Shear Deterioration of Reinforced Concrete Beam-Column Joints

J. W. Park, D. H. Kim, S. G. Yoon & J. Y. Lee
Dept. of Architectural Engineering, Sungkyunkwan University, Korea

SUMMARY
The reinforced concrete (RC) beam-column joint region is subjected to horizontal and vertical shear force, whose magnitude is many times higher than in column and adjacent beams, and consequently much larger bond and shear stresses may cause brittle bond or shear failure in the joint region. To prevent brittle failure, the seismic design for RC beam-column joints needs to consider post-elastic behavior. The critical deterioration of potential shear strength in the joint area should not occur until the ductile capacity of adjacent beams has reached the design demand. In this study, a method is provided to predict the deformability of reinforced concrete beam-column joints failing in shear, after plastic hinges develop at both ends of the adjacent beams. In order to verify the deformability estimated by the proposed method, experimental data of eight joint specimens were analyzed.

Keywords: RC beam-column joint, strain penetration, shear deterioration, seismic design, ductility

1. INTRODUCTION

Beam-column joints are the critical regions influencing stability of buildings for moment resisting reinforced concrete (RC) frames, when subjected to both lateral and vertical loads. Because a beam-column joint refers to the portion of a column within the intersection of connected beams, from the basic design philosophy termed “strong-column and weak-beam”, the joint needs to remain in the elastic range. When only the flexural strength of a well-detailed longitudinal beam limits its response, the whole structure commonly performs in ductile behavior, with elastic state of beam-column connections. The beam-fail mechanism is usually considered to be the most desirable for maintaining large deformability, without severe deterioration of resistance capacity. For this reason, in general earthquake resistant design philosophy, ductile frame buildings allow the beam to develop a plastic hinge to protect from the occurrence of critical damage of columns, and joint regions. Also, degradation of severe shear capacity is avoided, until the ductile capacity of adjacent beams reaches the demanded design capacity. However, when a severe lateral load like a seismic attack affects the buildings, joints could fail. As a consequence of seismic moment of opposite signs in beams and columns, the joint region is subjected to horizontal and vertical shear force whose magnitude is much higher than in adjacent columns and beams. Consequently large bond and shear stresses are required to sustain these forces, whose gradients may exceed the limits of capacity and stability for lateral deformation. After the formation of plastic hinges in adjacent beams, the shear capacity of a joint could be reduced by the phenomenon of yield strain penetration. Therefore, understanding joint shear behavior is important in figuring out the performance of beam-column connections and entire frames.

Since the 1970s, much research work has been conducted on improving understanding of RC joint shear behaviour, using analytical and experimental studies. In particular, coordinated efforts in relevant research in different countries, in the United States, New Zealand and Japan, have proven very successful. However, despite such advanced achievement, research results have been applied in different ways in each country. The United States code, the ACI Recommendation (ACI 352R-02, 2003), provides a design philosophy of two types, according to the magnitude of earthquake. In major earthquake regions, the design uses a special factor for steel components, to consider post-elastic performance. The Japanese code, the AIJ (Architectural Institute of Japan, 1999) guideline, is theoretically similar to the ACI Recommendations. The prediction of joint shear strength for both countries is based on the concrete arch mechanism, while in the New Zealand Standard (NZS, 1982) code, joint shear strength is predicted by both arch and truss mechanisms. While the three countries support the idea that the major factors to evaluate shear capacity are the compressive strength of concrete and the geometric character of the joint area, they disagree over other factors.

Individual researchers have discussed several factors and assessments in the prediction of joint shear strength for various types of RC beam-column connections. Fujii and Morita (1991), and Durrani and Wight (1985) performed experimental study to figure out the effect of certain variables. Attaalla et al. (2003) analyzed a great deal of technical literature, and proposed a prediction model for joint shear strength. Recently, Kim et al. (2007, 2009) suggested a model using a statistical approach. However the above research work focused on the maximum shear strength, and its capacity, so it has difficulty in determining the ductility or deformability of a joint. To evaluate ductility or deformability, we need to understand a decreasing gradient of potential joint shear strength as joint drift increases. Lee et al. (2003) tried to determine potential shear strength by considering the flexural deformation of adjacent beams; however, they did not consider joint panel deformation and the truss mechanism.

