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QUADRUPLE FLEXURAL RESISTANCE IN R/C BEAM-COLUMN JOINTS

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SUMMARY

This paper presents a new concept of quadruple flexural resistance in monolithic R/C beam-column joints. It is a behavior model to portray the resistance of beam-column joints to the moments coming from the members framing into the joint with four pairs of resultants on diagonal sections. Joint shear failure of beam-column joints is redefined as the failure of quadruple flexural resistance. The quadruple flexural resistance are implemented by a mathematical model. The model satisfies equilibrium equations relating magnitude of external forces such as column shear, beam shear and axial force in column, to the magnitude of stress resultants. Failure criteria of concrete, steel, bond and anchorage are combined with the equilibrium conditions to predicts the strength and failure type of beam-column joints including interior, exterior as well as knee joint, in consistent and unified way. Maximum joint shear strength is predictable by this model if the dimension, geometry and material properties are provided. Failure type is also predictable. Correlation studies for joint shear strength are shown in this paper to demonstrate that this model is a rational tool for understanding the behavior of R/C beam-column joints and it is superior to the existing models of strut and truss which regard joint shear failure as a failure of shear resistance residing in beam-column joints.

INTRODUCTION

The design of beam-column joints is an important part for earthquake resistant design of reinforced concrete (RC) moment-resisting frames. Considering normal practice in a structural analysis to assume that the beam-column connection is rigid and also recognizing the importance of the beam-column connection for structural integrity, the beam-column connection must be provided with stiffness and strength sufficient to resist and sustain the loads transmitted from beams and columns. In particular, the use of longitudinal reinforcing bars with high strength or large diameter in a relatively smaller column section, sometimes preferred in the design of buildings, causes high stress in the beam-column connections. A structural designer should carefully examine this increased stress in beam-column connection, or problems related to strength and/or stiffness may result. For example, the development of a lower strength than that based on full flexural strength of beams may result. Furthermore, a reduction in stiffness due to the formation of diagonal cracks and local crushing of concrete may occur. Thus failure of a beam-column connection must be pre-

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cluded to preserve the structural integrity of members jointed as rigid and strong as they are assumed in a structural analysis.

About a quarter century ago, the models of strut and truss were introduced by Paulay et al. [1, 2, 3] to understand the behavior of beam-column joint. The strut models consist of a single diagonal compressive strut held in equilibrium by forces developed in the horizontal and vertical tension ties. If longitudinal reinforcements are deformed and joint core is confined with joint shear reinforcement, joint shear resistance is also developed by truss action. The models inevitably leads to an view of failure observed in beam-column joint that the joint shear failure is failure of the strut or truss by crushing of the strut, by yielding of confining steel of joint or failure of anchorage of ties.

Although the strut and truss model qualitatively describes some of the variables influencing the strength and failure type of beam-column joints, including, (1) the amount of available joint reinforcement, (2) the bar size anchored through joint and (3) the concrete strength, etc.. However the model is useless to consider the effects of other critical factors such as the magnitude of column axial load, the flexural strengths of the members framing to the join etc. Prediction of the shear capacity of beam-column joint is impossible by the model otherwise the width of diagonal concrete strut is assumed correctly. But no researchers succeed to give reasonable method to determine it. The crucial deficiency of this model is that it gives no reasonable explanation of the empirical facts that the allowable joint shear stress of beam-column joint needs to be adjusted reflecting the number of framing elements into the beam-column joint for interior, exterior or corner (knee) joints. No theories or models have been agreed upon for use in practice for the design of RC beam-column joints in spite of the importance of this issue.

On the contrary, recent seismic design recommendations for RC structure, such as ACI 352 [4] and the AIJ design guidelines [5] provide empirical allowable joint shear stress to control failure type. They clearly recognize that allowable shear stress is smaller for exterior or knee joints for preventing from joint shear failure. The allowable shear stresses for exterior and knee joint are decreased by factors 0.75 and 0.60 respectively to that of interior joint in ACI-ASCE 352 [6]. The AIJ Guidelines [5] adopts reduction factors of 0.70 and 0.40 respectively. Because these factors for reduction are empirical, there is no agreement on the value from country to country. The reason of that reduction is sometimes argued that it is by recognition of the confining action and its beneficial effect on joint strength imparted by members framing into the joint. However such large enhancement in allowable shear stress is very doubtful to attribute such large variation only to the increase in concrete strength due to the confinement.

Recently a new and unique model was proposed by the author [7]. It was reported that in the test of beam-column joints, joint shear force and story shear force were not proportional, based on an investigation on the test of interior beam-column joints subjected to statically cyclic loading exhibiting joint shear failure (i.e., large joint shear deformation and degradation in story shear were observed). In most specimens, shear in the beam-column joint did not degrade until the end of test, while story shear degraded. In other words, the shear-resisting system in the beam-column joint reserves remaining shear strength until the end of the test. Because the existing models, such as models of strut and truss does not reflect this fact, a new behavior model was proposed to reflect the fact. The model is based on divided triangular segments, rotating due to bending moment coming from beams and columns. It was concluded from the preliminary analysis using the model that the new model is consistent with the observed behavior of interior beam-column joint. The model was then demonstrated how it works in predicting the moment-resisting capacity of beam-column joints taking into account the variety of parameters which are considered to have critical effects of behavior of interior beam-column joints.

This paper attempts to portray a rather simple, comprehensive and unified theory in which joint shear failure for all types of beam-column joints with different geometries, which intrinsically incorporated in the theory, by extending the preliminary study of the new model for interior beam-column joints by the author[7]. This theory require no empirical assumptions accounting the difference in strength and type of failure despite of various geometry of beam-column joints. However, due to the shortage of space to explain into detail, this paper introduces only the fundamental, but general assumptions and result of some correlation study of the new theory are presented to demonstrate the promising feature of this theory.

