SEISMIC BEHAVIOR OF SUBSTANDARD RC BEAM-COLUMN JOINTS DOMINATED BY CANTILEVER ACTION

Panuwat Joyklad [1] and Amorn Pimanmas [2]

[1] Lecturer, Civil Engineering Department, Kasem Bundit University,

panuwat_jo@civil.kbu.ac.th

[2] Associate Professor, Sirindhorn International Institute of Technology,

Thammasat University, amorn@siit.tu.ac.th

ABSTRACT

Based on practical design of medium to high-rise of RC building frames, the larger size of column compared to beam is common. This approach uses to ensure that column can sustain larger loads both for gravity and lateral directions. The larger depth of column with a few percentage of reinforcement, together by permanent axial compression force is enough to produce the high column flexural capacity and consequently increases the column-to-beam flexural strength ratio which is considered for capacity design concept. In addition, the larger depth of column may lead high joint shear strength and substantially increases the anchorage length which is necessary for preventing deterioration of bond during cyclic excitation. However, if the relative stiffness of column compared to the beam is too large, the expected mode of failure from beam-sway mechanism may be alternated to unexpected cantilever action mode which is uncommon in practical design. In this paper a study of two subassemblages representing interior frame at the bottom story compared to typical cruciform beam-column joint under reversed cyclic loaded is presented experimentally. The dimensions of beam-column joint for both specimens were designed identically except the column of one specimen was extended from the joint and fixed at strong foundation. The stiffness of column is designed relatively larger than the stiffness of beams. Even though the beam hingings were detected for both specimens but the global behaviors were significantly different. Behavior of cruciform beam-column joint was controlled by beams flexural strength while behavior of beam-column joint with column was controlled mainly by column flexural strength. It can be understood that the behavior of specimen with larger column compared to beams was dominated by cantilever action

rather than conventional beam-sway mechanism. In this paper, a simple index to check whenever the fame is dominated by cantilever action or by beam-sway action is proposed to classify the behavior beam-column joints.

KEYWORDS: Beam-column joint, beam-sway mechanism, cantilever action and column

1. INTRODUCTION

Under seismic excitation, columns especially in RC building frames can be vulnerable elements. The failure of column and column-sway mechanism may result in catastrophic soft story failures. To evade the sudden collapse mechanism, columns should be designed and detailed carefully. The plastic hinges shall be controlled to appear at beam ends close to column faces while columns above and below a beam-column joint should preferably remain elastic. These mechanisms can be achieved with the procedure so called “Capacity Design Philosophy” [1]. Based on this approach, to ensure that the desired weak beam-strong column develops, column-to-beam flexural strength ratio is required to have a substantial margin thus proscribing column-sway mechanisms. This design philosophy complies perfectly with traditional design of medium-to-high building frames because those structures normally have large column tributary area i.e. the size of columns is comparatively large while the beam size normally keep constant [2]. The larger size of column is also effective to provide better anchorage length to the beams bars which are passing through connection and avoidance the local failure due to the bond deterioration. Consequently, joint shear capacity is also increased automatically. However, there is no guarantee that column hinging is permanently removed. Since the test results of beam-column joint incorporating with monotonic slab showed that effective flexural strength of beams is significantly increased, hence resulting into reduced column-to-beam flexural strength ratio [3]. It is obviously understood that biaxial strength of column, mostly for rectangular shapes, is less compared to uniaxial strength and thus the reducing of column-to-beam flexural strength ratio may be induced the unconservative design of columns [4]. Therefore, various codes of practice recommend the minimum column-to-beam flexural strength ratio or overstrength factor is equal or greater than 1.0. ACI [5] suggests 1.2 for this factor while EN8 [6] and NZS [7] use 1.3 and 1.4, respectively.

However, if the relative stiffness of columns compared to beams is too high, the other unexpected failure mechanism so called “cantilever action” may be introduced especially in the first few floors of building frame [8]. A typical moment pattern for the case of beam-sway mechanism and cantilever action is shown in the Figure 1. Under occupation of cantilever action, beam hinging action do not have significant effect on behavior of buildings, therefore general capacity design procedures which normally assigned the beam as fuse of structure may not be applied directly because the weak link is already changed from beam to column base. It should be noted that even though the high values of overstrength factor ($\varphi = \sum M_{nc} / \sum M_{nb}$) are imposed while no any adjusting on relative stiffness, the column hinging still probably appear. Design shear force in those column based on a simple knowledge of beam-sway mechanism have to revise because the plastic hinges at beam ends do not mainly dominate. In addition, analytical theory based on general cruciform configuration of beam-column joint is not enough to understand this kind of action. Moreover, prior experimental dada did not provide sufficient information on the interaction behaviour of RC beam-column joint and column that was fixed at base, especially for large column stiffness compared to beams [9, 10].

In this paper experimental program was conducted to evaluate the seismic behaviour of beam-column joint specimen dominated by cantilever action compared to conventional cruciform specimen. The difference between beam-sway mechanism and cantilever action is discussed. The design recommendations for column and beam-column dominated by this action are also emphasized. Moreover, the criteria for differencing these failure mechanisms based on simple mathematical equations are proposed.