In this study, we calculate the deformability of reinforced concrete beam-column joints failing in shear, after plastic hinges develop at the end of the adjacent beam. This model considers not only post-elastic performance of adjacent beams, but also panel shear deformation. A new mechanism is constructed using the arch and truss mechanism, together with certain assumptions. In order to verify the deformability estimated by the proposed method, eight sets of experimental data from several researchers were inserted into the calculation procedure.

2. DUCTILITY PREDICTION MODEL FOR BEAM-COLUMN JOINT

Figure 1 illustrates common longitudinal strain distribution, since adjacent beams yield. Despite the ultimate strength of the joint being designed to be much higher than the flexural strength of the beam, the joint region could perform as post-elastic behavior as the penetrated strain increases. As the penetrated strain within the joint increases, so the principal tensile strain increases, and the effective compressive strength of the concrete decreases. This strain penetration phenomenon weakens the potential shear strength of the joint area.

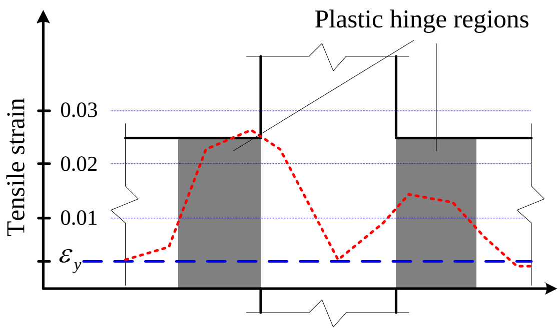


Figure 1. Longitudinal strain distribution

The research reported in this paper proposed a method to calculate the ductility of RC joint failing in shear after flexural yielding of adjacent beams. Figure 2 shows the calculation procedure for ductility of beam-column joint.

STEP 1: In the first step, the properties and geometric features of the specimens are input into the algorithm. The starting point is the yielding of the beam reinforcement. Figure 3 demonstrates the total physical joint deformation, which consists of each assembly's deformation when a beam-column joint is subjected to lateral loading. The column rotation angle is calculated by elastic theory; the joint panel rotation angle and beam rotation angle are obtained in the following steps, from the member rotation angle, R_m, which is defined as Δ/H.

