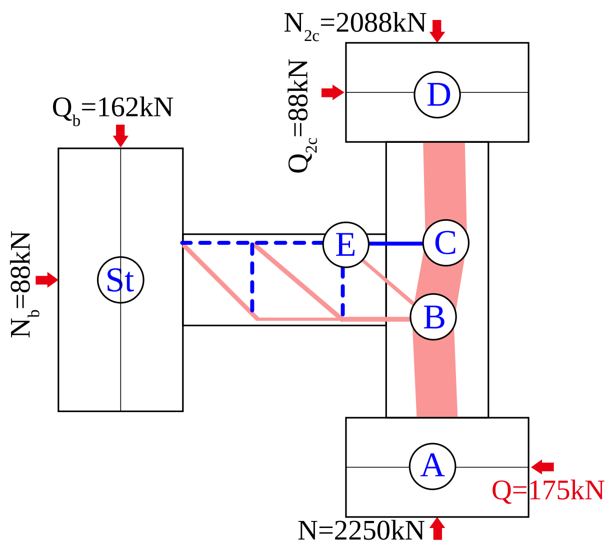

Fig.8 Strut and tie model of exterior joint (Closing)

5. STM FOR A PROTOTYPE FRAME

Figure 9 shows STM for a transverse frame of full-scale real structure. The building structure is assumed to be 5-story single bay apartment building

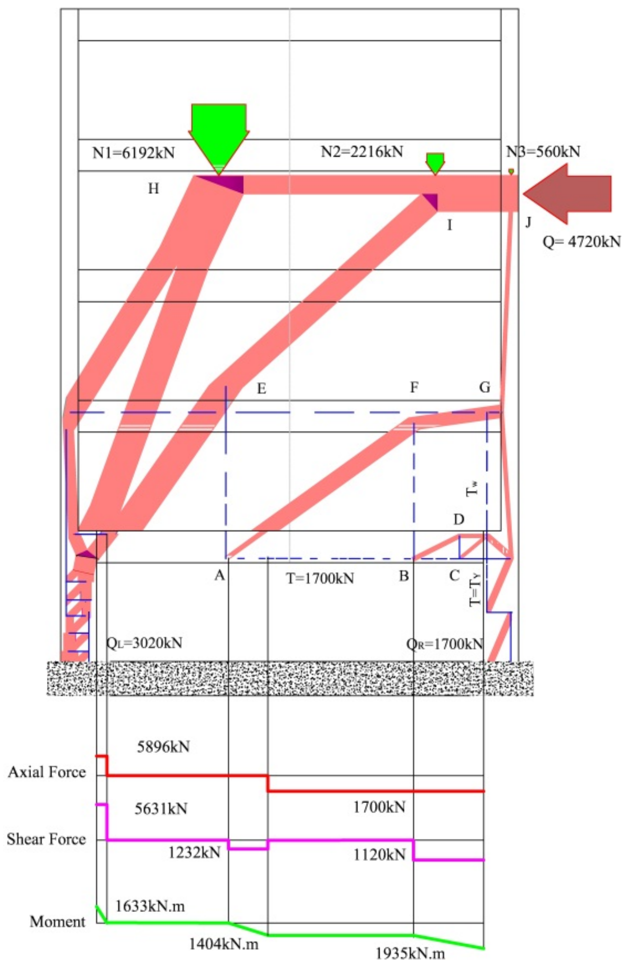

Fig.9 Strut and tie model of a 5 story building frame

with span length of 12m and the transverse frames are spaced 12m. This figure represents overall flow of forces in the lateral force resisting frame. Ties CD and BF are located close to each other which means if the beam-column joint is opening some shear reinforcing is required in the beam near the joint. On the other hand, tie AE near the mid-span, represents stirrups of a big portion of beam-span length which means small amount of stirrups are required in mid-span of the beam or if the beam-column joint closes no stirrups are required in the beam near the joint. In the test, stirrups near the joint yielded in the opening direction whereas did not yield in the closing direction. This difference can be attributed to presence of shear wall above the beam. The shear wall (its vertical reinforcement) also contributes to resist shear force of the beam in the joint opening direction. In the closing direction, the shear force is transferred to the shear wall above the boundary beam. In Fig. 9, shear force diagram shows contribution of shear wall and its vertical reinforcement along the beam span.