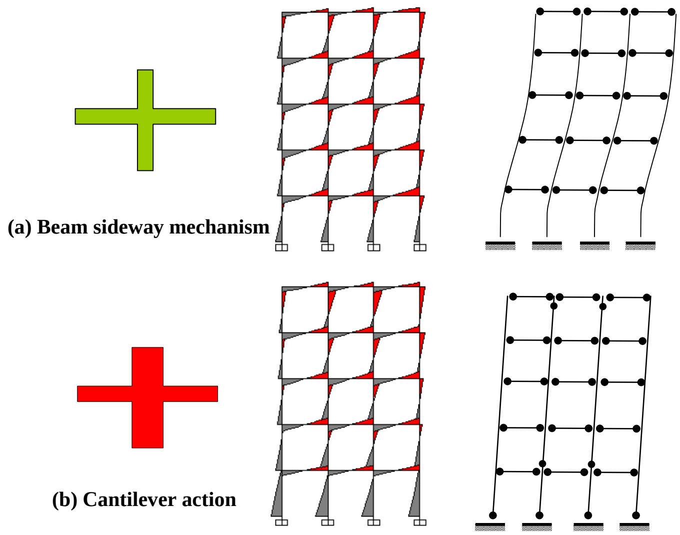


Figure 1 failure mechanisms of RC frame

2. EXPERIMENTAL PROGRAMME

Two half scale reinforced concrete beam-column joints were cast in order to investigate the distinguished behaviors between specimen with and without column extended from the joint. The first specimen was cruciform-shaped beam-column joint with the beam and column extended from joint faces to the member mid-length, where the inflection point is traditionally assumed to occur. This specimen is denoted as JL. The remaining specimen, namely CJL, had identical cross section and reinforcement details as the first specimen except the bottom column was extended and fixed at the base. These specimens are supposed to represent a part of building frame from the base to the mid-height of the second story (Figure 1 (b)); i.e., it fully includes the first floor of the building. Figure 2 shows the reinforcement and geometrical details of specimen CJL. It shall be noted that the entire length of lower column for CJL which measured from the bottom face of the joint to the fixed foundation is two times larger than the length of lower column for JL which measured from the bottom face of the joint to artificial pinned support.

The reinforcing details and structural geometry of tested specimens in this research were constructed based on collected data from surveying of structural drawing of existing buildings in Bangkok [2].The study focused on RC building frames having 5-21 stories. The building occupancy type included university, school, apartment, governmental office and hospital. The prototype buildings were designed for gravity loads based on the non-seismic provisions of the ACI318 code. These types of buildings were typically found in low to moderate region such as Thailand, Malaysia and Singapore. After the structural drawings were analyzed, there was classified into three categories based on tributary area namely small, medium and large. The tested specimens were selected from large column tributary area category in which the size of column is comparatively larger than beams. The outstanding of specimen constructed from existing structures is that the behavior of those specimens can exactly represent close to real behavior of structures.

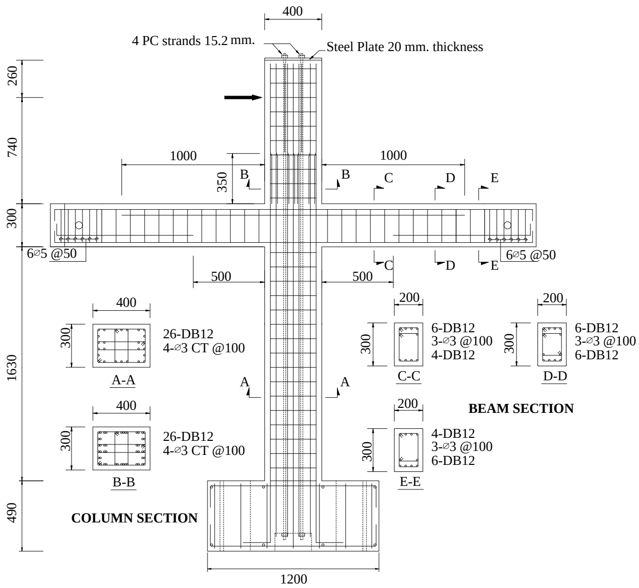


Figure 2 Configuration of CJL specimen (Unit in mm.)

The size of column in direction of loading was 300x400 mm. while beams dimension was 200x300 mm. Longitudinal reinforcement in beams and columns of both specimens i.e. JL and CJL were comprises of 12 mm. diameter deformed bars with 495 MPa actual yield strength whereas plain round steel bars of 3 mm. diameter and 385 MPa measured yield strength were using for transverse reinforcement. The normal weight of concrete of the specimen at the state of testing had average compressive cylinder strength of 26.40 MPa and 23.82 for columns and beams, respectively. Both specimens were cast in vertical direction similar to the actual practice.

The test set-up and boundary conditions are presented in Figure 3. The lateral displacement was applied at the top of the column through a 500 kN hydraulic actuator. The ends of beam were supported by rollers that allowed free horizontal movement to simulate lateral drift. The axial load of 12.5% of column axial capacity was applied to the column by means of vertical prestressing. The column was pushed forward and pulled backward in a reversed cyclic pattern with the target lateral drifts of 0.25%, 0.50%, 0.75% as shown in Figure 4. The target loop was repeated twice for each drift level. The load was continued until and beyond the peak load to trace the post-peak behavior. The tests were stopped when the load carrying capacity drop less than 80% of maximum load [11]. The strains of reinforcements during the test were measured by strain gauges attached on the reinforcement surfaces. Beams and columns rotation, shear deformation of the joint were measured by LVDTs located at specified location as shown in Figure 5.

