

Experimental Study on Beam-column Joint of Soft-first-story in RC Building

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SUMMARY:

Observations from the recent earthquakes' damages show that many damages or collapses occurred in soft-story configuration. Rupture in the beam-column joint at the top of such a soft-story is also often reported. In this study, tests are conducted to understand such failures. A multi-story apartment building is considered. The first story is single-bay moment resisting frame and the upper stories have shear wall with boundary columns. The depths of the columns in the first story are twice of those in the upper stories. The specimens are analysed and strut and tie models are developed for the specimens. Based on the test and analytical results and strut and tie models the effects of boundary beam and column, inclined reinforcement in the joint, stirrups, and the wall panel are discussed.

Keywords: Beam-Column Joint, Soft Story, RC Building, Strut and Tie Model

1. INTRODUCTION

Beam-column joint is the critical part of a RC building with moment-resisting frames. Many research works have been carried out considering different parameters such as anchorage of beam and column longitudinal reinforcements, joint shear reinforcement, concrete compressive strength, and bond strength which significantly influence the joint behavior (Shiohara, 2004; Shiohara and Shin, 2006; Shiohara, 2012).

In this research a multi-story apartment building is considered with parking lot at the ground floor. Shear walls with boundary columns are assumed to be provided in the upper stories, whereas the first story is single-bay moment resisting frame. The depths of the columns in the first story are twice of those in the upper stories. A large boundary beam is provided at the bottom of the wall. Specimens with different reinforcement layout are constructed with scale of one-half to examine the beam-column joint behaviour. The specimens are subjected to cyclic loading, with the displacement controlled method. The specimens are divided into two types: 1) the first floor column is extended toward outside, and 2) the first floor column is extended toward inside. The specimens are analysed with standard method assuming the beam has infinite rigidity. Strut-tie-models are developed for each specimen. The test result for both types of specimens and their strut-tie-models are compared and discussed with the analysis results.

2. SPECIMENS

Two types of specimens are constructed: (1) the first-story column is extended toward outside (O-series) as shown in Fig. 2.1a; (2) the first-story column is extended toward inside (I-series) as shown in Fig. 2.1c. For each type there are two specimens, Fig. 2.1b and 2.1d, where the parameter is whether the inclined reinforcement is provided or not at the corner of the joint. The axial force is

varied considering the effects of overturning moment of the building in order to verify the strength in both sides of joints in a single-bay frame under cyclic loading. The area denoted by the broken line in Figs.2.1a and 2.1c are constructed as specimens. Specimens are constructed upside down to apply the loads easily as shown in Figs. 2.1b and 2.1d. A strong stub-column is attached at the middle point of the span so that it provides the strength and rigidity of the other side of the span. The upper stub is located such that the lateral loading point will be the middle point of the first-story height. This means the shear span of the first-story is assumed to be half of the story height. Details of the specimens are described below for each series.

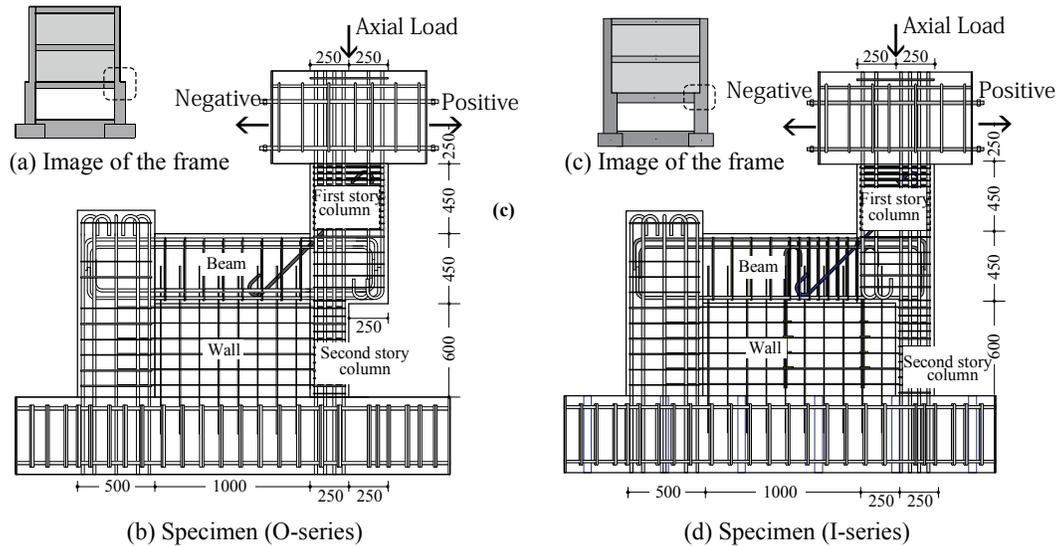


Figure 2.1 Perspectives of Specimens (mm)