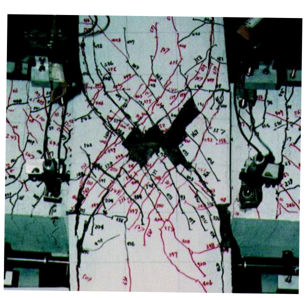
QUADRUPLE FLEXURAL RESISTANCE IN BEAM-COLUMN JOINTS

Behavior Model

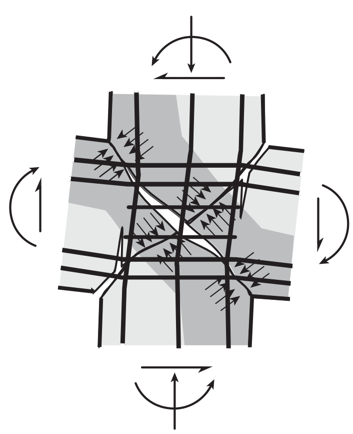
The new behavior model for shear failure of an interior beam-column joint is illustrated in Fig. 1. In the test, two diagonal opening of cracks usually becomes dominant as shown in Fig. 1(a). So in this model, shear deformation in the joint is assumed primarily due to the rotation of the four triangular concrete segments and the crack opening. Thus rotational movements of the segments cause uneven opening of the diagonal cracks like flexural cracks at the diagonal boundaries of the segments. When moment is applied to the segments from adjacent beams and columns, rotation of the segments occur as shown in Figs. 1(b) and 1(c) for interior and exterior joint respectively. Because in this behavior model, four sets of flexural resisting action are identified in the moment resistance of the beam-column joint, it is named as quadruple flexural resistance. This behavior model meets the requirements to account for the realistic behavior of the joint shear failure, in which shear deformation of the beam-column connection increases while the remaining shear-resisting capacity of the connection is reserved [7].

Principles of the Theory

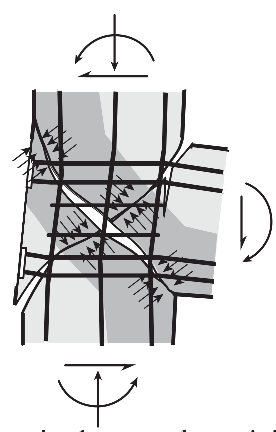
Although the quadruple flexural resistance plays an essential role for defining the joint shear failure, the overall behavior of a beam-column joint subassembledge is not governed only by the joint shear failure. In design, by achieving desirable hierarchy of the strength, beam-column joint need to be designed such that the contributions to total story drift due to shear deformation in the joint should be minimized. Therefore the objective of the design is primarily to avoid ill behavior of the moment-resisting frame in the design due to large shear deformation of the joints. So it is necessary to incorporate a method to control the shear deformation into the quadruple flexural resistance. Hence a more comprehensive and mathematical models for beam-column joints were developed [8]. The model considers two sets of critical sections associated with two independent deformation modes, called J-mode and B-mode. J-mode is equivalent to the quadruple flexural resistance and featured by the diagonal crack as shown in the Fig. 2. When a beam-column joint is loaded, local curvature arises at the critical sections. This action causes observed joint shear defor-



(a) typical appearance of joint shear failure



(b) interior beam-column joint



(b) exterior beam-column joint

Figure 1: Behavior model for joint shear failure [7]

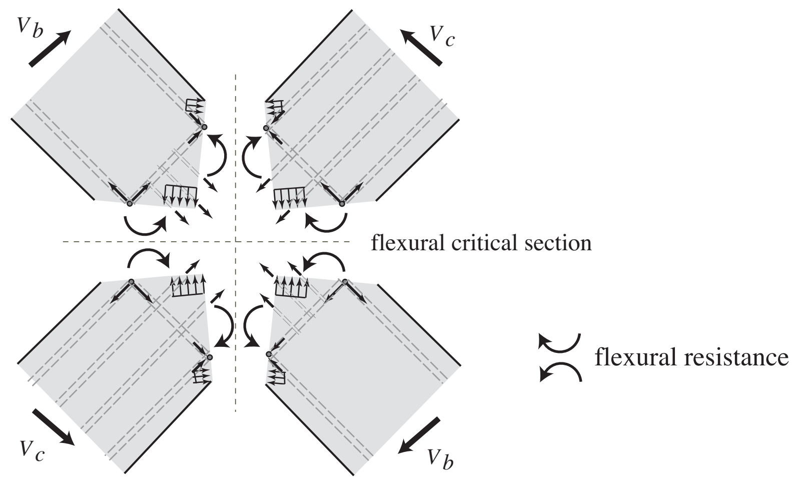
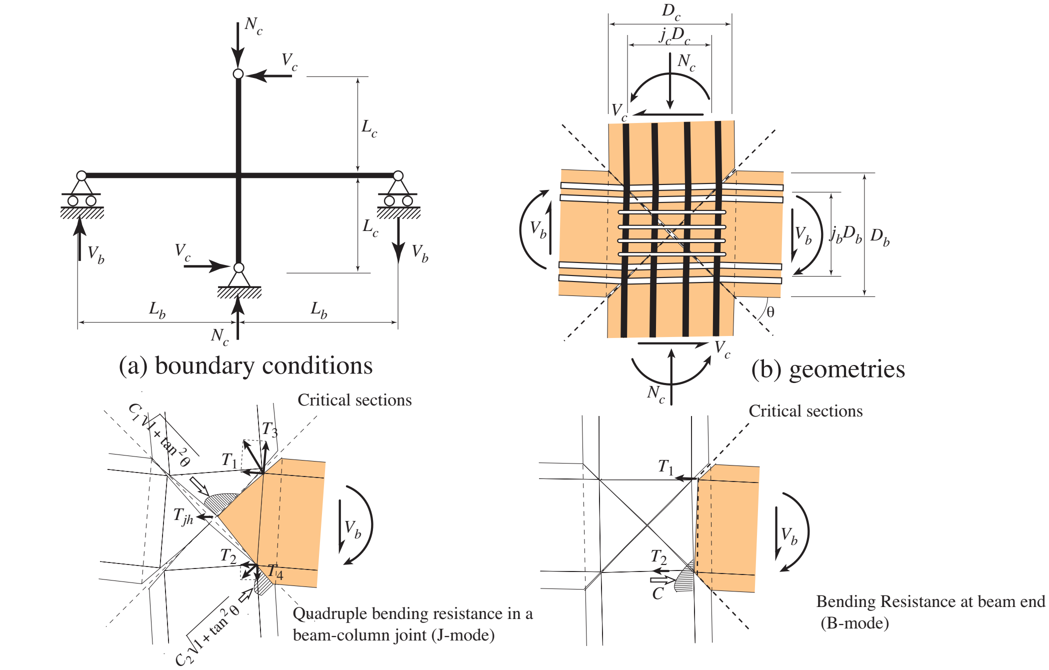