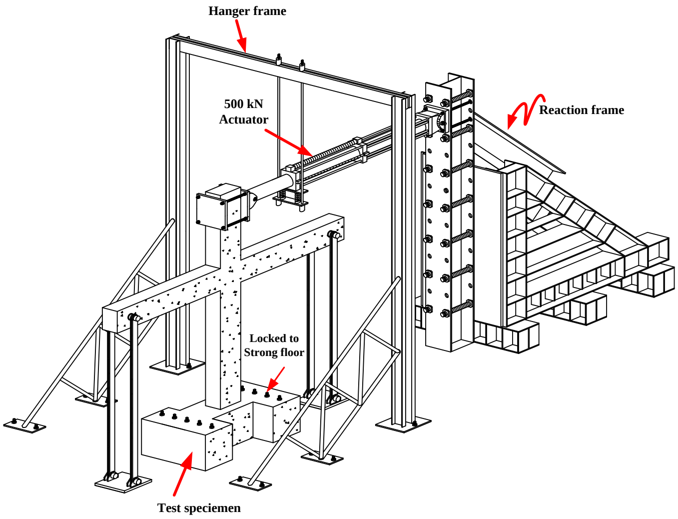


Figure 3 Schematic test set-up of CJL

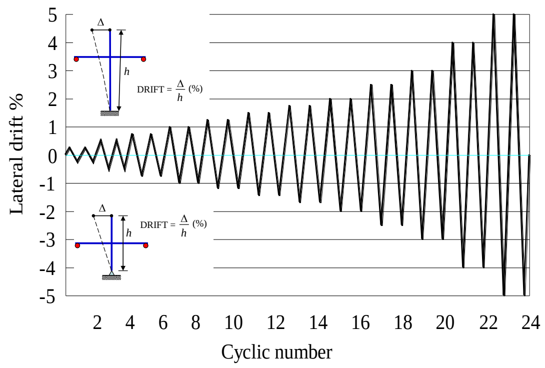


Figure 4 Displacement history of specimens

Figure 5 Instrumentation of specimen

3. TEST RESULTS AND DISCUSSIONS

3.1 Cracks development and hysteretic response

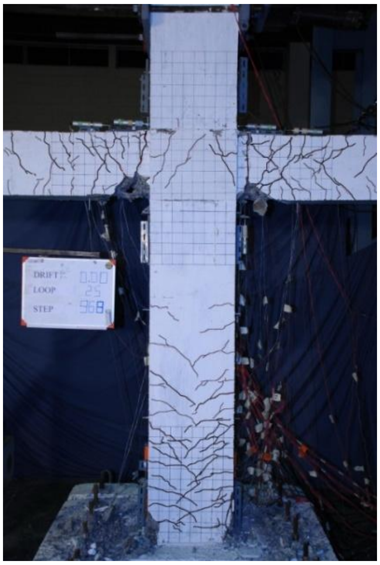
Figure 6 (a) and Figure 8 (a) show the crack patterns observed at the final stage of loading and hysteresis loops recorded during the test for both specimens.

For specimen JL, beam flexural cracks were observed at 0-0.25% drift. The behavior of specimen remained elastic until 1.00% drift (Figure 8 (a)). This agrees with the development of beam bars yielding at drift close to 1.00% (Figure 7 (a)). The cracks grew in size and number as drift increased. During the loading cycles, bond splitting cracks appeared along the longitudinal beam reinforcements, and the small diagonal tension cracking developed in the joint core at 1.25% drift. The column shear force gains up slightly while the story drifts increase rapidly. The beam maintained the yielding load until 4.50% drift when concrete at the compression fibers severely crushed and spalled off, exposing beam bars as shown in Figure 6 (a). The maximum shear force i.e. 91.91 kN was found at 4.50% drift. During the test, moreover, no cracks were observed at column bar lap splice region above the joint. Although diagonal tension cracks were found at the joint zone but the hysteresis loops was large. It reveals that the specimen failed by flexural failure from beam hinging at column faces and, moreover, specimen also presented high ductile behavior with ductility up to 4.50%. According to the columns above and below the joint performed elastic behavior while the serious beam hingings were developed therefore the failure can be classified as strong column weak beam or beam-sway mechanism which is preferable in design of RC building under earthquake loading.

For specimen CJL, the initial cracks were observed during the drift 0.0 to 0.25%. The behavior of specimen was elastic until 1.00% drift as observed in Figure 8 (b). After both beam bars yielding at 1.00% drift (Figure 7 (b)). The flexural cracks grew in size and number. The strain of longitudinal beam bars was observed to decrease rapidly (Figure 7 (b)) while the column shear force was developed continuously (Figure 7 (b) and Figure 8 (b)). At the same drift, the flexural cracks propagated around the upper and lower parts of beams close to column faces. The cracks at beam ends continued to concentrate and wider in width. The splitting cracks, moreover, along the bottom reinforcements of beams were clearly found at 3.00% drift. Up to 3.00% drift, concrete at bottom fiber of beams started to crush. The substantial damage of concrete at bottom fibers of beams were took place and spalled off, exposing beam bars. It can be seen that the vertical flexure cracks were largely developed at the location of the first stirrup which was closely placed from the column faces. These vertical flexure cracks jointed from the top and bottom fibers of beams at 2.00% drift. The initial flexural cracks in column were found at distance equal to effective depth of column, 300 mm, at 0.50% drift. The yielding of main reinforcements in column base was also recorded at 1.75% drift (Figure 7 (c)). The flexural cracks in column extended and uniformly distributed along the plastic hinge zone, around 1.5 time of column section measured from the base at 1.50% drift are shown in Figure 6 (b). At 4.00% drift, the spalling of concrete cover at column base was found and spalling was found to be increased with higher drift ratio. The cracks were found to extend towards the center of column but no significant damage appeared until the specimen reached the failure. The specimen CJL attained the peak load of 177.79 kN. at 3.00% drift as shown in Figure 8 (b). The maximum column shear force complies with crushing of concrete at bottom section of column and maximum strain of column longitudinal bars measured at column base is shown in Figure 7(c). It should be notified that at drift beyond 3.50%, bare bars at both sides of column faces were buckled and specimen lost strength abruptly. After the specimen reached the maximum load and loading starting to drop, specimen also continuously received the load until 4.00% drift with the same time of the broken of bottom longitudinal of beam bars. The testing was stop at 4.50% drift when the concrete at column base section was crushed heavily. The specimen failed together due to flexure of the beam ends and hinging of column at the base section. Throughout the test only few diagonal tension cracks were observed at the joint region. It should be noted that even though the yielding of beam reinforcements for both specimens were developed but the maximum column shear forces of these specimens were significantly different. Moreover, the cumulative cracks at beam sections close to column faces of CJL specimen were concentrated only at the bottom fibers while the cracks at the same section for JL were distributed both at top and bottom fibers.

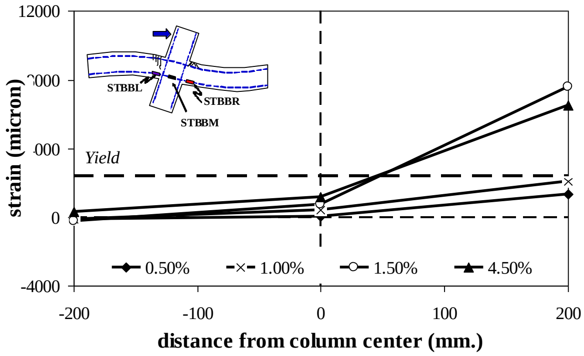
Figure 7 (c) shows the strain distribution versus drift of column longitudinal reinforcement at interface region i.e. between column and foundation whereas Figure 7 (d) shows the strain distribution versus drift ratios of column longitudinal reinforcement at top section (A section located at 50 mm. down from bottom fiber of joint) of lower column. The observations show that, from the initial drifting state, the strain distributions of both locations are moved in the same direction. At each load step, therefore, one side of column received only one kind of stress, tension or compression. As a result, the inflection point of CJL was moved closely to the center of joint while the point of inflection of JL was artificially assumed at the mid height of column. Although it cannot theoretically confirm that entire column measured from base to center of joint plays the same direction of bending moment however it can practically say that this column is dominated by single curvature behavior or cantilever action.