2.1. The First Story Column Extended Toward Outside (O-series)

Two specimens O-1 and O-2 are constructed. The details are shown in Fig. 2.2. O-1 is the prototype specimen. The cross-section of the beam and the second story-column are shown in Figs. 2.2b and 2.2a, respectively. Figs. 2.2d and 2.2e show the first-story-column section and detail of the beam-column joint, respectively. The bars, seven #6 and two #6 in the two outermost layers of the first-story column are anchored with 180 degree hook at the joint, while the rest are passed into the second-story column (See Fig. 2.2a). Enough hoops and ties are provided in the first-story column to prevent shear failure in the first story.

For O-2 specimen, the first and second story columns cross sections are shown in Figs. 2.2f and 2.2c respectively. Three of the interior longitudinal bars passed into the second story column are replaced by five #6 inclined bars at the corner of the beam and the first story column joint. The inclination angle is 45 degrees as shown in Fig. 2.2g. The contribution of the five #6 inclined bars to the flexural strength of the first story column is considered equal to that of three #6 in O-1 specimen (Fig. 2.2d), $5 \cdot \cos(45) \approx 3$. The beam cross-sectional details are remained same as that of O-1 specimen.

2.2. The First Story Column Extended Toward Inside (I-series)

Two specimens, I-1 and I-2 are constructed, and the details are shown in Fig. 2.3. I-1 is a prototype specimen. Fig. 2.3a shows the beam section. The second story columns details are same for both specimens and shown in Fig. 2.3c. First story column section and beam-column joint details are shown in Figs. 2.3d and 2.3e, respectively. In this case nine #6 bars are provided in the innermost two layers of the first story column and anchored in the joint as shown in Fig. 2.3e. The beam upper reinforcement (4#6) is less than that of O-Series (10#6), because the tensile force in the reinforcement caused by the negative loading (closing direction) is expected to be less than that of O-series

specimens.

For specimen I-2, beam section, first story column section and joint details are shown in Figs. 2.3b, 2.3f and 2.3g, respectively. The second story column section is identical with that of I-1. More stirrups (6 #2@62.5mm) are provided in the beam near the joint in a length almost equal to beam depth compared to I-1 (4 #2@125mm).