Figure 2: Mathematical model for quadruple flexural resistance (J-mode) [8]

mation. So this type of resistance and associated deformation mode is hereafter called J (joint)-mode. If the J-mode deformation increases, concrete will crush and cover concrete will spall off adjacent to the crossing point of diagonal cracks. Whereas, B-mode is featured by the cracks along the column or beam face. The increase of local curvature due to B-mode deformation primarily causes rotation at the end of beam of column.

The objective of the introduction of the two modes is to obtain two joint shear strengths calculated based on the J-mode and B-mode. The two modes of J-mode and B-mode usually give different values for strength of joint reflecting the difference in choice of critical sections. The smaller value of strength is interpreted the real strength and its mode is dominant deformation mode. Thus, by comparing the magnitude of the two strengths, prediction of dominant deformation mode is feasible.

Hence the analytical methods to calculate the strength of the both modes are necessary. The method of calculation of the strength is based on similar principle of the classical flexural theory where section curvature causes moment resistance by a pair of force resultants in tension and compression arising in reinforcing bars and concrete. This theory adopts slightly different assumption from classical flexural theory on the point that the assumption of plane section remains plane is not used. It is because local slip-in and slip-out of longitudinal bars have critical effects on the real behavior of the beam-column joint as pointed out by Hakuto et. al [9,10]. As a result, the influence of the bond capacity of longitudinal bar is included as one of parameter. The result of analysis on the effect of bond capacity for interior beam-column joints using this model has already been reported in the reference [8]. It has been also extended recently to the exterior beam-column joint [11] and knee joint [12]. Although they are not discussed here in details, the models implemented for interior, exterior and knee joints are depicted in Figs. 3, 4 and 5 respectively. Each illustration shows how two sets of critical sections and their inherent stress field as well as deformation pattern are selected.



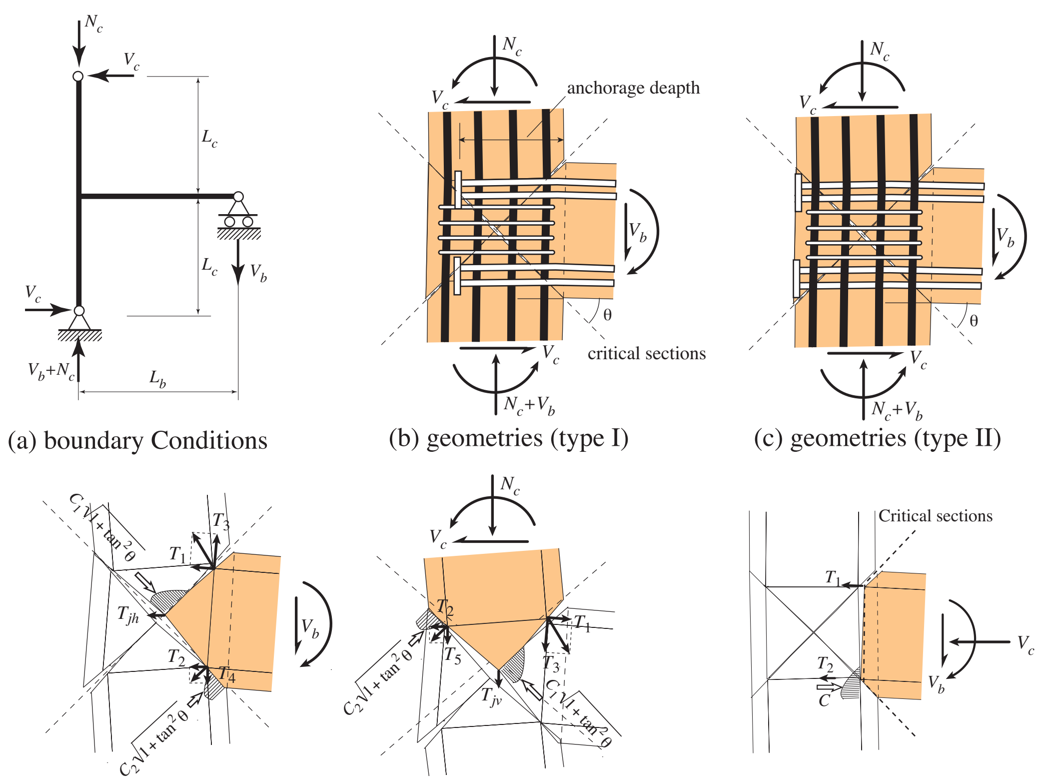
(a) boundary conditions

(b) geometries

(c) J (Joint)-mode resistance and deformation

(d) B (beam) -mode resistance and deformation

Figure 3: Model for interior beam-column joints [8]