(a) Specimen JL



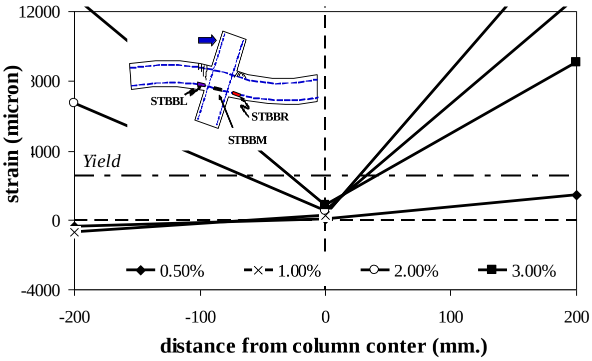
(b) Specimen CJL

Figure 6 Crack patterns of tested specimens

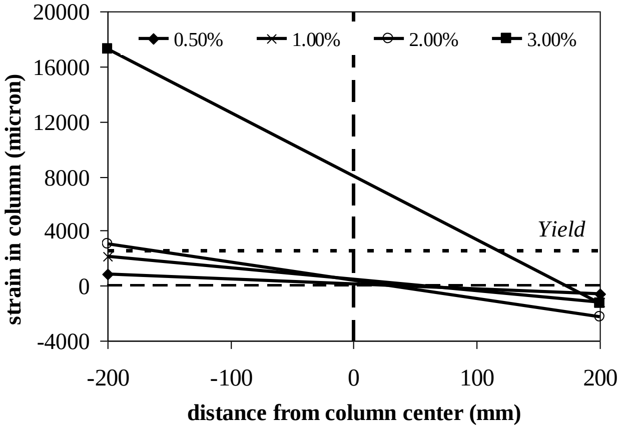


(a) Strain at bottom bar for JL

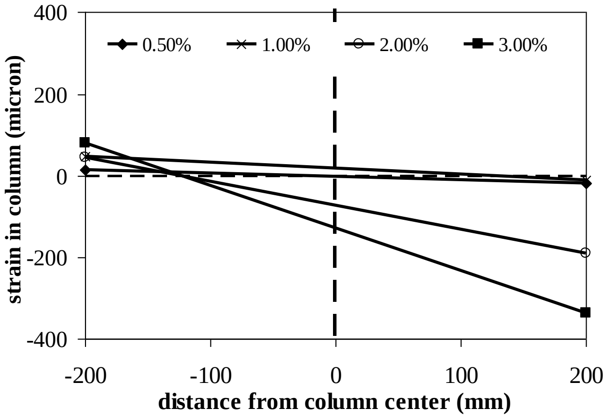
Figure 7 Strain developments at various sections of specimens



(b) Strain at bottom bar for CJL

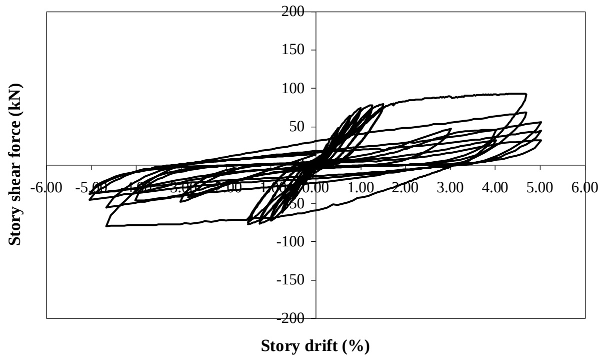


(c) Stain at column base of CJL

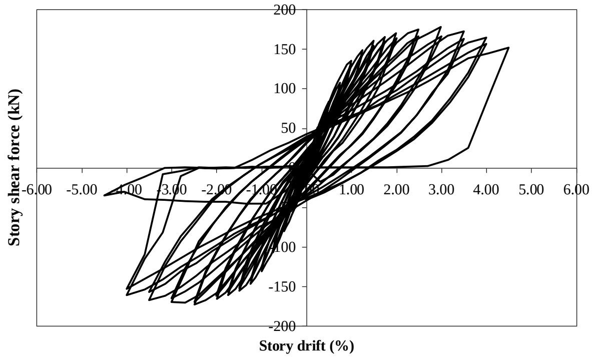


(d) Strain at top column below the beam of CJL

Figure 7 Strain developments at various sections of specimens



(a) Specimen JL



(b) Specimen CJL

Figure 8 Hysteresis loop for test specimen

3.2 Envelope of column shear force and joint shear force

In order to clearly demonstrate the development of strength and stiffness of tested specimens, the envelopes of horizontal column shear forces versus drift are plotted in Figure 9. It can be seen that strength and stiffness of both specimens are significantly different from each other.

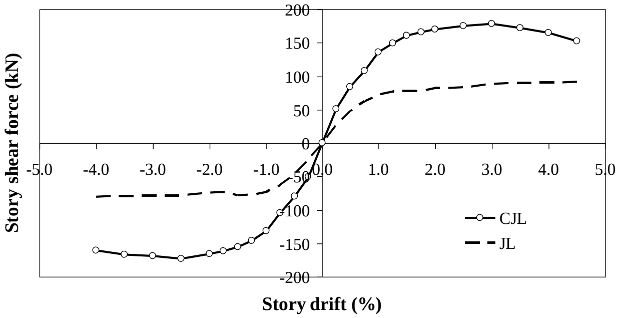


Figure 9 Envelope of column shear forces

Total increase in column shear force of specimen CJL was increased up to 122.9% as compared to the specimen JL. This may emphasize that column has large contribution to change the behavior of beam-column joint especially when it is controlled by cantilever action. Further, these results confirmed that typical test set-up of beam-column joint is not sufficient to explain behavior of beam-column joint especially in the case of first few floors of structure. It can be clearly observed from the Figure 9 that the initial stiffness, measured from the initial slope of the force-deformation curve, of CJL is larger than JL. This is attributed to the stiffness of column of CJL which controls the structural deformation is larger than beam stiffness of JL specimen.