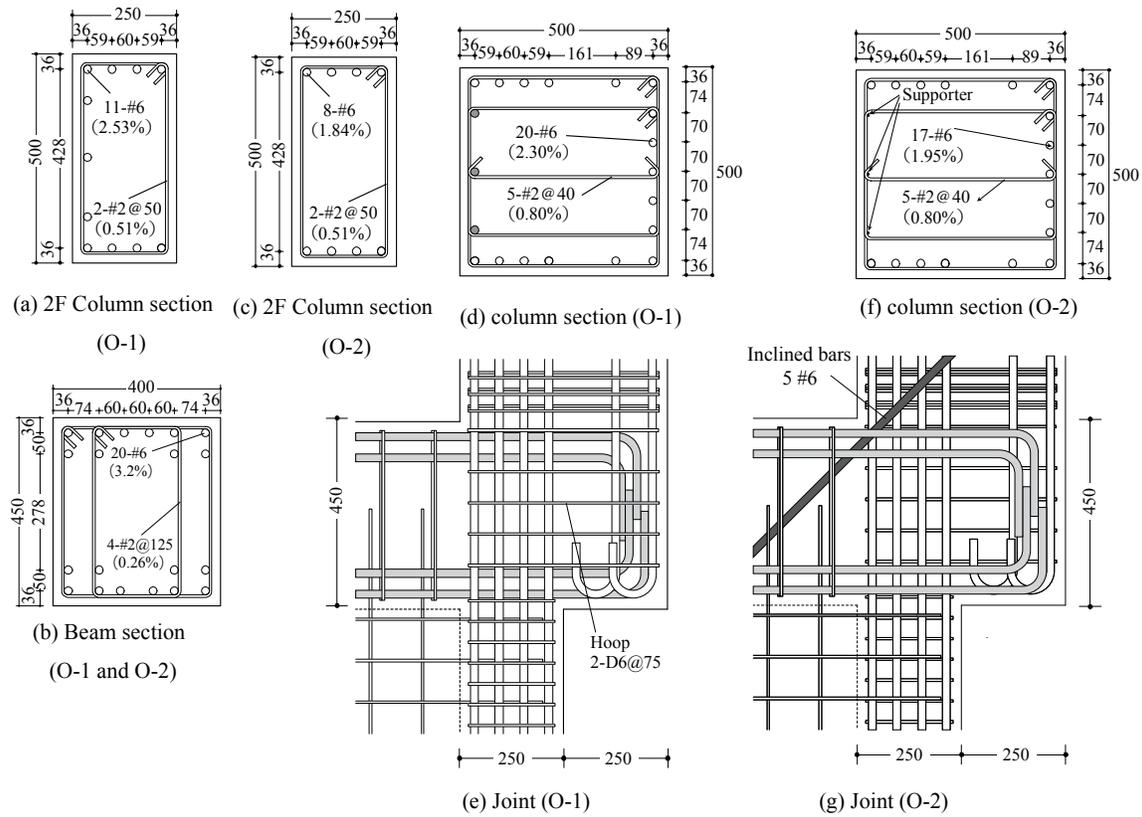


Figure 2.2 Details of O-Series Specimens (mm)

Specimen I-2 has also the inclined bars same as that of O-2 specimen. Additional 2#6 bars are provided and anchored in the wall panel. One is placed near the inner face of the first story column, and the other is placed just at the end of inclined bars to prevent expected shear failure mode of beam at this point (Fig. 2.3g). Cross-ties are provided for the #6 bars to prevent the split failure of the wall with 100mm thickness.

Material properties of concrete are listed in Table 2.1, where f_c' is the compressive strength, f_r is the modulus of rupture, and E_c is the elastic modulus. Material properties of steel bars are indicated in Table 2.2, where f_y is the yield strength, f_u is the tensile strength, and E_s is the elastic modulus.

Table 2.1. Material Properties of concrete

	f_c' (N/mm ²)	f_r (N/mm ²)	E_c (kN/mm ²)
O-Series	28.7	2.70	24.5
I-Series	26.4	2.32	24.5

Table 2.2. Material properties of reinforcement

	f_y (N/mm ²)	f_u (N/mm ²)	E_s (kN/mm ²)
#6(D19) bars	375	585	193
#2(D6) bars	369	509	199

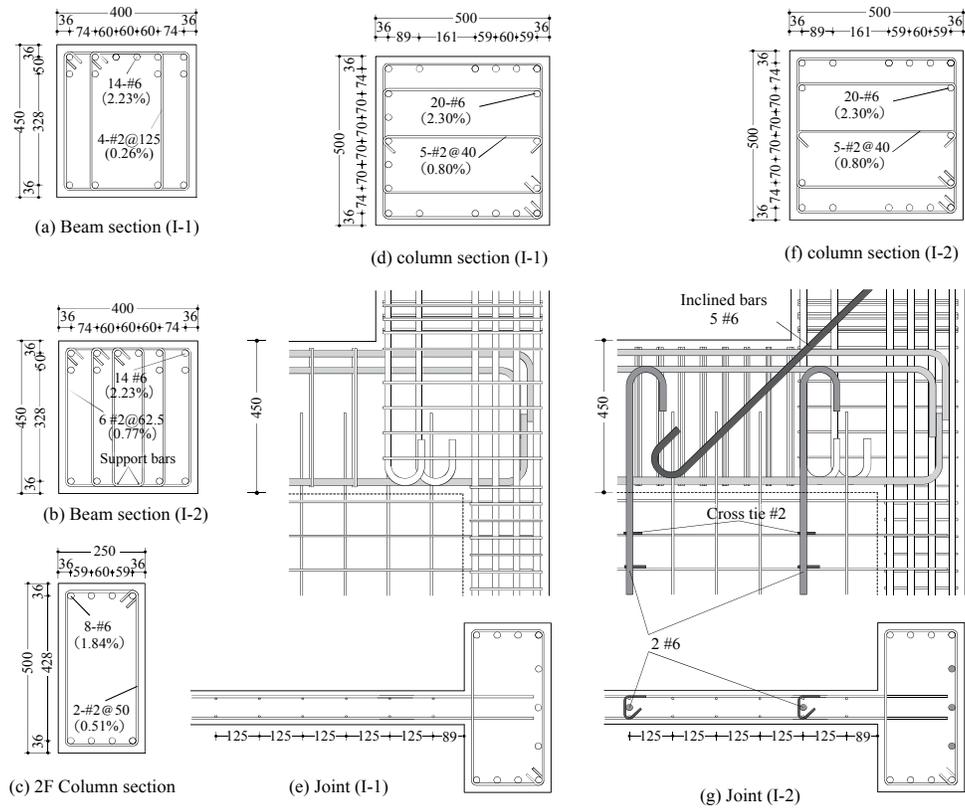


Figure 2.3 Details of I-Series Specimens (mm)

3. TEST SETUP

Test setup is shown in Fig. 3.1. Displacement was controlled in lateral loading point. Lateral drift is defined as the ratio of the lateral displacement at the loading point to the length between this point and the bottom of the first story beam (700mm).