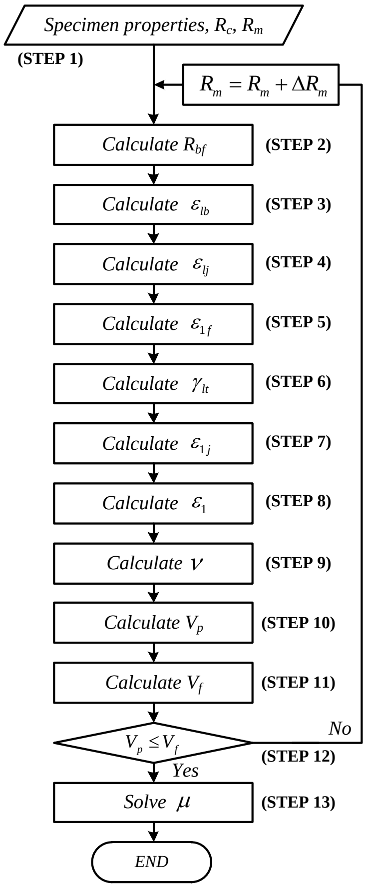


Figure 2. The calculation procedure

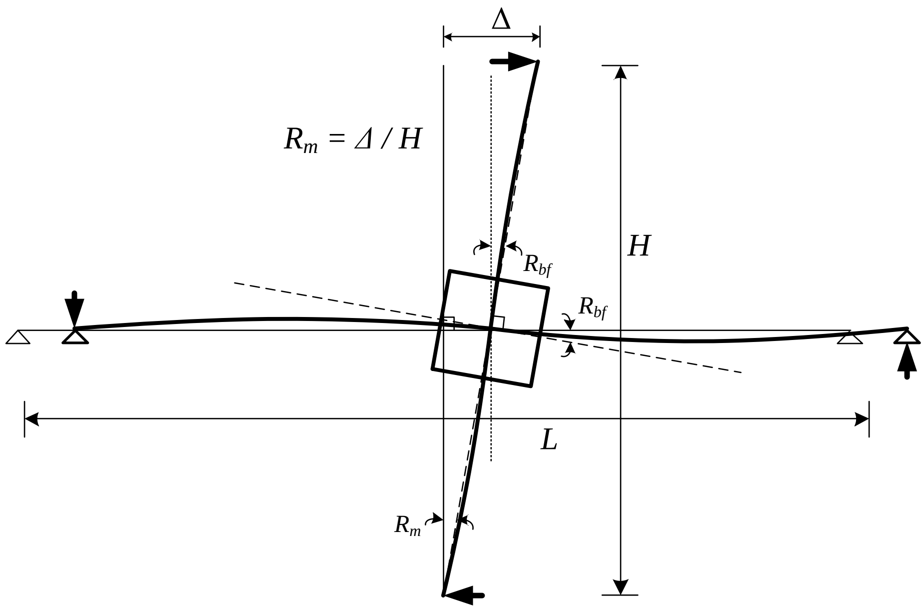


Figure 3. Joint deformation

STEP 2: The beam rotation angle, R_{bf}, is regarded as the member rotation angle, R_m, as shown in Figure 3.

STEP 3: The longitudinal strain of a plastic hinge subjected to cyclic loading is determined by using Equation 1, as proposed by Lee and Watanabe (2003):

$$\varepsilon_{lb} = \varepsilon_y + \left(\frac{R_{mpp} + R_{mnp}}{2l_p} \right) z \tag{1}$$

where ε_y is the yield strain of beam reinforcements, R_{mpp} and R_{mnp} are the rotation angles for the plastic performance range, z is the distance between tensile and compressive reinforcements, and l_p is the length of the plastic hinge region.

$$l_p = 0.5 \left(\frac{M}{Vh} \right) d \quad (0.75\mathrm{d} \le l_p \le \mathrm{d}) \tag{2}$$

where $M/(Vh)$ is shear span-to-depth ratio, and d is the effective depth of beam.

STEP 4: The horizontal joint strain, ε_{lj}, is considered as

$$\varepsilon_{lj} = K\varepsilon_l \tag{3}$$

where K is the correlation factor between ε_{lb} and ε_{lj}. K represents the penetration effect of longitudinal strain for the beam reinforcement at the plastic hinge region.

STEP 5: ε_{lj} makes joint panel deformation; so shear strength of joint area is decreased. The principal tensile strain of joint, ε_{1f}, is calculated by strain compatibility equation shown as below.

$$\varepsilon_{lj} = \varepsilon_{1f} \sin^2 \alpha + \varepsilon_{2f} \cos^2 \alpha \tag{4a}$$

the principal compressive strain, ε_{2f}, is much smaller than the principal tensile strain caused by flexural deformation of beam, ε_{1f}, and this principal tensile strain can be simplified as below.

$$\varepsilon_{1f} = \frac{\varepsilon_{lj}}{\sin^2 \alpha} \tag{4b}$$

STEP 6: Shear strain in joint panel is also calculated by strain compatibility equation.

$$\frac{\gamma_{lt}}{2} = (-\varepsilon_{2f} + \varepsilon_{1f}) \sin\alpha \cos\alpha \tag{5}$$

STEP 7: The extra tensile principal strain caused by shear deformation is determined from Attaalla's (2003) research. Figure 4 demonstrates the shear deformation of the joint.

$$\varepsilon_{lj}' = \frac{\gamma_{lt} \sin\alpha \cos\alpha}{h_c} (h - 2c_j) \tag{6}$$

where ε_{lj}' is horizontal strain generated by shear strain. c_j is the depth of the compression zone in the joint.