The envelope curves of joint shear force vs. story drift are compared in Figure 10. Joint shear force was calculated using equation (1) and or (2). In these equations, the stress of beam longitudinal bars was transformed from recorded strain by Ramberg-Osgood relationship.

$$V_{ju} = T + T' - V_{col} \tag{1}$$

$$V_{ju} = A_s f_s + A'_s f'_s - V_{col} \tag{2}$$

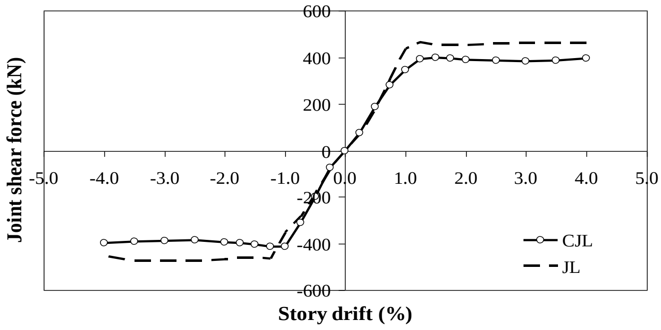
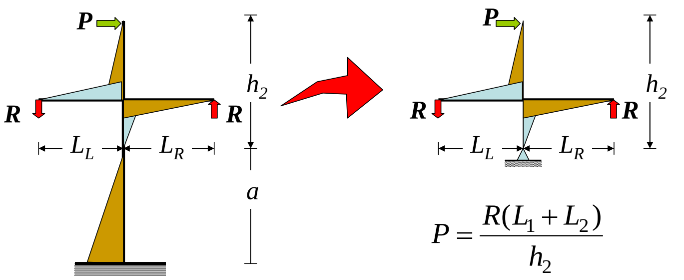


Figure 10 Joint shear forces

As presented in figure 10, the maximum normalized of horizontal joint shear stresses are 0.82$\sqrt{f'_c}$ and 0.70$\sqrt{f'_c}$ MPa for JL and CJL, respectively. The joint shear stress of specimen JL is 17.14% higher than specimen CJL. The lower joint shear force of specimen CJL even though both beam reinforcements of those specimens were reached the yield strength also reflects to one manifest of high stiffness column contributed to beam-column joint.

3.3 Influence of cantilever action to beam-column joint behavior

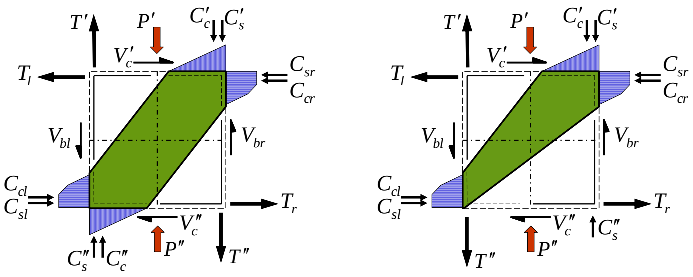
As aforementioned, there were distinct behaviors observed from tested specimens. Although both of them were design accordingly the same beam-sway concept. For CJL specimen, the dramatic increase of column force resulted from the shift up of point of contra-flexure in lower column. According to column distance (h_2) as shown in Figure 11 was shortens therefore under the constant beam flexural capacity, the column shear force can be amplified greatly.



$$P = \frac{R(L_1 + L_2)}{h_2}$$

Figure 11 Schematic moment distribution for specimens

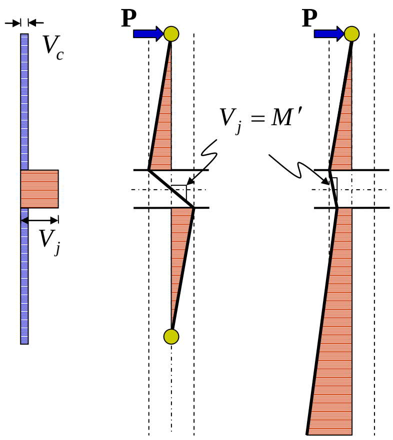
The single curvature trend of lower column not only changes the column shear force and joint shear stress as explained in 3.2 but also develops new force. It also forms the new force transfer mechanism in a joint panel. Based on conventional theory of beam-column connection in beam-sway building, the horizontal shear forces in a joint can be transferred by a combination of two mechanisms. The first is a large strut formed between the opposite corners of the joint in compression, and the second is a panel truss mechanism formed by intermediate joint ties acting as tension members and smaller inclined struts action in compression [1]. In the case of substandard beam-column joints, however, the truss mechanism cannot active due to lacking of joint shear reinforcements [12]. Therefore, in substandard beam-column joints, only large compressive strut mechanism is transferred symmetrically from one joint corner to other diagonal opposite corner (Figure 12 (a)). However, this theory is true for typical beam-column joint in beam-sway building/specimen at which the inflection point is assumed to locate at mid height of column only. In the case of beam-column joint occupied by cantilever action, the compressive stress block exerted by lower column, below the joint, vanished. Therefore the total compressive force from the top corner of the joint is transferred directly to beam compressive zone (Figure 12 (b)). The new load transfer mechanism lead the compressive damage at bottom beam section (Figure 6 (b)) and probably induced the rupture of steel bars at larger load reversal as explained in section 3.1. This is due to the reason that compressive stress is concentrated only at bottom fiber of beam sections rather than distributed to column and beam adjacent the joint corner which usually found in the case of conventional load transfer mechanism. Consequently, it can believe that the compressive force in the joint which connected to column that dominated by cantilever action has the lower value when compared to typical beam-column controlled by beam-sway mechanism due to smaller of size of compressive strut. Therefore, the induced joint shear force in the joint is relieved. The horizontal component of compressive force from diagonal strut is balanced by compressive stress result in bottom fiber of beam as shown in Figure 12 (b). The vertical component of compressive force, which is resulting in lower part of column is equilibrated by tension force of main column reinforcement (T') instead of compressive stress result in lower part of column as found in those typical beam-sway mechanism. The probability of column hinging above the single curvature column may rise, with increase in tension force in the main reinforcement of column above the joint.