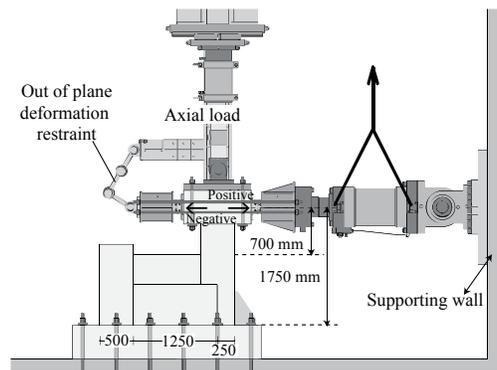


Figure 3.1 Test setup (mm)

Before the specimens experience 0.5% drift, constant axial load (1125kN: approximately 15% of the axial capacity of the first story column) was applied. After that, axial load (2250kN: approximately 30% of the axial capacity of the first story column) was applied in the negative direction (see Fig. 3.1) and no axial load was applied in the positive direction considering the overturning mechanism of the

building. Axial load was changed when lateral drift was zero.

4. ANALYTICAL AND TEST RESULTS

4.1. Analytical Results

The specimens are analyzed assuming that the beam and joint panel are rigid according to usual design procedure. The analytical model is shown in Fig. 4.1; in this model the upper and bottom stubs are rigid. The first story column is assumed to deform in the length h_c (Fig. 4.1), which is determined as the summation of column height and one-fourth of the column depth. The term one-fourth of column depth refers to the deformation of joint panel and is determined by the AIJ standards (Architectural Institute of Japan, 2010). The curvature of the first story column is computed according to the bending moment at the critical section and is assumed to be distributed proportional to the moment distribution. The shear deformation of the first story column is computed assuming the elasticity. The wall including the stub column and the boundary column are assumed to deform in the length h_w (Fig. 4.1). Deformation of the wall is calculated similarly to that of the first story column. Lateral displacement of the loading point is determined as summation of the displacements due to wall panel and first story column deformations.

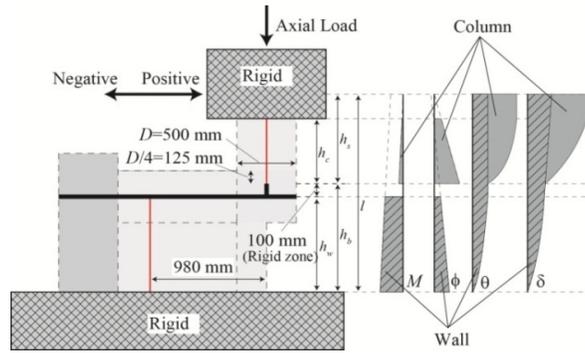


Figure 4.1 Analytical model (O-Series)

4.2. Test Results

4.2.1. Case of O-series

Load-drift relationship for O-1 specimen is shown in Fig. 4.2. Fig. 4.2a shows the relationship before 0.5% drift (axial load is about 1125kN).

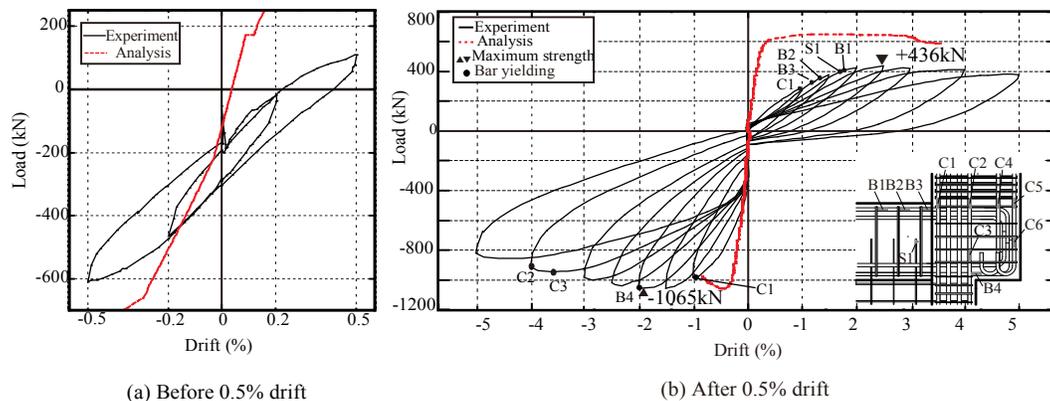


Figure 4.2 Load-drift relationship (O-1 specimen)

Note that horizontal force (120 kN in the analysis and 200 kN in the test) is required to resist the axial force and to keep the drift zero. The measured rigidity from Fig.4.2a is almost 40 percent of the analytical result. This difference indicates that the beam and joint deformed as well in this experiment, contrary to the assumption in the analysis.