(a) boundary Conditions

(b) geometries (type I)

(c) geometries (type II)

(d) J-mode

(e) B-mode

Figure 4: Model for exterior beam-column joints [11]

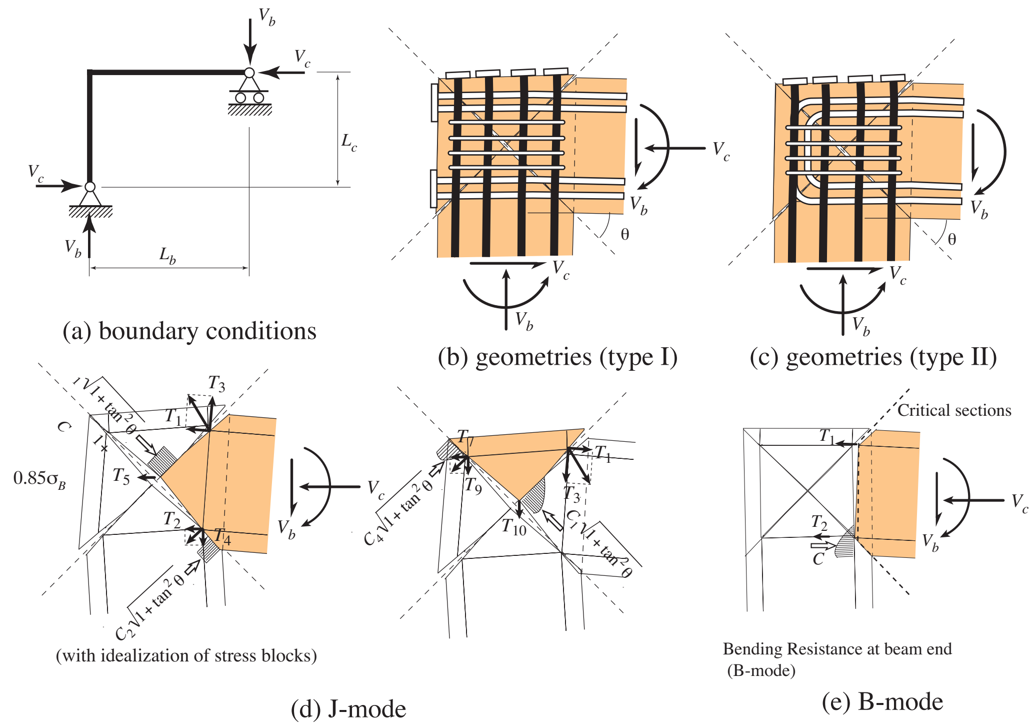


Figure 5: Model for knee joints loaded to closing direction [12]

Assumptions of the Theory

The followings are some general assumptions of the analysis based on the theory:

- Equilibrium should be satisfied. Equilibrium equations are established for each segment for horizontal and vertical directional forces as well as moment for J-mode. Equilibrium equations are established for axial force and moment for B-mode. The equilibrium conditions are used to obtain the relations between the external forces and the stresses resultants at the critical sections. Compatibility condition may not be necessarily satisfied. Number of independent equations for models for Figs. 3, 4 and 5 are five, nine, and nine respectively.

- Stress resultants does not exceed their material strength. The resultants in longitudinal bar do not exceeds the yield strength of the reinforcement. Reinforcing bars resist only to the axial force, whereas the dowel action is neglected for simplicity. The bond stress also does not exceed its bond capacity. Bond stress is assumed to distributed uniformly between two critical section.

- Diagonal cracks exist in two direction due to cyclic loading in beam-column joint. Concrete transfers no tensile force across the crack. Resultant force in compression are transferred by compressive reinforcement and/or across concrete cracks. In J-mode, each direction of principle stress on the critical section is assumed parallel to the diagonal direction of the joint panel. In B-mode, effect of shear transfer by concrete is neglected. On the critical sections, distribution of the concrete stress is assumed as a rectangular stress block, where the concrete stress is σ_c. The value of σ_c is chosen by considering the average stress of concrete stress block at flexural ultimate condition. The depth of the stress block should be larger than zero but less than the total depth of the section.

APPLICATION EXAMPLE OF THE THEORY: THE SIMPLEST CASE

To demonstrate, how the theory works for the prediction of the strength and type of failure, an example of application to the simplest interior beam-column joint is examined.