(a) Beam sideway mechanism (b) Cantilever action

Figure 12 force transfer mechanisms

Alternative interpretation of shear force development in the joint core can be viewed by means of moment gradient in the joint. The moment diagrams of the column for typical column and column dominated by cantilever action are shown schematically in Figure 13. Based on structural mechanics, shear force distribution along the member can be computed by gradient of bending moment. In the case of conventional beam-column joint, the high shear in the joint can be developed typically four to six times, as observed from the higher moment gradient in the joint core, in the case of cruciform beam-column joint [8] (Figure 13 (b)). However, shear force for beam-column dominated by cantilever action is somewhat lower since moment gradient which drawn from upper to lower moment diagram of column is very small. This low value of moment gradient is due to the reason that the same side of bending moment diagram are presented in the top and bottom columns (Figure (c)).



(a) (b) (c)

Figure 13 Bending moment diagram of column (a) shear force

(b) beam-sway mechanism (c) cantilever action

3.4 Influence of cantilever action to shear design of column

Under cantilever action, the capacity design philosophy which assigned the beam hinging at column faces as a fuse of structure cannot be applied individually. It can be seen from experimental results of CJL, although the beam hingings are started but column shear force is still increased. To demonstrate the importance role of moment capacity at column base of cantilever action structure, the equilibrium of forces at ultimate state are presented in Figure 14.

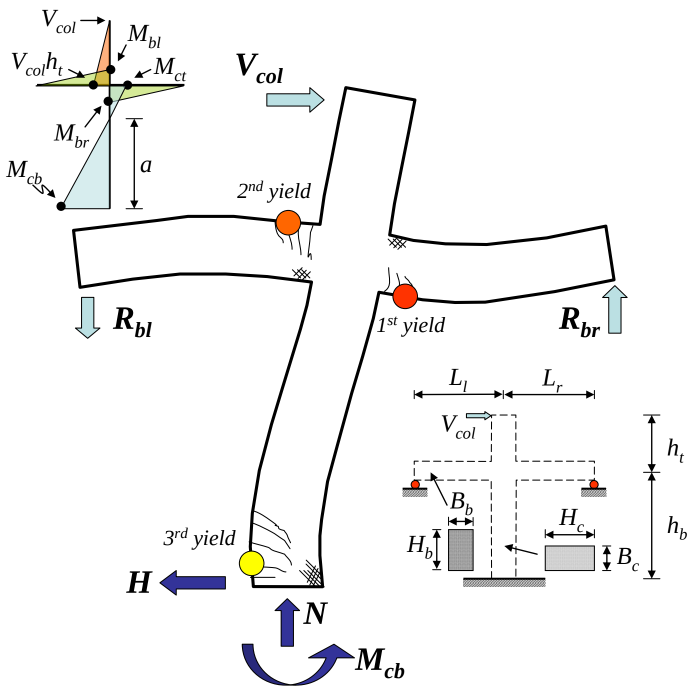


Figure 14 Equilibrium of specimen at ultimate stage

Based on figure 14, the column shear force at ultimate state can be computed as follows;

$$V_{col} = \frac{R_{bl}L_l + R_{br}L_r + M_{cb}}{H} \tag{3}$$

where R_{bl} and R_{br} are the reactions corresponding to nominal moments of beam at the left and right section of column faces, respectively. The M_{cb} is nominal moment of column at base section and H $(= h_t + h_b)$ is the total height of specimen i.e. summation of h_t

and h_b. However, if column dimension is neglected, therefore, equation (3) can be simplified conservatively as

$$V_{col} = \frac{M_{bl} + M_{br} + M_{cb}}{H} \tag{4}$$

where M_{bl} and M_{br} are the nominal moments of beam at left and right section of column faces, respectively. Equation (4) presents the column shear force in term of moment capacity at critical sections. The mechanism starts when the 1st yielding of beam section based on positive moment at column face takes place and follows by yielding of beam section based on negative moment. When beams section developed the plastic moments, force demand will be restored equal to sectional capacity until the ultimate capacity of sections achieved (Figure 14). Aforementioned, even though beam hinges are developed, however, the structure do not lost its capacity suddenly due to the reason that structure still possesses some strength and stability. This remaining strength and stability due to last degree of determinacy which took place on section column at base. The column shear force can develop further until section of column at base meet the ultimate capacity. On another hand, the global behavior of specimen is controlled by flexural capacity of column at base.

Generally, column hinges at the 1st floor of beam-sway building have been achieved by various standards [6, 7]. This column shear force (V_{uc}) typically presents in the form of following equation

$$V_{uc} = \frac{M_{nc,top} + M_{nc,bot}}{\phi_s L_c} \tag{5}$$

where V_{uc} = column shear force corresponding to the simultaneous development of the anticipated maximum moments at both ends of the column, acting at the same sense; $M_{nc,top}$, $M_{nc,bot}$ = column moment capacity at top and bottom ends of the column (based on increased yield stress primary to recognize strain hardening in the reinforcement); and L_c = clear height of the column. ϕ_s is reduction factor for shear recommended by code, for example ACI [5] use 0.75. Equation (3) implies that column shear force in the lowest column of the building frame may be designed based on simultaneous hingings of column sections. This is true especially for beam-sway building. Moreover, design by equation (3) presents very conservative approach and, therefore, use only for high seismicity. However, in the case of cantilever action where column have large column bending capacity, the use of equation (5) is not economical since the probability of both column hingings at the same column may not easily to occur. The design shear force, therefore, in column should be design based on single hinging at column base as follow.

$$V_{uc} = \frac{\beta M_{n,bot}}{\phi_s L_c} \tag{6}$$

To prevent unexpected mode of failure, however, Paulay and Priestley [8] suggested β = 1.50 for practical propose. The recommendation by equation (5) can be verified from test result of CJL. Since the flexural capacity of column (M_{nc}) computed based on ACI318 approach equal to 288.25 kN-m therefore shear force in column, V_{col} = 288.15/1.63 kN when 1.63 is clear height of bottom column (Figure 2). It can be seen that the computed column shear force by mean of equation (6) is close to column shear force obtained from the experimental result (Figure 9).

