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STATE-OF-THE-ART SEISMIC EVALUATION OF REINFORCED CONCRETE BUILDINGS BASED ON BEAM-COLUMN JOINT CAPACITY.

PART 1: Theoretical Background

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ABSTRACT

Formulae to estimate shear capacity of beam-column joint of reinforced concrete structure shave been recently established and adopted in the AIJ design guidelines on ultimate strength concept. However, this is not yet reflected in the current design method. In this paper, simplified formulae to estimate shear stress τ cp of a column at shear failure of the connecting beam-column joint are derived as functions of several dimensions of the beam and of the column and shear capacity of the joint.
By this formulae, it can be understood that τ cp are generally important and the equations can be applied in structural planning of new buildings, evaluation of existing buildings and evaluation of damaged buildings.
In this report, the following items concerning the mean shear stress of a column, τ cp at shear failure of the beam-column joint are described.
a. Derivation of simplified equations to estimate τ cp based on dimensions of column, beams and joint panel strength.
b. Ranges of roughly estimated values of τ cp
c. Comparison of τ cp to exactly calculated values.
d. Utilization of the equations to structural design of new buildings, diagnosis of existing buildings and estimation of earthquake damages.

KEYWORDS

Shear capacity, shear cracking strength, plastic deformation capacity, beam-column joint

INTRODUCTION

The current seismic provisions in Japan was prepared mainly to prevent shear failure of beam and column, and to increase the ductility of building in view of the experience during the 1968 Tokachi-Oki earthquake where many school buildings were damaged due to members shear failure. Presently, a design method that can expect large plastic deformation capacity is being compiled to improve the ductility of reinforced concrete structures regarding the deformation ability of beam and column members even in a moderately high combined flexural-shear stress using high strength shear reinforcement . Usually seismic design of joint is not imposed in Japan regardless of the past several experimental results on seismic capacity beam-column joint subassemblages obtained, because examples of beam-column joint damage is very rare. However, until after the January 17, 1995 Hyogo-Ken earthquake, a modification has been recently followed.
Presently, buildings were constructed with strong column and beam in accordance with the existing provisions. However, during the 1995 Hyogo-ken earthquake, buildings with damaged joint were found only in buildings designed according to the current provisions, thus, joint investigation becomes necessary since this failure mode is characterized by a brittle failure.
As presented in this paper, there is a large difference between the seismic performance of a moment resisting frame of typical of beam and column dimensions failed in usual flexural stress and the seismic performance at joint shear failure. For this reason, a necessary understanding in design of connections must be paid a closed attention.
From this view point, since October 1995 design of beam-column joints is implemented also in the design of reinforced concrete buildings.

Also, based on the enforced law in December 1995 seismic evaluation and retrofit of existing buildings is on revision and investigation of beam-column joint is imposed. Moreover, the recent method of design of joint is based on ultimate strength design method at failure mechanism. For this reason, it can be applied in the similar type of ultimate design method, but it cannot be applied directly in the allowable working stress design. Also, the seismic evaluation of joint panel in existing buildings can be applied in the 3rd level screening, but the present design of joint cannot be applied in the first and second level screening which are based on column and beam capacity.

From the above-mentioned background, the seismic design method of reinforced concrete joint, its simplified investigation method and application method will be discussed.

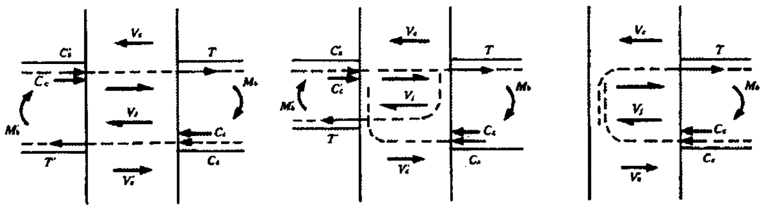


Fig.1. Beam-column joint shear force, V j

JOINT SHEAR FORCE, V j

Joint shear force of a moment resisting frame at beam flexural yielding due to lateral forces, is expressed as the difference between the tensile forces in beams and column shear force as shown in Fig.1.