Fig. 4.2b shows the relationship after 0.5% drift (axial load is zero in positive direction, and 2250kN in the negative direction), maximum strength, analytical load-drift relationship and yield of the reinforcements. The maximum strength agrees with the analytical result in the negative loading direction, but is smaller in the positive loading direction.

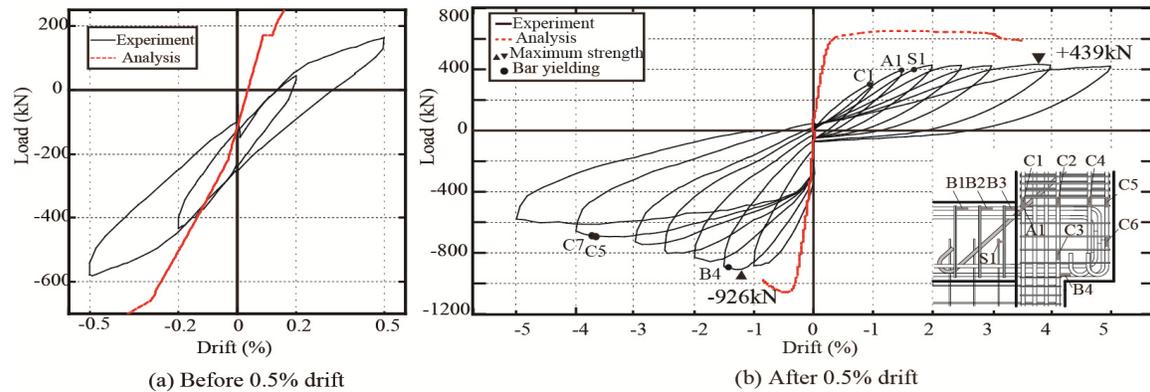


Figure 4.3 Load-drift relationship (O-2 specimen)

Load-drift relationship for O-2 specimen is shown in Fig. 4.3. Again, the measured rigidity is much smaller than the analytical result (see Fig. 4.3a). Maximum strength, analytical load-drift relationship and yield of the reinforcements are shown in Fig. 4.3b. The maximum strength of O-2 specimen in the negative loading direction is smaller than that of O-1; and is almost the same in the positive loading direction. In O-2 specimen, five inclined bars were provided at the joint instead of three vertical main bars of the first story column. In the case of O-1 specimen, it was observed that the vertical main bars were yielded in compression under negative loading (see Fig. 4.2b, gauge C1), whereas the inclined bars did not yield in the case of O-2 specimen. It can be concluded that the inclined bars were not effective to resist compressive force compared to the vertical reinforcement. Therefore, the neutral axis of O-2 specimen becomes larger than O-1 specimen which leads to smaller strength.

4.2.2. Case of I-series

Load-drift relationship for I-1 specimen is shown in Fig. 4.4. Fig. 4.4a shows the relationship before 0.5% drift and is almost same as to O-1 specimen. The observed rigidity in this case is also much smaller than the analytical result.

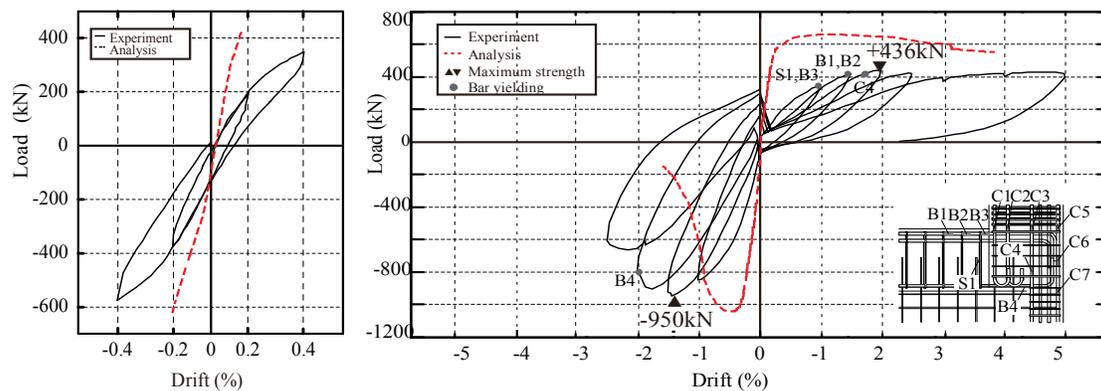


Figure 4.4 Load-drift relationship (I-1 specimen)