3.5 Influence of cantilever action to column hingings

As aforementioned, the capacity design based on beam-sway mechanism desires hinging of beam ends close to column faces rather than at column end sections. To achieve this concept, therefore, the flexural strength of column to beam is controlled to greater than 1. In the case of cantilever action, however, the column hingings, especially for first few floors, can be appeared even though the column to beam strength ratio is too high. Hence, a column section may receive the moment higher than the average sum of beam moments at the same connection which is mostly expected during primary design step. Consequently, in the case of cantilever action, a flexural demand shall be calculated based on the new joint equilibrium concept which is expressed below

$$M_{uc} \geq \frac{\lambda \sum M_{nb}}{\phi_c} \tag{7}$$

Where $\sum M_{nb}$ and M_{uc} are the sum of moment capacities of beam sections around joint along loading directions and moment in the column, respectively. Factor λ and ϕ_c are distribution factor and reduction factor for column as recommended by codes. For general beam-sway mechanism, factor λ equal to 0.5 while ϕ_c equal to 1.0. It shows that the summation of moment around joint are shared equally for upper and lower column sections.

In the case of higher mode response or cantilever action, factor λ may different from 0.5. Kelly [13] suggested that the value for λ could be set around 0.80 to 1.3, the higher value being observed for a rather flexible frame and the smaller value being representative of a relatively rigid frame. For general propose, Park and Paulay [1] suggested to use $\phi_c = 0.9$ therefore the design moment in the column under these situation may be taken as $M_{uc} \geq 0.98 \sum M_{nb}$ and $M_{uc} \geq 1.58 \sum M_{nb}$ for λ = 0.8 and 1.3, respectively.

4. CRITERIA FOR CANTILEVER ACTION

The problem deal with cantilever action cannot be performed economically by simple design procedures (i.e., beam-sway mechanism). Therefore the selection of design procedure is depend upon the behavior of frame and if the designer found that frame is controlled by cantilever action, the design procedures as recommended from 3.3 to 3.5 shall be applied. However, the problem is how we can roughly check whether the designed frame is controlled by cantilever action or not. Here an easy and simple equation for differentiating frame behavior is presented. It is required especially in the case of primary design. All indices for formulating of mentioned criteria are exhibited in Figure 14.

The formulation approach is performed by simple virtual force method in which, column shear force (V_{col}) is selected as redundant and assumed and uniform modulus of Elasticity, E is assumed for all members. The reactions at beam ends can be expressed as follows.

$$R_{br} = \frac{3V_{col}L_L h_b I_b (h_b + 2h_t)}{2L_r(L_l L_r I_c + 3L_l h_b I_b + 3L_r h_b I_b)} \tag{8.1}$$

$$R_{bl} = \frac{3PL_r h_b I_b (h_b + 2h_t)}{2L_l(L_l L_r I_c + 3L_l h_b I_b + 3L_r h_b I_b)} \tag{8.2}$$

The end moment of top column is,

$$M_t = P \times h_t \tag{9}$$

The moment at each end of bottom column can be computed by equilibrium at the joint; $\sum M_{joint} = 0$;

$$M_t + M_{ct} - M_{br} - M_{bl} = 0 \tag{10}$$

When M_{bl} and M_{br} are moments in beam at column faces which can be calculated by multiplying R_{bl} and R_{br} by L_l and L_r, respectively. From equation (10), moment at top end of bottom column can be expressed as,

$$M_{ct} = \frac{P(3L_l h_b^2 I_b + 3L_r h_b^2 I_b - 2h_t L_l L_r I_c)}{2(L_l L_r I_c + 3L_l h_b I_b + 3L_r h_b I_b)} \tag{11}$$

The moments at bottom end, base, of bottom column can be computed from equilibrium from global system.

$$P(h_t + h_b) + M_{cb} - R_{bl}L_l - R_{br}L_r = 0 \tag{12}$$

Thus,

$$M_{cb} = \frac{P(2h_t L_l L_r I_c + 2h_b L_l L_r I_c + 3L_l h_b^2 I_b + 3L_r h_b^2 I_b)}{2(L_l L_r I_c + 3L_l h_b I_b + 3L_r h_b I_b)} \tag{13}$$

From (11) and (13), the shear span can be calculated base on similar triangle.

$$a = \left(\frac{M_{cb}}{M_{ct} + M_{cb}} \right) h_b \tag{14}$$

Substituted (11) and (13) into (14), thus

$$a = \frac{2h_t L_l L_r I_c + 2h_b L_l L_r I_c + 3L_l h_b^2 I_b + 3L_r h_b^2 I_b}{2(L_l L_r I_c + 3L_l h_b I_b + 3L_r h_b I_b)} \tag{15}$$

For simplicity, the beam lengths, L_l and L_r, are assumed as L. Hence,

$$a = \frac{(h_t + h_b) L I_c + 3h_b^2 I_b}{L I_c + 6h_b I_b} \tag{16}$$

Above expression implies that if the length of shear span (a) approach to $h_b - \dfrac{H_b}{2}$ then the bending moment diagram of bottom column has single sign convention. Hence, the frame is dominating by cantilever action. On another hand, if the shear span is higher than $h_b - \dfrac{H_b}{2}$, it's mean that column of building frame is fully governed by cantilever action. Because h_t is assumed equal to half of clear height of second floor. Therefore, it shall be noted that equation (16) is correctly applied if and only if the inflection point of upper column is not greater than half of its clear height. For convenience, the cantilever action may be assumed to present at the first floor story and h_t is placed at half of second floor column height. Beyond this assumption, a small error may be occurred using equation (16) but it also still in acceptable range. Hence, for summarization, equation (16) can be rewritten as

$$\frac{(h_t + h_b) L I_c + 3h_b^2 I_b}{L I_c + 6h_b I_b} \geq h_b - \frac{H_b}{2} \tag{17}$$

If equation (17) is true, it means that the bottom column behave as cantilever action. In contrast, bottom column may be controlled by double sign convention and beam-sway mechanism is still effective. Moreover, equation (16) was derived based on elastic analysis. In the case of large deformation of structure, the plastic hinge in beam ends can move up the inflection point a bit. According to nonlinear analysis of frame dominated by cantilever action, it can be found that approximate 20% of column shear span may be increased after both plastic hinges of beam were developed. Here, based on experimental data the left hand side of equation (17) shows 1779.44 mm. while the right hand side is 1630 mm. Hence equation (17) is true and column presents cantilever action. This small calculation shows how well of proposed equation compared to the test results.