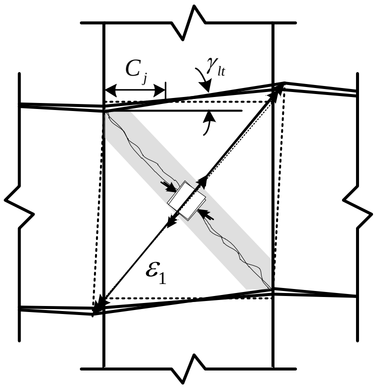


Figure 4. Increasing shear deformation

STEP 8: Therefore, the principal tensile strain is

$$\varepsilon_1 = \varepsilon_{1f} + \varepsilon_{j1} \tag{7}$$

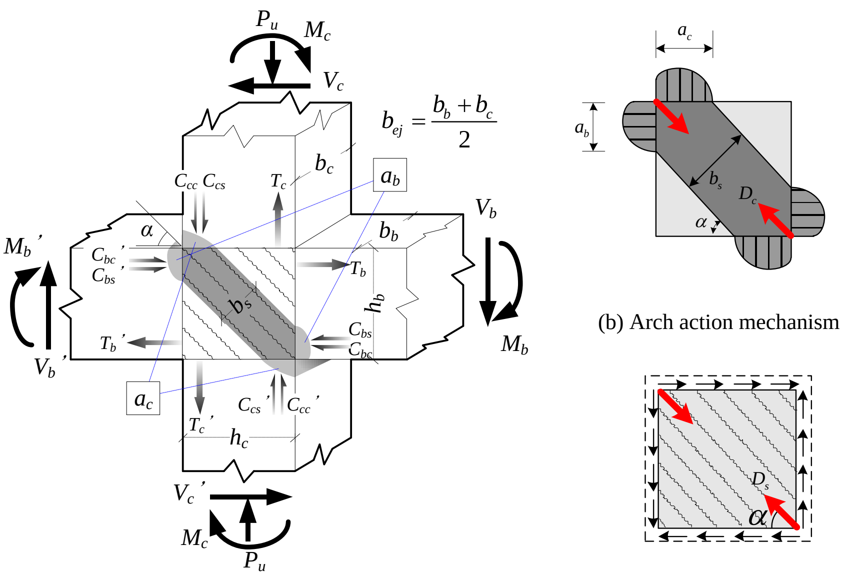
STEP 9: The principal tensile strain weakens the compressive strength of the strut. The strength reduction factor of the joint, v, is calculated using the equation suggested by Belarbi and Hsu (1995):

$$v = \frac{5.8}{\sqrt{f_c'(1+400\varepsilon_1)}} \le \frac{0.9}{\sqrt{1+400\varepsilon_1}} \tag{8}$$

STEP 10: The potential shear strength is determined based on both arch action and truss action. Figure 5 shows the forces acting on the joint. Then, each component, the arch and truss action mechanism, can be expressed, respectively, as the following:

$$V_{ch} = v_c f_c'(b_s \cdot b_{ej})\cos\alpha \tag{9}$$

$$V_{sh} = v_s f_c'(h_c \cdot b_{ej})\sin\alpha\cos\alpha \tag{10}$$



(a) Forces acting on the joint

(b) Arch action mechanism

(c)Truss action mechanism

Figure 5. Two components to predict potential shear strength

where v_c, v_s are the strength reduction factors applied to concrete compressive strength, and α is the inclination of the diagonal plane with respect to the horizontal axis. The shear strength for the beam-column joint is the summation of the above two components:

$$V_p = V_{ch} + V_{sh} \tag{11}$$

STEPS 11~13: Ductile capacity of the joint is determined at the point where the potential shear strength of the joint (V_p) crosses the flexural yield strength of the beam (V_f). If V_p corresponding to that given by STEP 12 is greater than the shear strength corresponding to the development of the plastic hinge, a new value of R_m is applied, and the steps are repeated until V_p equals V_f.