$$V_j = T + C'_s + C'_c - V_c = T + T' - V_c \qquad (1)$$

Where V_c is the shear force in column correctly obtained at beam yield mechanism. The following items about V_j are explained.

a) As the beam tensile yield forces increase, i.e., the quantity of beam main bar and yield point strength increases, the value of V_j becomes larger because V_j is proportional to tensile forces.

b) In the case of constant moment M_y in beam, the shear force across the joint increases as the beam depth decreases.

c) The actual material strength of beam main bar which is nearly equal the yield point strength should be used in the calculation of T', T and M_b. In the calculation of total tensile force, T not only area of beam top bar but also slab reinforcement within effective flange width of about one meter is necessary.

From the above items, it is necessary to establish an upper limit value for reinforcement ratio within the beam effective flange and lower limit value for beam depth in order to prevent shear failure in joint.

JOINT STRENGTH FORCE, V ju

The shear strength of joint can be expressed as

$$V_{ju} = \kappa \cdot \phi \cdot F_j \cdot b_j \cdot D_j \qquad (2)$$

where:

$\kappa = 3$ for ┼ -type joint

$\kappa = 2$ for ├ - or ┬ -type joint

$\kappa = 1$ for ┌ -type joint

ϕ : joint restraint condition coefficient

$\phi = 1.1$ for joint with two transverse beams

$\phi = 1.0$ if joint reinforcement coefficient $p_s \cdot \sigma_y / \sigma_B \geqq 1.0$

$\phi = 0.9$ except all of the above

F_j = Fundamental joint concrete shear strength

$F_j = 0.1 \cdot \sigma_B$ (σ_B is concrete compressive strength)

D_j : column depth or embedment length of 90 beam main bar

b_j : effective width of joint Eq. 3

$$b_j = b_b + b_{a1} + b_{a2} \qquad (3)$$

where: b_b -beam width, b_{a1} is $b_i/2$ or D/4 whichever is smaller, b_i is distance from outer side of column to outer side of beam, D is column depth.

Here if $\phi = 0.9$, or $b_j = (b_c + b_b)/2 = 0.5 \cdot b_c \cdot (1+p)$, $D_j = D_c$ (column depth) and $\sigma_B = F_c$ are conservatively assumed values, and shear joint forces for (┼), (┬ , ┐) and (┌) types are expressed in Eqs. 4, 5, and 6, respectively.

$$V_{ju}(┼) = 0.135 \cdot F_c \cdot (1+p) \cdot b_c \cdot D_c \qquad (4)$$

$$V_{ju}(┬, ├) = 0.09 \cdot F_c \cdot (1+p) \cdot b_c \cdot D_c \qquad (5)$$

$$V_{ju}(┐) = 0.045 \cdot F_c \cdot (1+p) \cdot b_c \cdot D_c \qquad (6)$$

Moreover, Eq. 2 is the empirical equation derived from the lower bound values of joint experimental results using concrete strength of 100 to 400 kg/cm^2 having a low standard of deviation. Consequently, using Eq. 2 and its definition the following joint characteristics are summarized.

a) Beam-column joint configurations can be classified as interior (┼), exterior (├) and tee or knee (┬, ┐) types according to number of beams and columns framing the joint. Their capacity can be examined to be in proportion of about 3:2:1 for (┼),(├) and (┬,┐)types, respectively which shows a large variation.

b) Beam-column joint capacity is proportional to the effective section given by the product of average width of column and beam, and column depth. However, in case of narrow beam width connecting eccentrically with column, the effective area is reduced.

c) Beam-column joint capacity is directly proportional to compressive strength of concrete, but the increase in shear reinforcement increases the capacity by only 10%. On the other hand, shear capacity of interior joint with two transverse beams is 20% higher than the joint with only one beam.

d) Shear failure on joint is mainly governed by diagonal compressive failure of concrete, and the behavior of this kind failure shown is not so violent.