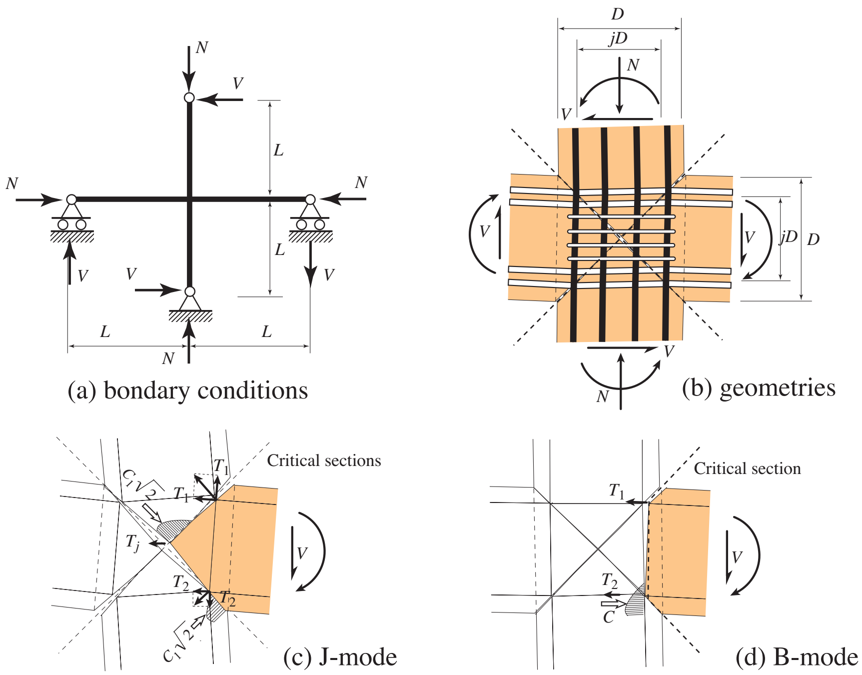


Figure 6: Symmetric interior beam-column joint

Geometry and Notations

Let us consider a symmetric interior beam-column joint in Fig. 6. To reduce the number of independent equilibrium equations, the geometry of the joint is assumed to be symmetric in horizontal and vertical direction. The depth of column and beam is assumed to be same. Thus the shape of the joint panel is square of D and $\theta = 45$ degree. The thickness of beam and thickness of column as well as joint were assumed be b_b b_c and b_c respectively. The distance between the center of the joint to the contra flexural points in the beam or column is L. External shear loads of V are applied at the contra flexural points. Beams and columns are under constant axial force of N. It is also assumed that the flexural capacity of columns is sufficiently larger that that of beams.

Notations for Internal Forces

Critical sections of B-mode and J-mode are shown in Figs 6(c) and 6(d). The stress resultants are given by the following notations. The notations T_1, and T_2 represent the resultant tensile forces in longitudinal bars, while C_1 and C_2 represent the resultant forces in compression acting on the concrete boundaries. The values of C_1 and C_2 is the horizontal component of compressive resultant in concrete. The forces T_1 and T_2 are common variables for both B-mode and J-mode. All of the compressive stress in concrete on the critical section is assumed to be normal to the critical section and distribution is assumed to be stress block with compressive stress of σ_c. The notation T_j represents the resultant forces in joint shear reinforcements distributed within beam-column joint, which confine the joint core.The distance of tensile and compressive longitudinal bars is assumed to be jD.

Equilibrium Equations for J-mode

The equilibrium condition for the segment is given by Eq. (1) and Eq. (2) for horizontal and vertical direction respectively,

$$-T_1 - T_2 - T_j + C_1 + C_2 - N = 0 \tag{1}$$

$$T_1 - T_2 - C_1 + C_2 - V = 0 \tag{2}$$

The Eq. (3) represents the equilibrium of moments around the center of beam-column joint.

$$jD(T_1 - T_2) - \frac{C_1^2}{b_c\sigma_c} + C_2\left(D - \frac{C_2}{b_c\sigma_c}\right) - LV = 0 \tag{3}$$

As the symmetry for the geometry of beam-column joint is assumed, the four parts divided by diagonal lines in beam-column joint are subjected to same loading condition. So the number of independent equilibrium equations for J-mode in this subassembly system is three.

Equilibrium equations for B-mode

The equilibrium condition for the segment about B-mode are given by Eq. (5) for horizontal direction and moment around the center of the beam-column joint by Eq. (6).

$$-T_1 - T_2 + C - N = 0 \tag{4}$$

$$\frac{1}{2}jD(T_1 - T_2) + \frac{C}{2}\left(D - \frac{C}{b_c\sigma_c}\right) + LV = 0 \tag{5}$$

Failure Criteria of Concrete

The concrete stress at the concrete stress block is assumed as 85% of concrete compressive strength considering the typical value used for flexural analysis of ultimate strength of section of reinforced concrete line elements.

$$\sigma_c = 0.85\sigma_B \tag{6}$$

Failure Criteria of Reinforcing Steel

Tensile force is transmitted by the longitudinal bars. Thus it is assumed that they do not exceed the yielding force. The restrictive conditions are given by Eqs. (7) and (8).

$$T_1 \leq \Sigma a_t f_y \tag{7}$$

$$T_2 \leq \Sigma a_t f_y \tag{8}$$

where, Σa_t : total cross section area of tensile reinforcements, and f_y : tensile yield point of longitudinal reinforcement. Joint shear reinforcing bars are assumed that they are concentrated at the mid-height of the joint and always equal to the yielding force and given by Eq. (9),

$$T_j = p_w b_c (jD) f_{sy} \tag{9}$$

where, p_w : joint shear reinforcement ratio, f_{sy} : tensile yield point of joint reinforcement.

Failure Criteria of Bond strength

In the past tests of interior and exterior beam-column joint, it was reported that joint shear failure initiated after the anchorage force saturates in beam longitudinal bar passing through the beam-column joint for interior joint [7]. Thus, the following restrictive conditions are required to evaluate the joint shear capacity as follows. The total bond force B is expressed as the difference of stress resultants T_1 and T_2. The bond force B is demand on bond strength necessary to be transferred to concrete by local bond within joint. If the bond capacity is assumed to B_u, then the value of $T_1 - T_2$ could not exceed the bond capacity B_u. So the restrictive conditions from bond strength is givens as shown in Eq. (10)

$$B = T_1 - T_2 \le B_u \tag{10}$$

where, B_u: anchorage capacity of beam bars, assumed to be estimated with Eq. (11) based on the test within beam-column joint. For the value of the k of 1.8 was suggested for modeling of bond capacity of non yielding tensile bar passing beam-column joint by Lowes [13].

$$B_u = k\sqrt{\sigma_B}\Sigma\varphi D \qquad \text{(in MPa)} \tag{11}$$

where, $k\sqrt{\sigma_B}$: averaged bond strength in beam-column joint in MPa, and σ_B: concrete compressive strength in MPa, $\Sigma\varphi$: total perimeter length of longitudinal bars in mm, and D: beam (column) depth in mm. It is also assumed that the bond strength is kept constant after bond strength is reached in spite of bond-slip increase, because this type of bond resistance is ductile within beam-column joints due to good confinement.