3. EXPERIMENTAL DATA

3.1. Specifications of specimens

In order to verify the applicability of the calculation procedure in predicting the deformability of BJ-failure joints, the proposed procedure was compared with experimental results of reinforced concrete joints reported in the technical literature. The subjects are Lee et al's (1993) 3 specimens, Kaku et al's 2 specimens (1993), Hayashi et al's 1 specimen (1993) and Yoshino et al's 2 specimens (1992). All specimens failed in shear at the joint area after beam flexural yield. Table 1 shows the properties and variables of specimens. Shear strength is determined by ACI 352R-02 as the following equation:

$$V_j = 0.083\gamma\sqrt{f_c'}b_j h_c \tag{12}$$

where V_j is the shear strength of joint, γ is a value that depends on the connections classification and seismic magnitude, b_j is the effective joint width, and h_c is the depth of the column in the direction of joint shear being considered.

Table 1. The material properties of specimens

Specimens		Beam		column		f_c' (MPa)	V_{j1} (kN)	V_{j2} (kN)	V_{jby} (kN)	V_{j1}/V_{jby}	V_{j2}/V_{jby}
		Reinforcing bar		Reinforcing bar							
		f_{by} (MPa)	N-n_b	f_{cy} (MPa)	N-n_c						
Lee	BJ1	509.9	6-D16	514.4	12-D29	40.0	1194.2	895.7	802	1.06	0.80
	BJ2	509.9	5-D16	514.4	12-D29	40.0	1194.2	895.7	614	1.25	0.94
	BJ3	509.9	4-D16	514.4	12-D29	40.0	1194.2	895.7	500	1.54	1.16
Kaku	J31A	372.6	4-D25	371.8	14-D19	57.9	1209.0	906.8	1432	0.84	0.68
	J32B	372.6	4-D25	371.8	14-D19	57.9	1209.0	906.8	1432	0.84	0.68
Hayashi	No.47	382.2	6-D19	644.8	12-HD19	49.0	1626	1220	1253	1.30	1.04
Yoshino	No.1	382.4	4-D13	378.6	8-D16	28.6	477	358	397	1.20	0.96
	No.3	382.4	3-D16	378.6	8-D16	28.6	477	358	453	1.05	0.84

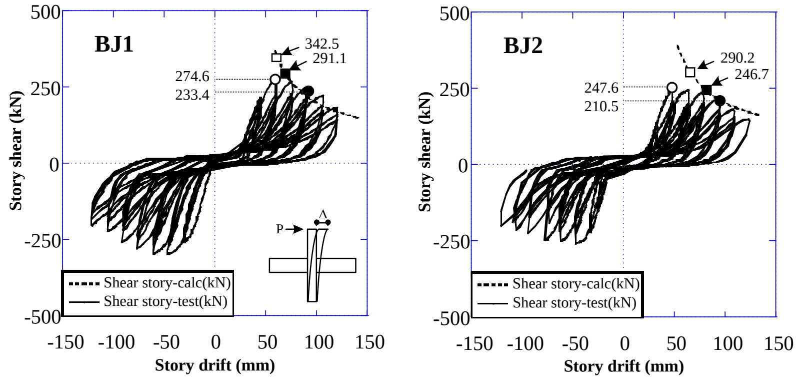
f_{by}, f_{cy}: The yield strength of beam and column reinforcement, N : The number of reinforcement, n_b, n_c: Grade description of beam and column reinforcement, f_c': The compressive strength of concrete, V_{j1},V_{j2} : The joint shear strength proposed by ACI 352R-02, V_{jby} : The joint shear strength when the beam bar yields