SIMPLIFIED EVALUATION METHOD OF JOINT SHEAR FAILURE

Relationship Between Joint Shear Force, Q_j and Beam or Column Shear Force (Q_c,Q_g)

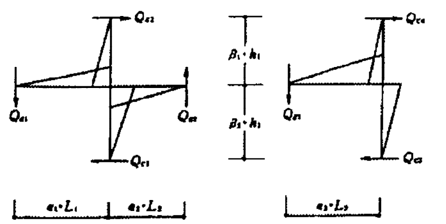

(a) ┼-Type Joint (b) ├-Type Joint

Fig.2. Distribution of bending forces in (+) and (├) type joint

The equilibrium of forces in (┼) and (├) type joint under lateral force as shown in Fig. 2, can be expressed in Eqs. 17 and 18. Likewise, for (┬) and (┌) joint types a similar equations can be derived, but with some modification. Here the relationship between the inflection points in column and beam, and the shear forces Q_c (top) and Q_c (bottom) are simplified in the following. This simplification is an important assumption in this study and it affects the accuracy of the equations giving the relationship between Qj, Qc, and Qg.

$\alpha_1 \cdot L_1 + \alpha_2 \cdot L_2 = L$ (L is the average span on the left and right side) (7)

$\beta_1 \cdot h_1 + \beta_2 \cdot h_2 = h$ (h is average height of upper and lower column) (8)

$\alpha_1 \cdot L_3 = 0.5 \cdot L$ (9)

$Q_{g1} = Q_{g2}$ (10)

$Q_{c2} = Z \cdot Q_{c2}$, $Q_{c3} = Z \cdot Q_{c4}$ (11)

$Z=[N(N+1)-i(i+1)]/[N(N+1)-i(1-1)]$ (12)

Where N is number of story, i-th story location of joint under consideration, and $T_1 \sim T_3$ are tensile forces in beam due to $Q_{g1} \sim Q_{g3}$. Also Eq.12 is based on the inverted triangular external force distribution.

$Q_{g1} \cdot L = 0.5(1+Z)Q_{c1} \cdot h \text{ --> } Q_{c1}=L/[0.5(1+Z)h] \cdot Q_{g1}$ (13)

$0.5Q_{g3} \cdot L=0.5(1+Z)Q_{c3} \cdot h \text{ --> } Q_{c3}=0.5L/[0.5(1+Z)h] \cdot Q_{g3}$ (14)

$T_1 =Q_{g1} \cdot (L-D_c)/2/j_g$, $T_3=Q_{g3} \cdot (L-D_c)/2/j_g$

$Q_{j1}=2T_1 -Q_{c2} =Q_{g1}[(L-D_c)/j_g - 2 \cdot Z \cdot L/(1+Z)(1/h)]$ (15)

$Q_{j3}=T_3 - Q_{c4} =0.5Q_{g3}[(L-D)/j_g -2 \cdot Z \cdot L/(1+Z)(1/h)]$ (16)

Here jg is the lever arm distance in beam. Substituting $L=n \cdot h$, $D_c=m \cdot h$, $j_g=q \cdot h$, $r=m/n$, the following functions are obtained.

$Q_{j1}=Q_{c1} [0.5(1+Z)(1-r)-Z \cdot q]/q$ (17)

$Q_{j3}=Q_{c3} [0.5(1+Z)(1-r)-Z \cdot q]/q$ (18)

$f_j = \tau_c / \tau_j =q/[0.5(1+Z)(1-r)-Z \cdot q]$ (19)

This means Eq. 19 can be expressed as the ratio of the average shear stress in lower story column regardless of exterior or interior column ($\tau_c =Q_c /b_c \cdot D_c$) and average shear stress of joint at upper end of column (τ_j in this case is the value in column section in the following, $\tau_j =Q_j /b_c \cdot D_c$) in case of Eqs. 7 to 12 are approximately consist of moment resisting frame as defined in the followings.

a) ratio of beam lever arm(j_g) to story height(h), $q'=j_g/h$.

b) ratio of column depth(D_c) to span(L), $r=D_c/L$.

c) ratio of shear forces in the lower story to shear forces in the upper story under consideration, Z (from Eq. 12).