Solution for J-mode Equilibrium

The unknown variable C_1 and C_2 are solved from Eq. (1) and Eq. (2). The results are given by Eq. (12) and (13) respectively. To eliminate the unknown variable C_1 and C_2, they are substituted to Eq. (3) then it yields Eq. (14).

$$C_1 = \frac{1}{2}(-T_j - N - V + 2T_2) \tag{12}$$

$$C_2 = \frac{1}{2}(T_j + N + V + 2T_2) \tag{13}$$

$$jD(T_1 - T_2) - \frac{(-N - T_j - V + 2T_2)^2}{4b\sigma_c} + (N + V + 2T_2)\left(D - \frac{N + T_j + V + 2T_2}{2b\sigma_c}\right) - LV = 0 \tag{14}$$

Equation (14) is an quadratic equation for unknown variable V, acounting J-mode equilibrium. So by solving the Eq. (14), V is obtained as a function of T_1 and T_2. The maximum value of $V_{\text{J-mode}}$ is obtained as optimized value under the restrictive condition of T_1 and T_2 given by Eqs. (7), (8) and (10). Then values of T_1 and T_2 are also determined as the value corresponding to the maximum value of V.

Solution of B-mode Equilibrium

By combining the Eqs. (4) and (5), Eq. (15) is obtained. Then $V_{\text{B-mode}}$ is calculated by substituting the values of T_1 and T_2 obtained above into Eq. (15).

$$V = \frac{1}{L}\left[\frac{1}{2}jD(T_1 - T_2) + \frac{1}{2}(T_1 + T_2 + N)\left(D - \frac{T_1 + T_2 + N}{b_b\sigma_c}\right)\right] \tag{15}$$

Strength of Beam-column Joint

The predicted strength V_{uj} of beam-column joint is obtained in terms of column shear as the smaller strength of J-mode and B-mode, as follows,

$$V_{uj} = \min\{V_{\text{J-mode}} \ , \ V_{\text{B-mode}}\} \tag{16}$$

Joint Shear

The expression of capacity of beam-column joint by joint shear stress is sometimes preferred. In that case, joint shear stress τ_j at the strength of beam-column joint is calculated by Eq. (17) from column shear V, assuming the distance of tensile and compressive resultants is jD.

$$\tau_j = \frac{2}{D(b_c + b_b)} \cdot \left[\frac{2L - jD}{j} - 1\right] V \tag{17}$$

Type of failure

Based on the calculated hierarchy of strengths of J-mode and B-mode, as well as the stress level T_1 in tensile reinforcement at the critical section in the beam, one of the four types of failure fro **J**, **BJ**, **B** or **CB** are determined by applying rules given by the Table 1.

Table 1: Prediction of Failure Types [8]

Failure type	Condition	Explanation
J	$V_{\text{J-mode}} < V_{\text{B-mode}}$ and $T_1 < \Sigma a_t f_y$	Joint shear failure without beam yields
BJ	$V_{\text{J-mode}} < V_{\text{B-mode}}$ and $T_1 = \Sigma a_t f_y$	Joint shear failure after beam yields
B	$V_{\text{J-mode}} > V_{\text{B-mode}}$ and $T_1 = \Sigma a_t f_y$	Beam yielding
CB	$V_{\text{J-mode}} > V_{\text{B-mode}}$ and $T_1 < \Sigma a_t f_y$	Compressive flexural failure at beam end

PARAMETRIC STUDY OF JOINT SHEAR STRENGTH

As an example of the parametric study, results of J-mode strength for interior and exterior beam-column joints are calculated and discussed in this section. The loading conditions, boundary conditions and the notations are shown in Figs. 3 and 4. Table 2 lists the control parameters for numerical analysis common for both interior and exterior joints. Longitudinal reinforcements in beams and columns are assumed infinitely strong to calculate the strength of Failure type J in table 1. For exterior beam-column joint, anchorage of beam bar is assumed to locate inside of longitudinal bars in column. As a result, the magnitude of tensile resultants T_2 in Fig. 4(d) is assumed to always zero and the development length of beam bar in joint is assumed to 80% of column depth D_c. For exterior beam-column joint, the anchorage strength of beam bars is assume to be infinite while, the bond stress along column longitudinal bars on the side of beam (T_3 – T_4) is assumed to be reached to its bond strength B_u. The bond strength is assumed to be given using the Eq. (18).

$$B = T_3 - T_4 \le B_u = k\sqrt{\sigma_B}\Sigma\varphi D_c \quad \text{where,} \quad k = 1.8 \tag{18}$$

The equilibrium equations and solutions are not shown here but described in detail in the reference [8] for interior joint and the reference [11] for exterior joint. To obtain the numerical solutions by solving the simultaneous equation, symbolic mathematical programing software Maple was used.

Figure 7 shows the J-mode strength in terms of joint shear stress, normalized with concrete compressive strength. Horizontal axis is the stress resultant T_1 divided by stress of stress block and column width b_c. Maximum joint shear stresses are 0.3211 and 0.184 for the interior beam-column joint, and the exterior beam-column joint respectively. As shown in Fig. 8, The strength of the exterior beam-column joints is 53% of the strength of the interior beam-column joint with parameters shown in Table 2.