Load-drift relationship for I-2 specimen is shown in Fig. 4.5. Maximum strength, analytical load-drift relationship and yield of the reinforcements are shown in Fig. 4.5b. The maximum strength in both directions agreed with the analytical result. In I-2 specimen, five inclined bars were provided at the joint instead of three vertical main bars of the first story column. Comparing to I-1 specimen, maximum strength of I-2 specimen is larger in both positive and negative loading directions. It was observed that the vertical bars of the column yielded in positive and negative loading. In the case of I-1 specimen, the vertical bars did not yield neither under positive nor negative loadings (see Fig. 4.4b).

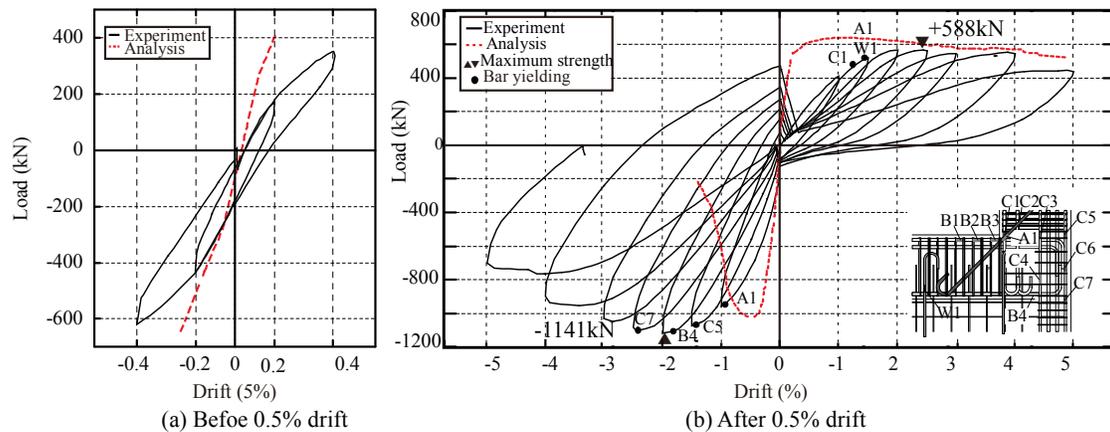


Figure 4.5 Load-drift relationship (I-2 specimen)

Fig. 4.6 shows photographs of the specimens after the test. In the case of O-1 specimen, both the column and beam bars yielded under positive and negative loading. Thus, O-1 specimen had joint failure under both positive and negative loading as denoted by A in Fig. 4.6a. The inclined cracks denoted by B were prominent under positive loading.

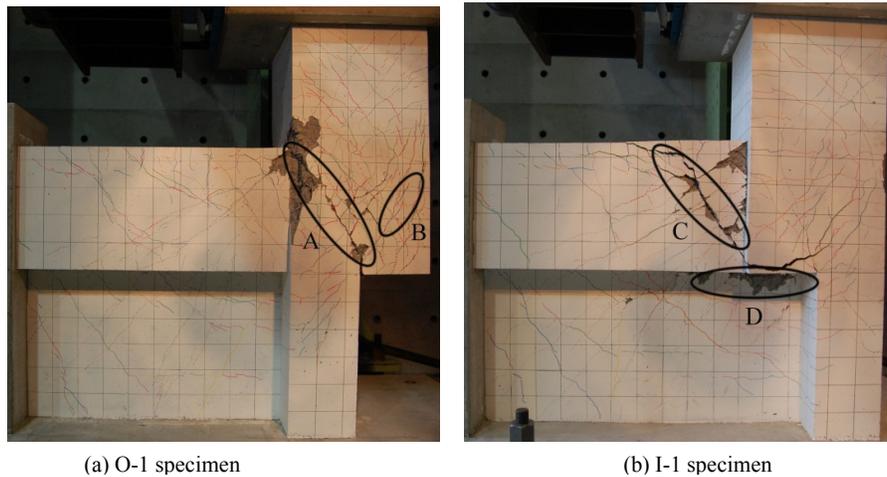


Figure 4.6 Specimens after the test

In the case of I-1 specimen, beam bottom bars yielded and beam failure occurred under positive loading denoted by C. The wall panel above the joint had crushed under the negative loading denoted by D (see Fig. 4.6b). There were enough stirrups provided along the beam in the I-2 specimen. The stirrups were effective to prevent beam failure under positive loading. Also, it can be said that the inclined bars played effective role to distribute the compressive force due to negative loading along

the wall panel, and prevented crushing of the wall panel.