Shear stress, τ_c using the joint shear force capacity calculated from Eq. 2 as in τ_j of Eq.19 is proportional only to the function f_j such as location of joint under inspection, members length and depths. And even the how large the column and beam shear and flexural stress are, it did not exceed that value.

Accordingly, whatever degree of f_j and τ_c values may become, an investigation on typical sectional dimensions of building components is carried out.

Consideration on Ratio of Shear Stress in Column and Joint

Fig. 3 shows the variation of Z value using the N and i values. As shown in the figure, Z is around 0.6 to 1.0. The ratio of $\tau_c(Z=1.0)/\tau_c(Z=0.6)$ is 1.20 using L=6 m, h=3 m, D_c =60 cm, $D_g = j_g/0.8$=60 cm, therefore the effect of Z is not so big. Accordingly, considering the joint above the first story column of three-story building (N=3, i=1, Z=0.833), the value of $f_j = \tau_c/\tau_j$ can be calculated using Eq.19 by combining the values of r and q'as shown in Fig.4 where the values of $j_g = 0.8D_g$ and q=0.8q' are plotted. As shown in the figure the value of r is not so sensitive to the value of f_j. Also for exterior and interior joint, τ_c/τ_j is about 0.1~0.5, but especially for a small range values of q'(0.1~0.2), f_j = 1.2q'~1.4q'.
That means, τ_c is (0.015~0.04)σ_B because τ_j is about (0.15~0.2)σ_B and for small value of σ_B, τ_c is approximately lower than 10 kg/cm 2 . Therefore, column capacity at joint deterioration is not so large and the joint failure is usually possible to occur.

Column Stress at Joint Failure

The relationship between exterior and interior joints average shear stress, τ_{ju} and lower columns average shear stress, τ_{cju} is approximately expressed as

$$f_j = \tau_{cju}/\tau_{ju} = 0.8q'/[0.5(1+Z)(1-r)-0.8Z \cdot q'] \tag{20}$$

Assuming interior frame joint restraint of ϕ=1.1 or 0.9, σ_{cju} of column framing the exterior and interior joint with σ_B=180 or 210 kg/cm 2 , the results are shown in Fig.5 using the values $p = b_g/b_c$ =2/3, r=60/700, N=3 and i=1. From this figure, the ranges of values of column shear stresses in the exterior and interior joints of the inner frame are 6.0~26.7 and 4.0~17.8 kg/cm 2 , respectively. This shows relatively small values. Also in column with less reinforcement in the exterior frame, using σ_B=180 kg/cm 2 the value is around 0.7 τ_c as shown dushed line in Fig.5.