Table 2: Control parameters of a beam-column joint subassembledge

Parameters	
$L_b = 1500$ mm	$j_c = 0.75$
$L_c = 1000$ mm	$N_c = 100$ kN
$b_b = 250$ mm	$N_b = 0$
$b_c = 300$ mm	$\sigma_B = 30$ MPa
$D_b = 300$ mm	$p_w = 0.3\%$
$D_c = 300$ mm	$f_{sy} = 300$ MPa
$j_b = 0.75$	$\sigma_c = 85\%$ of σ_B
beam (column) bars : 4-D13 (First layer)	
development length of beam bar = 0.8 D_c	

The calculated joint shear for the cases of $p_w = 0.6\%$ and $p_w = 0.9\%$ are also shown to examine the effects of the amount of joint shear reinforcement. Amount of joint shear reinforcement has no effect on the joint shear strength for interior joints. This fact agrees with currently admitted consensus on the joint shear strength of beam-column joints based on tests described in design codes [4, 5]. On the contrary, the calculated joint strength for exterior beam-column joints of type II increase with increasing of joint shear reinforcement ratio p_w. This prediction also agrees with currently admitted consensus.

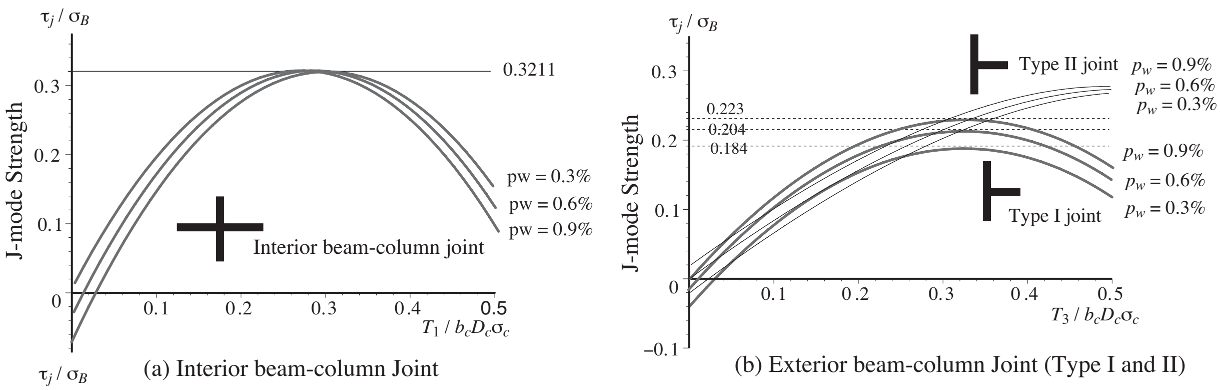


Figure 7: Relation of stress resultants in tensile longitudinal reinforcement in beam at column fact to pseudo joint shear strength [11]

By comparison of joint strength of interior and exterior beam-column joints, shape factor adopted in current code seems to be larger for exterior beam-column joint than both ACI [4] and AIJ [5]. It means that interior beam-column joint has larger safety margin than exterior beam-column joint for J-mode strength. However, the assumptions in this paper neglect special cases, such as the premature yielding of column longitudinal reinforcement, which causes joint shear failure is not considered here. In addition to that, some test results of exterior beam column joints showed joint shear failure initiated beam bar yielding, while story shear is much lower than calculated by flexural theory [14]. Hence the prediction by this thory may give higher strength than tests in some case. The premature yielding of bars in longitudinal reinforcement in columns and subsequent joint shear failure may also happen, if the bending capacity of column is not sufficient in particular in the case the columns are subjected to varying axial force including tension.

Effect of bond capacity on joint shear strength

As bond capacity is considered as one of key factors for this model which determines the strength and failure modes of beam-column joints, The effects of the bond capacity is investigated. Figure 8(a) shows the calculated value of J-mode strength with parameter of *k* defined in Eq. (11) and (18). Larger value of *k* generally gives larger joint shear strength. But the joint strength is not proportional to the bond strength nor have significant effects. If bond capacity increase twice, the increase of joint shear strength remains approximately 10%. This theory predicts that the bond capacity is not so sensitive to the J-mode joint shear strength. Nonetheless this result is important, because this model explains the reason of enhancement in strength in 3-D beam-column joint with transverse beams observed in tests. Transverse beams covering beam-column joint is effective to increase the bond strength, as a result, joint shear strength increases.