This figure shows the case without diagonal bars. If diagonal bars are provided at the joints as I-2 specimen, the struts in the wall panel would be distributed more widely resulting in larger strength of the frame.

6. DESIGN EQUATIONS

Following the results of strut and tie analysis, simplified design equations are proposed in this research.

The ultimate lateral strength Q_u is a smaller value of the strength of the first story column Q_c and that of the joint Q_j.

$$Q_u = \min[Q_c,\ Q_j] \tag{1}$$

The critical sections of the column failure and joint failure are assumed as broken lines in Fig. 10 based on the strut and tie model. The strength of the first story column Q_c is calculated by the following equation.

$$Q_c = \frac{M_{c1}}{L + l_c} \tag{2}$$

where,
M_{c1}: the moment capacity at the top of the column
L: the length between the point of contra flexure and top of the first story
l_c: the distance between the top of the first story column and the centroid of the beam bottom bars

In Eq. 2, $(L + l_c)$ is the shear span length whereas L is usually called the shear span length. In opening direction, the critical section is l_c upper from the top of the first story column as shown in Fig. 10a. In closing direction, the critical section is assumed along the rectangular which is connected to two struts (BD and CD in Fig. 10c) in the first story column. The moment capacity of the first story column is calculated at the height of the centroid of the compressive force C_v. The location of this point is close to the centroid of beam bottom bars in usual beams. In fact, the length of l_c is from 56 mm to 61 mm in this research and the critical sections of I-1 and I-2 are 57 mm and 78 mm upper from the top of the first story column, respectively.

In addition, the effective depth of the first story column in the opening direction is smaller than the depth of the first story column because concrete outside the anchorage of the beam top reinforcement is not effective to compressive force as shown in Fig. 10a. In this research, 439 mm is used as the effective depth in the opening direction. On the other hand, full depth of the first story column is assumed to be effective in the closing direction as shown in Fig. 10c. To consider the effective depth in the moment capacity of the first story column, the following equation is used.

$$M_{c1} = \frac{D_{eff}}{D_{c1}} \cdot M_0 - N \cdot \frac{D_{c1} - D_{eff}}{2} \tag{3}$$

where,
D_{eff}: effective depth of the first story column
D_{c1}: actual depth of the first story column
M_0: the moment capacity of the column without considering the effective depth

The strength of the joint Q_j is calculated by the following equations.