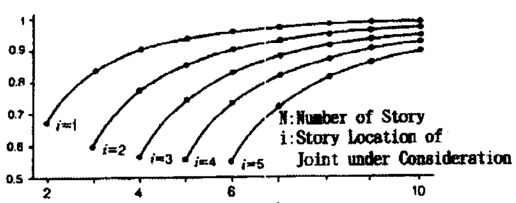


Fig.3. The ratio of shear forces of column of up story to down story, Z

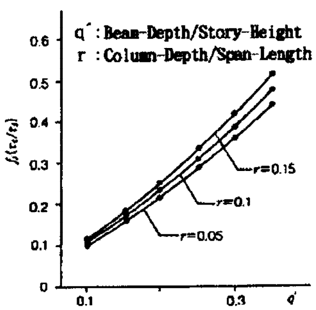


Fig.4. The f j value depend on q' and r

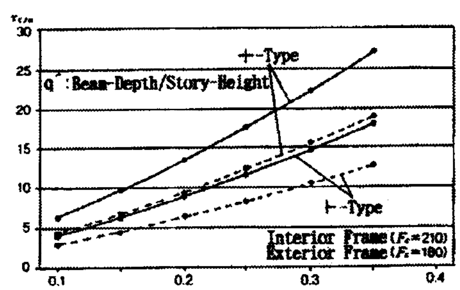


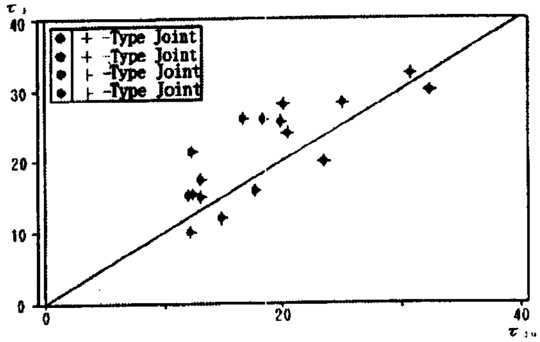
Fig.5. Relationship between q' and τ_{ju} of column at joint failure

Comparison of Simplified and Exact Values of Joint Stress

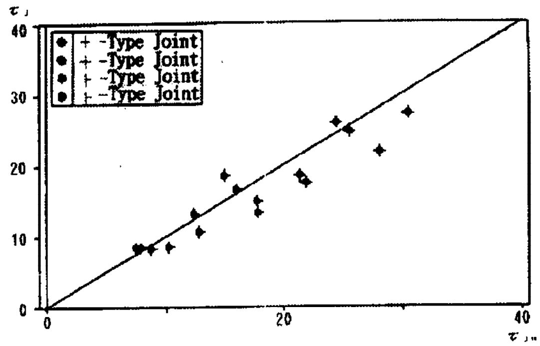
In the design examples of three- and five-story RC buildings, the results of the simplified calculation of joint shear stress ($\tau_j = Q_j/b_c \cdot D_c$) at joint failure mechanism from Eqs.17 and 18, and the exact calculation of joint shear stress at beam yielding ($\tau_{jo} = V_j/b_c \cdot D_c$) as shown in Fig. 6 are categorized as of top, middle and bottom story. From the figures, simplified values in the bottom and middle story are much closer to the exact values, but in the top story there is a big difference. This is due to the fact that in the top story or in T-type joint yielding occurs in column, but in the exact calculation beam yielding is assumed.
However, from these figures the ratio of exact and simplified calculations can be modified by certain factors depending on the desired location of joint.

INVESTIGATION METHOD OF SEISMIC PERFORMANCE AT JOINT SHEAR FAILURE

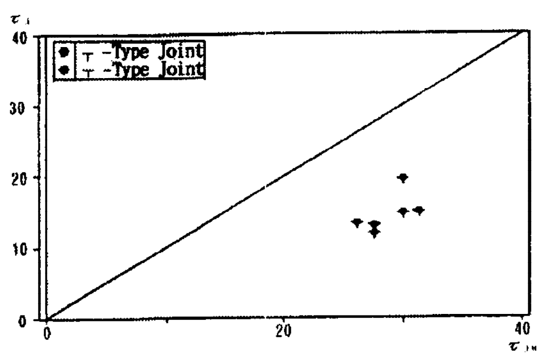
The f_j value which is the ratio of shear stress of a joint panel and shear stress of column at joint failure is almost constant in a story of a building regardless of the individual column. On the other hand, buildings with joint failure during the 1995 Hyogo-Ken earthquake similar behavior was confirmed in different stories of different joints.
In this regard, in a certain building if same lateral deflection produce joint deterioration and the story shear force is expressed as joint shear stress which is the sum of column shear stress are assumed, it can be presumed that a desirable dimension of beams and columns are necessary to prevent joint collapse, because shear coefficient in some story at joint failure is a function of column to floor area ratio and beam depth to story height ratio.