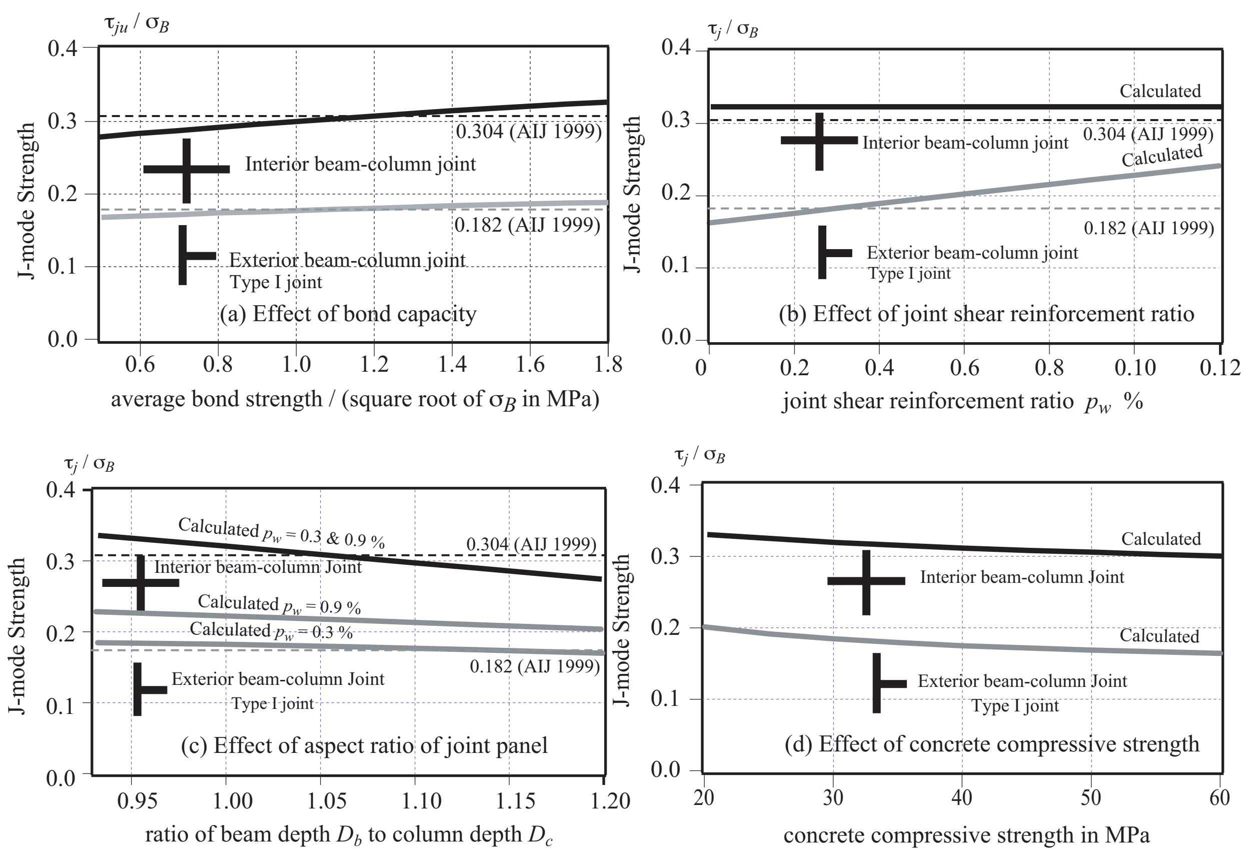


Figure 8: Effect of selected parameters [11]

Figures 8(a) 8(b) and 8(c) compare the calculation and the average strength reported in the commentary of AIJ Guidelines [5] derived from Japanese database of the tests of beam-column joint without transverse beams. They are given by the equations,

$$\tau_{ju} = 1.56 \times \sigma_B^{0.712} \quad \text{for interior beam-column joint} \tag{19}$$

$$\tau_{ju} = 1.13 \times \sigma_B^{0.718} \quad \text{for exterior beam-column joint} \tag{20}$$

where, τ_{ju} : joint shear strength in MPa and σ_B : concrete compressive strength in MPa.

So as to compare the interior and exterior joint shear strength based on same joint area A_j, the value of Eq.(20) are factored 80% in Figs.10 and 11, because Eq. (20) is based on the effective depth equal to development length from column face to the anchor end in AIJ Guidelines. For both interior and exterior joint, calculated values agree well with the average observed joint strength.

Effect of Joint Shear Reinforcement Ratio

Figure 8(b) shows the relation of the joint shear reinforcement ratio and joint shear strength. It is worth noted again that the joint shear reinforcement ratio has significant effects for enhancing the joint shear strength only for exterior beam-column joints, while it has no effects on the shear strength of interior beam-column joints.

Effect of Aspect Ratio

Figure 8(c) shows the other important factor seemingly affecting the joint shear strength much. It is the aspect ratio of beam-column joint panel. The aspect ratio is defined as the ratio of beam depth to column depth. For interior and exterior beam-column joints, larger aspect ratio gives lower joint shear strength. It is interesting prediction and need to be investigated experimentally because prediction of the effect is very critical compared to the other parameters. This factor need to be taken into account as one of most important factors in future code revision.

Effect of Concrete Compressive Strength

Figure 8(d) shows the relation of concrete strength and joint shear strength. It seems that joint shear strength is not proportional to concrete compressive strength. It is because the joint shear strength is partly governed by the bond capacity and it is assumed to be proportional to square root of concrete compressive strength in this study. In reality, bond capacity may changes due to various factors such as axial force level of column due to confining effect or transverse beam covering joint. Bond strength is also affected by the thickness of cover concrete and location of bar in beam or column section as well as diameter of bars and number of bars, yielding of longitudinal steel and cyclic loading. They should be considered if necessary. The effect of bond deterioration due to loading history and yielding of reinforcing bars may also cause strength decay for joint shear failure after beam yielding.

CONCLUDING REMARKS

Quadruple flexural resistance and a new theory of reinforced concrete monolithic beam-column joint is briefly presented. The theory is based on a very simple concept. But it has an ability to predict both strength and failure type, reflecting almost all critical parameters which are considered to be have effects on the behavior of beam-column joints. The model contains no empirical factors accounting the difference between interior, exterior or knee joint like a shape factors adopted in current design codes. The principles, assumptions, mathematical formulation and numerical demonstration of the theory are described. Some conluding remarks obtained from the numerical calculation are as follows.

1. It is demonstrated that common concept of quadruple flexural resistance is applicable for interior, exterior and knee joint despite of their difference in number of framing member in rational way. The calculated value of joint shear strength agree well with the average observed joint strength derived from Japanese tests database of beam-column joint.

2. It is predicted by the theory that joint shear reinforcement ratio has significant effects for enhancing the joint shear strength of exterior beam-column joints, while it has no effects on the shear strength

of interior beam-column joints. This prediction endorses the state-of-the-art on the beam-column joint.

3. By comparison of joint shear strength of interior and exterior beam-column joint, shape factor adopted in current code seems to be larger for exterior beam-column joint. It means that interior beam-column joint is larger safety margin than exterior beam-column joint.

4. Bond capacity increase enhanses the joint shear strength increase and the bond capacity is key parameter in the new models. Nevertheless the joint strength is not proportional to the bond strength nor have significant effects.

5. Beam-column joints with larger aspect ratio have lower joint shear strength. The aspect ratio seems to have critical impact on the joint shear strength and further experimental investigation is recommended.

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