V_{jby} is the joint shear strength when the beam yields by flexure. V_{jby} is calculated as:

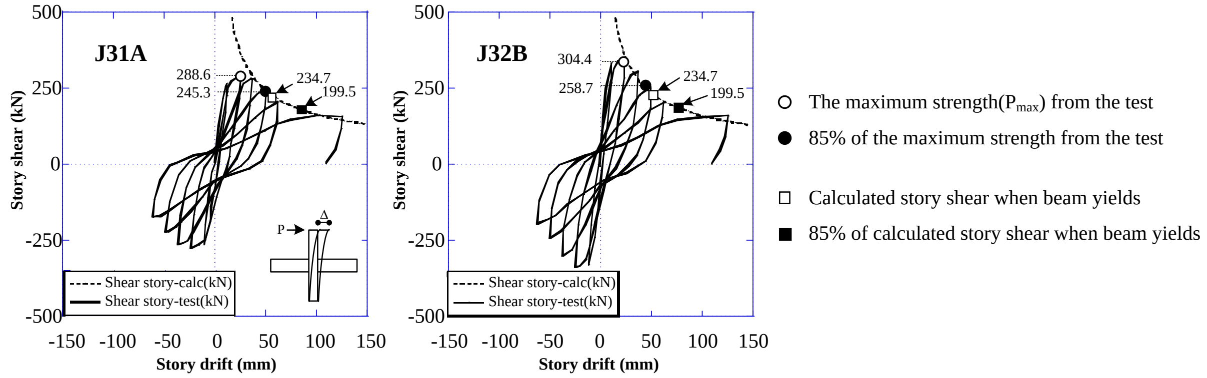
$$V_{jby} = \left(\frac{H}{z_b} - 1 \right) V_{by} \frac{L}{H} - \frac{h_c}{z_b} V_{by} \tag{13}$$

where l_c is the column height, h_c is the joint height, z is the distance between centroid of upper and lower beam bars and V_{by} is the beam shear strength when beam bar yields.

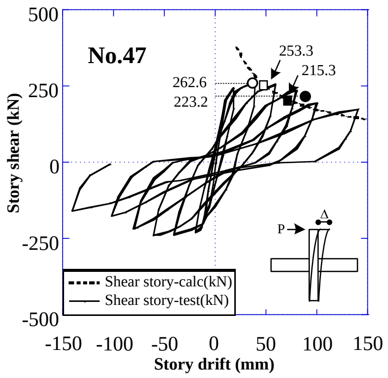
3.2 Ductile capacity of subjected specimens



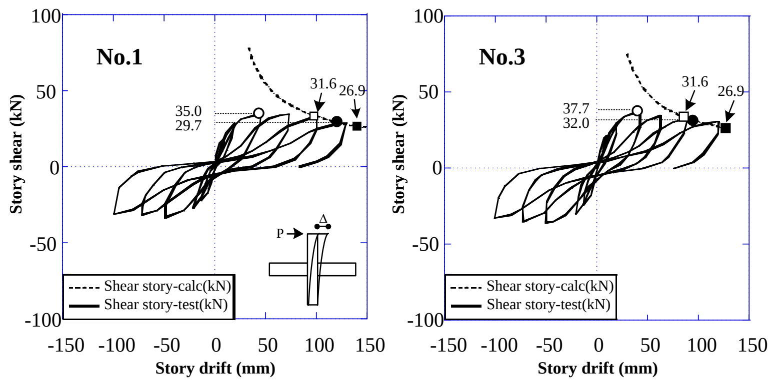
(a) Calculated curves compared with Lee ' s test