(a) Bottom Story



(b) Middle Story



(c) Top Story

Fig.6. Comprison of simplified values (τ j) and exact values (τ j o)

Joint Shear Capacity in Every Region and its Summation

Converting the joint shear stress in Eq. 2 into shear stress per column area will give Eq.21.

τ ju =Vju /(b c ・D c)= κ ・ ϕ ・0.5・(1+p) α ・F j = α j ・F j (21)

where, α j = κ ・ ϕ ・0.5(1+p)・ α

Calculating α j in a certain location of joint and assuming a general values for α and p, the results are summarized in Table 1. As shown in this table there is a big deviation in values of α j for example a building with four columns and buildings with many spans in both directions.

For building with m spans, n frames and with equal column sections, the average value, $\bar{\alpha}$ j of α j can be expressed as Eq. 22 and the results are listed in Table 2.

$$\bar{\alpha}\ j = \frac{m[4.04+2.48(n-2)]}{n(m+1)} \qquad (22)$$

From the table, it is clear that a fairly large difference of $\bar{\alpha}$ j for buildings with few spans which depends on the number of frames by expressing the different capacity of all the joints failed simultaneously.

Table 1. The ultimate strength coefficient α j depend on location of joint

Location	├-Type Joint	┼-Type Joint
Exterior Frame	2×0.9×0.75×0.75=1.01	3×0.9×0.75×1=2.02
Interior Frame	2×1.1×0.75×0.75=1.24	3×1.1×0.75×1=2.48

※P= b_K / b_c =0.5, ├-Type : α = D_j / D_c =0.75

Table 2. The avarege value $\bar{\alpha}$ j of α j for building with m spans and n frames

n	m=1	m=2	m=5	m=∞
2	1.01	1.34	1.68	2.02
3	1.09	1.45	1.81	2.17
5	1.15	1.53	1.91	2.30
∞	1.24	1.65	2.08	2.48

Shear Force Coefficient C jui at Joint Failure

The shear coefficient, C jui of a moment resisting frame in the i-th at joint failure is expressed in Eq. 23 as the ratio of total column shear capacity and building weight at the i-th floor.

C jui = Σ Q cju i /W i (23)

where, Q cju =f j ・ τ ju ・A c , τ ju = α j ・F j , W=w i ・ Σ A fj

wi is the average floor unit weight (t/m 2)

Σ A fi is total floor area in the i-th floor

a ci = Σ A c i/ Σ A f i (column to floor area ratio at ith floor) also, if Σ(f j ・ τ j u ・A a) i is expressed as f ji ・F ji ・ α ji ・ Σ A c i

C jui = Σ(f j ・ τ ju・A c)i /w i ・ Σ A fi ≒ F ji ・f ji ・ $\bar{\alpha}$ ji ・a ci /wi (24)

where, F ji =0.1 ・ σ B , f ji can also be taken conservatively as 1.2 ・q'

C jui =0.12 ・ σ B ・q' i ・ $\bar{\alpha}$ ji ・a ci/w i (25)

As shown in Eq. 25, shear coefficient at joint failure can be approximately expressed as a function of compressive strength of concrete, beam to story height ratio (q') column to floor area ratio (a c) building unit weight and joint coefficient, α j , which is greatly influenced by the number of spans. For this reason, increasing the amount of main bars in column and in beams cannot induce a large increase in shear coefficient. Also in buildings with four columns the column to floor area ratio should be 2.5 times as in buildings with many columns, in order to prevent joint collapse. From Eq. 25, the floor to column area ratio, a ci at the i-th floor can be expressed using coefficient A i of shear distribution in the design application as in Eq. 26 to ensure a desirable base shear coefficient C B

a ci =(A i ・C B ・w i)/(0.12 σ B ・q' i ・ $\bar{\alpha}$ ji) (26)

Substituting A i =1, C B=0.4, w i =1200kg/m 2 , σ B=200 kg/cm 2 and $\bar{\alpha}$ ji as listed in Table 2 in Eq. 26, the results are listed in Table 3.