5. STURT AND TIE MODELS

Strut and tie models are developed for the specimens. In this section, the strut and tie models for specimens O-1 and I-1 are briefly explained. The strut models are developed based on the observed strength Q of each specimen and the applied axial force.

5.1. Case of O-series

The strut and tie model and the cracks appeared during the test for positive loading are shown in Fig. 5.1a. The compressive force of the strut and its angle in the first story column are calculated as a resultant force of the applied lateral force Q and the axial compressive force $N+T_{AE}$, where T_{AE} is the ultimate tensile strength of column main bars (three layers in this case) of the tensile zone. The compressive strut bends at node B where the tensile force of the beam bottom reinforcement acts, and the compressive strut BC and its angle is determined from equilibrium of forces at node B and the geometric dimensions, respectively. The horizontal and diagonal struts in the beam are considered so as to satisfy equilibrium at node C.

The depth of each strut is calculated by the Eqn. 5.1.

$$x_n = \frac{C}{0.85 \times \sigma_B \times b} \quad (5.1)$$

C : compressive force of the strut σ_B : compressive strength of concrete b : width of each element

The summation of the applied lateral force Q and the horizontal compressive force at the top of the beam C_{CE} is less than the tensile capacity of beam bottom reinforcement (T_y). In this case, the beam bottom bars are very close to yield. In the test, the beam bottom bars and the first story column bars yielded.

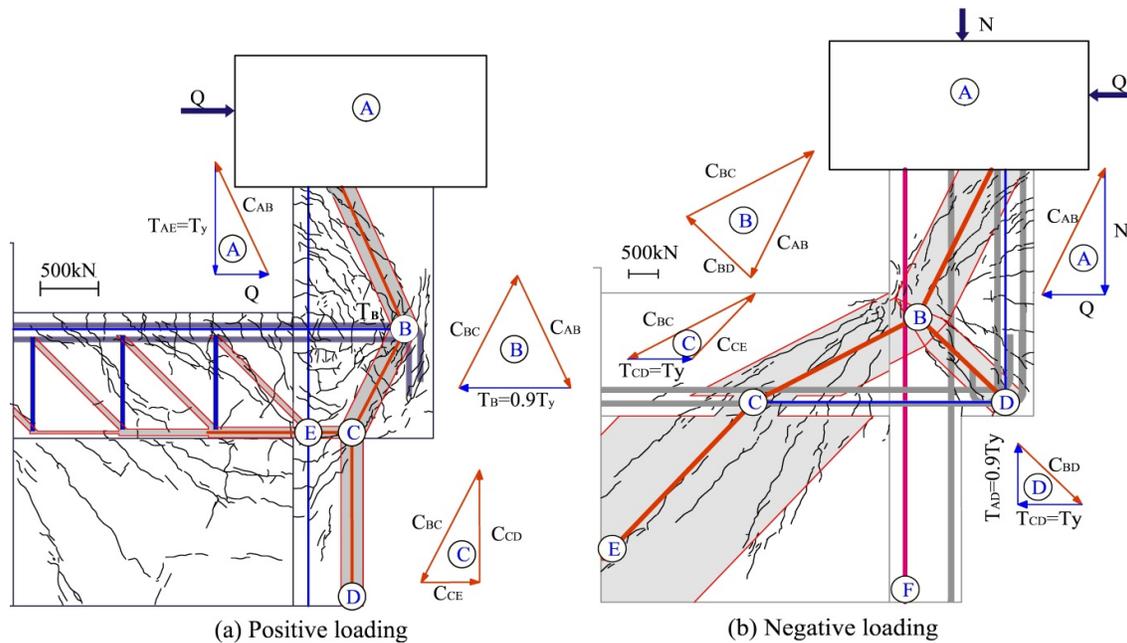


Figure 5.1 Strut and tie model (O-specimen)

For the negative loading (see Fig. 5.1b), it is assumed the compressive force in column main bars AF is equal to that of the tensile force in bars AD. The depth of strut BC is narrower in the column than in the beam because the width of the column is wider than that of the beam. The strut BD is defined by its vertical T_{AD} and horizontal T_{CD} components. The horizontal component T_{CD} is assumed equal to the tensile strength of the beam upper reinforcement. The vertical component T_{AD} is computed based on the equilibrium at node D and determined to be $0.9T_y$. In the test, the beam upper bars yielded but the column bars did not yield. Strut BC bends at node C where tensile force of the beam upper bars T_{CD} acts.