5. CONCLUSION

This paper presents the experimental results of two half scale reinforced concrete beam-column joints under reversed cyclic loading. The size of columns is relatively large as compared to the beams. These tested joints are representing for large tributary area category which is usually found in medium-to-high rise buildings. The specimens were designed with identical dimension and reinforcing detailing except that the column was extended and fixed at base for second specimen. The specimen with extended column was constructed for investigating the behavior of beam-column joint in 1st floor story compared to typical cruciform test specimens. The test results found that those specimens behave with different manner. Behavior of cruciform beam-column joint specimens was controlled by beams flexural strength while behavior of beam-column joint with extended column was controlled by column flexural strength so called “cantilever action”. This action may be found in structure with low relative stiffness of beams compare to columns. By domination of cantilever action, the behavior of beam-column joint cannot be explained by simple theory of beam-sway concept.

1) The design column shear force of cantilever dominated column can be reduced safety. However, the transverse reinforcement for design shear should be checked together with required for confined concrete because the height of inflection point is moved along the height of column.

2) The symmetrical strut mechanism in the joint core for typical beam-sway concept was changed to tapered strut according to compressive part of lower column was

vanished. To maintain the equilibrium of the joint, however, the tensile force in column significantly increased because it has to compensate the vertical part of compressive strut. Therefore, the probability of upper column hinging can be observed.

3) Without scrupulous analysis of structure, there is difficult to differentiate the cantilever action and beam-sway mechanism. Consequent, authors propose a simple criterion for evaluating the cantilever action based on standard properties of member adjacent the joint at 1st floor column of structure. With proposed evaluation criteria, the cantilever action can be primarily detected and special detail design can be planed carefully.

Acknowledgement

The authors are very grateful to The Royal Golden Jubilee Ph.D Programme (PHD/0050/2550) for providing the research fund to carry out the research.

References

[1] Park, R. and Paulay, T. (1975). **Reinforced concrete structures**. New York : John Wiley & Sons,.

[2] Chaimahawan P. and Pimanmas A. (2006). "Seismic vulnerability of existing reinforced concrete buildings in Bangkok". **Proceedings of the 5th International Symposium on New Technologies for Urban Safety of Mega Cities in Asia (USMCA2006).** 2006. Thailand :545-555.

[3] Ehsani, M. R. and Wight, J. K. (1985). "Effect of transverse beams and slab on behavior of reinforced beam-to-column connections". **ACI Structural Journal.** March-April 1985. Vol.82 (No.2) : 188-195.

[4] Leon, R. and Jirsa, J. O. (1986). "Bidirectional loading of R. R. beam-column joints" **EERI Earthquake Spectra**. May 1986. Vol.2 (No.3) : 537-564.

[5] American Concrete Institute. (2008). **Building code requirements for reinforced concrete-ACI318-08.** ACI. Detroit. Michigan.

[6] European Standard. (2003). **Eurocode 8: Design of structures for earthquake resistance-Part 1 : General rules, seismic actions and rules for buildings.** EC8. Brussels.

[7] Standard Association of New Zealand, (1995).**Code of practice for design of concrete structures-NZS 3101, Part 1.** SANZ. Wellington.

[8] Paulay, T. and Priestley, M. J. N. (1992). **Seismic design of reinforced concrete and masonry buildings**. New York : John Wiley & Sons,.

[9] Sugano, S. and Nagashima T. (1985). "Structural Design and Research of Reinforced Concrete Tall Buildings". **Proceeding of the Seminar on Reinforced Concrete High-Rise Buildings in Japan with Special Concern on Aseismic Design of Beam-Column Joints (Kurose Y. and Jirsa J. O. (eds)).**1985. Japan : 85-129.

[10] Chiba, O., Fukuda, T., Toritani T., Kikuta S., Teramoto T. And Kihara, H. (1992) "Experimental and Analytical Studies on the Seismic Behavior of Reinforced Concrete Frame-wall structural Building". **Proceedings of the 10th World Conference on Earthquake Engineering.** 1992. Balkema, Rotterdam. : 3245-3251.

[11] Committee Euro-International Du Beton. (1996). **RC frames under earthquake loading, CEB**. Switzerland : Thomas Telford.

[12] Hakuto, S., Park, R. and Tanaka, H. (2000). "Seismic load tests in interior and exterior beam-column joints with substandard reinforcing details". **ACI Structural Journal.** January-February 2000. Vol.97 (No.1) : 12-25.

[13] Kelly, T. E. (1974). "Some seismic design aspects of multistory concrete frames". **Master of Engineering Report**. University of Canterbury. New Zealand.

Panuwat Joyklad is a lecturer in the Department of Civil Engineering, Kasem Bundit University, Bangkok, Thailand. He is a secretary and member of sub-committee of Structure and Bridge of Engineering Institute of Thailand. His research activities cover a wide range of topics related to concrete structures such as advanced behavior of reinforced concrete structures, reinforced and prestressed bridge design and blast resistance of RC structure.

Amorn Pimanmas is Associate Professor in the School of Civil Engineering and Technology, SIIT, Thammasat University, PathumThani, Thailand. He is a chairman of sub-committee of Structure and Bridge of Engineering Institute of Thailand and also member of Council of Engineer of Thailand. He has been responsible for the design of a number of bridges and structures in Thailand and oversea-countries. He is currently involved in research for seismic retrofit and advanced behavior of concrete structures.