(b) Calculated curves compared with Kaku ' s test



(c) Calculated curves compared with Hayashi ' s test



(d) Calculated curves compared with Yoshino ' s test

Figure 6. Tested and calculated result for story drift and story shear relation

Figure 6 demonstrates the hysteresis curves of all specimens, and decreasing potential shear strength curves obtained by calculation procedure, as shown in Figure 2. In the figure, the solid and dotted line indicate the test and calculated result respectively. Also, the empty circle and square depict the tested maximum strength, P_{max}, and the calculated story shear strength when the beam yields, respectively. The solid circle and square values correspond to 85% of the empty circle and square values, respectively. The values of solid squares and circles are the reference points regarding the ultimate displacement. At the ultimate displacements, shown as the solid circles and squares in Figure 6, the test result of Lee et al. showed a 7~32% error range. Other test results by Kaku et al., Hayashi et al. and Yoshino et al., showed 28%, 28%, and 11~25% error ranges, respectively. The coincidence on the graph between the calculated potential shear strength and the envelop curve of tested specimen indicates the reasonableness of the proposed method. Figure 7 plots the ultimate displacement of test versus the ultimate displacement for calculation, and Figure 8 shows those ratios corresponding to V_{j1}/V_{jby} with statistical analysis. The mean of the specimens is 0.97, the standard deviation is 0.23, and the coefficient of variation is 0.24.

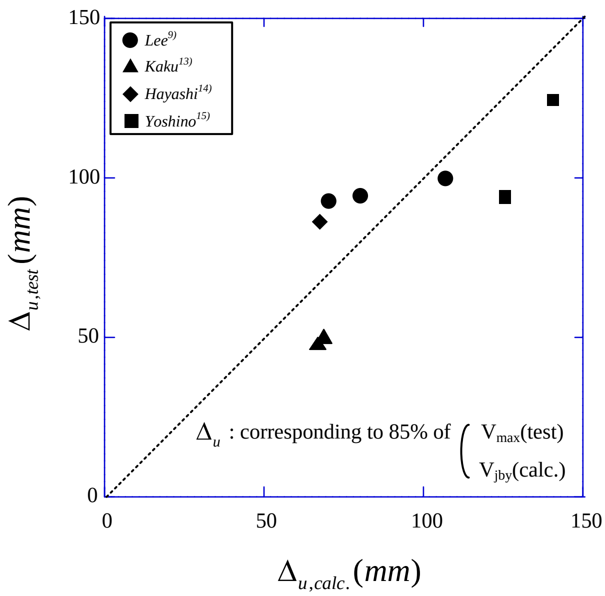


Figure 7. Calculated displacement vs. tested displacement

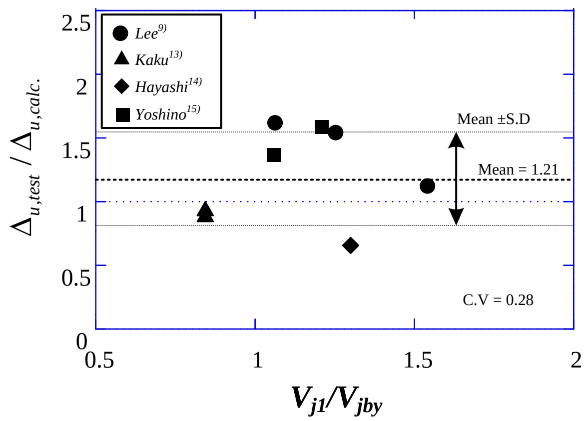


Figure 8. Comparison of deformability ratio

4. CONCLUSIONS

A method to predict the deformability of RC joints failing in shear after plastic hinge develops at the end of the adjacent beams is proposed with consideration to the degradation of compressed concrete due to strain penetration. It is another approach to evaluate RC beam-column joint and summarized as:

1) The proposed model calculated decreasing potential shear capacity of joint using joint deformation of each components and failure mechanism with some technical literatures as member rotation increase.
2) Comparison results between calculated and tested displacement showed that the deformability of joint could be predicted by analytical method with reasonable error range.

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