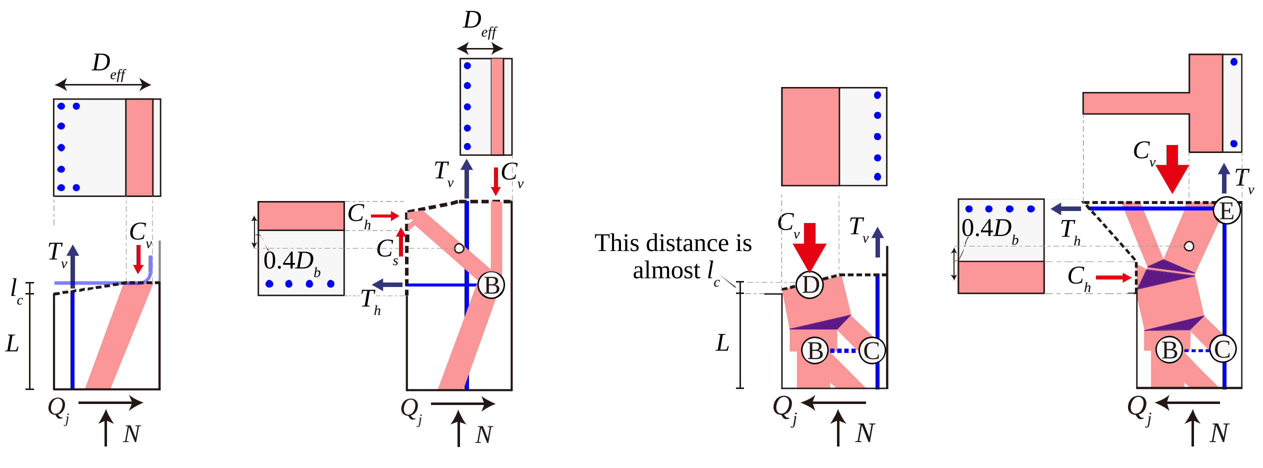

Fig. 10 Critical sections

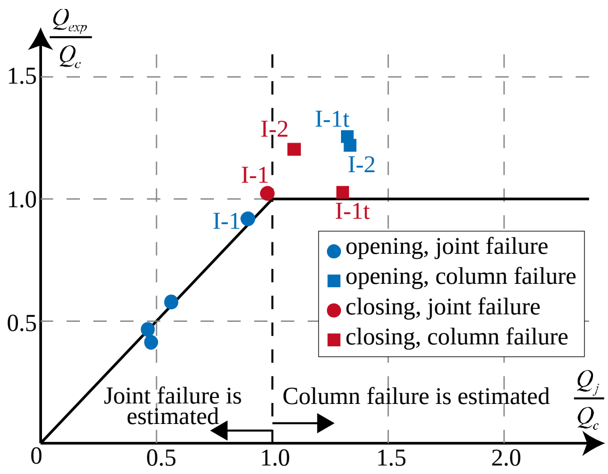

Fig. 11 Estimated and observed strengths

$$Q_j = \frac{M_j}{L + D_b/2} \quad (4)$$

$$M_j = M_{c2} + M_b \quad (5)$$

$$M_b = \begin{cases} M_{b0} - 0.4Q_j D_b & \text{for opening} \\ M_{b0} + 0.4Q_j D_b & \text{for closing} \end{cases} \quad (6)$$

where,

M_{c2}: the moment capacity of second story column around the center of the first story column

M_b: the moment capacity of beam

M_{b0}: the moment capacity of beam without axial force

D_b: beam depth

The moment capacity of the joint is defined around the white circle shown in Fig. 10b. This point can be chosen arbitrary, and the intersection of centerlines of the beam and the first story column is chosen in this paper. Therefore, the shear span length is D_b /2 longer than usual as seen in Eq. 4. Moment capacity of the joint is computed as sum of the capacity of the beam and the second story column because the second story column helped the beam in strut and tie analysis. In Fig. 10, forces acting on the critical sections are shown. The contributions of the vertical forces acting on the critical section are computed as the moment capacity of the second story column. Similarly, the contributions of the horizontal forces acting on the critical section are computed as the moment capacity of the beam. In Ref. 2, similar assumption is adopted for computing the moment capacity of the joint but the axial force in the beam is not considered.

The horizontal force C_h in Fig. 10b is the axial force in the beam, and its value is C_h = T_h - Q_j. It means the tensile axial force Q_j is applied to the beam. Therefore, its contribution is computed as the second term in Eq. 6. In this calculation, iteration method is needed because this term includes the joint strength Q_j.

In addition, the vertical component of the diagonal strut C_s is acting on the critical section. Its contribution is included in the moment capacity of the second story column.

In closing direction, forces acting on the critical section are shown in Fig. 10d. From the equilibrium in horizontal forces, it is clear that the compressive axial force of the beam is equal to Q_j. The distance between the centroid of C_h and the reference point is almost $0.4D_b$. Its contribution is computed as the second term in Eq. 6. Similarly, tensile axial force acting on the beam is equal to Q_j in the opening direction as shown in Fig. 10b. Its contribution is also computed as the second term in Eq. 6 and the sign of the second term is opposite to that in the closing direction.

Figure 11 shows the comparison between observed and calculated strength. Proposed design equations evaluate the strengths of the specimens appropriately including the specimens used in Ref. 2. Observed failure modes are represented by the shape of the marks. Estimated failure modes agree with observed failure modes in all specimens.

7. CONCLUSIONS

(1) For opening load, embedment length of beam bars should be regarded as effective depth of the first story column.

(2) Inclined reinforcement in the beam-column joint was effective both in opening and closing directions.

(3) Strength related to the joint failure can be approximated to the sum of the strengths of beam and that of the second story column including the effects of the wall panel and stirrups in the beam.

(4) Strength and failure modes predicted by the proposed equation have good agreement with the test results.

(5) Comparison between strut-and-tie models for exterior joint and for the specimens shows that the flow of forces and corresponding strength of usual exterior joints are very different from that of the specimens.

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