5.2. Case of I-series

The strut and tie model and the cracks that appeared during the test for positive loading are shown in Fig. 5.2a. Node B is defined at the centroid of the beam bottom reinforcement.

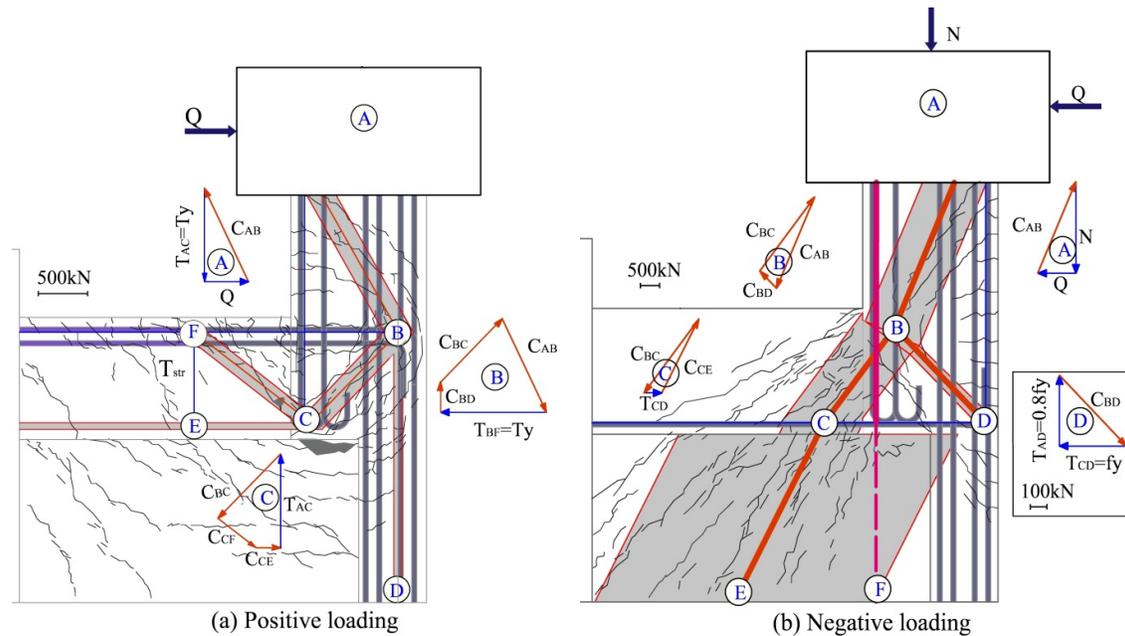


Figure 5.2 Strut and tie model (I-1specimen)

Node C is defined at the centroid of the column main bars anchored in the joint. Strut BC is defined to satisfy equilibrium of forces in the horizontal direction at node B. The struts in the beam are defined to satisfy equilibrium of forces at node C. In the test, beam bottom bars were yielded and the column main bars were close to yield.

For the negative loading of I-series (see Fig. 5.2b) procedure same as to O-series is used. It is assumed that the compressive force in column main bars AF is equal to that of the tensile force in bars AD. Strut BD is defined by the horizontal T_{CD} and vertical T_{AD} ties at node D; the required force in the vertical direction is smaller than the ultimate strength of the five bars in one layer of the first story column; thus only one layer of the column main bars is considered. In the test, beam upper bars yielded and the column bars were close to yield. It can be said the compressive force of column main bars AF is transferred to the wall panel which has 100mm thickness and caused compressive failure of the wall panel above the joint as shown in Fig. 4.6b.

The strut and tie model was used to justify and realize how the specimen behave under the loads. It seems that the strut and tie models developed for the specimens agree with the crack patterns appeared during the test.

6. CONCLUSION

Most of the specimens showed joint failure or beam failure mode rather than flexural failure of the first-floor column. The observed stiffnesses were approximately 40% of the computed ones where the boundary beam is assumed stiff and strong enough as assumed in conventional design. The difference is attributable to the deformation of the joint and the beam. The observed strengths were also smaller than those of the analysis.

The followings are concluded:

1. If inclined reinforcement is not provided at the corner of the joint, extension of the first-story column toward outside is more beneficial to increase the strength of the first soft-story than extension toward inside.
2. Inclined reinforcement at the corner of the joint is beneficial if the first-floor column is extended toward inside. But it is detrimental if the column is extended toward outside.
3. The strut and tie models developed for the specimens agree with the crack patterns observed during the tests.

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