As shown in Table 3, the necessary column to floor area ratio which depends on number of spans, number of frames, and beam to story height ratio should be different to each other to assure a yielding shear coefficient of 0.4, and the difference in maximum and minimum values is about four times. And also using the values in the table to determine the value of C in order to prevent joint failure in buildings, the necessary section of column can be decided in the preliminary design stage.

Table 3. The necessary a ci to prevent joint failure for building with m spans and n frames (Unit : cm^2/m^2)

Number Of Span	m=1			m=2			m=5			m=∞		
q'	0.20	0.25	0.30	0.20	0.25	0.30	0.20	0.25	0.30	0.20	0.25	0.30
n=2	99	79	66	75	60	50	60	48	40	50	40	33
n=3	92	73	61	69	55	46	55	44	37	46	37	31
n=5	87	70	58	65	52	44	52	42	35	43	35	29
n=∞	87	65	54	61	48	40	50	38	32	40	32	27

JOINT SHEAR CAPACITY OF STEEL REINFORCED CONCRETE BUILDING

The ultimate joint shear capacity of steel reinforced concrete buildings is expressed in the following equations as the moment capacity which is product of effective beam stress distance, j and joint shear capacity V_{ju} in Eq. 2 (Refer to AIJ Standard for definition of terms).

$${}_jM_a = {}_cV_e({}_jF_s \cdot {}_j\sigma + {}_wp \cdot {}_w\sigma_y) + 1.2\,{}_sV \cdot \sigma_y/\sqrt{3} \quad (27)$$

Eq. 27 is can be expressed in the following if Eq. 2 and similar equations is adopted and using the spacing between both side bars of beams and columns as 0.8×member depth, or beamand column steel flange distance as 0.6× member depth

$$V_{ju}(SRC) \fallingdotseq 0.5(1+p)\kappa \cdot F_j \cdot \phi_{SRC} \cdot b_c \cdot D_c \quad (28)$$

$$\phi_{SRC} \fallingdotseq 0.96 + 0.8\,{}_wp \cdot {}_w\sigma_y/(\kappa \cdot F_j) + 0.33 t_w \cdot {}_s\sigma_y/(b_j \cdot \kappa \cdot F_j) \quad (29)$$

Substituting p=0.2%, ${}_w\sigma_y = {}_s\sigma_y = 3000$ kg/cm 2 , t=0.9 cm, $b_j = 50$ cm,

$\kappa \cdot F_j = 0.3 \cdot 200$ kg/cm 2 in Eq. 28.

$\phi_{SRC} \fallingdotseq 0.96+0.06+0.33=1.35$

In case of 9 mm web plate thickness in SRC structure is used an increase in value of ϕ of about 30% in RC structure will result which is from 0.9～1.1. In Eqs. 3～26 ϕ_{SRC} may be used instead of ϕ.

Generally, SRC structure is used in medium- and high-rise buildings in Japan and it is considered that SRC joint panel is more likely to be damaged under strong motion than the joint of RC structures even the joint shear strength capacity SRC is larger than RC structures,because the sectional dimension of column in SRC structure is 10～20% smaller than RC structure due to its strong load bearing capacity resulting from the use of steel members.

JOINT SHEAR CRACKING CAPACITY

The joint shear cracking capacity, V_{jsc} can be estimated as

$$V_{jsc} = F_t\sqrt{1+\sigma_0/F_t} \cdot b_j \cdot D_j \quad (30)$$

where, F_t - concrete tensile strength, i.e., $1.6\sqrt{\sigma_B}$

σ_0 - average axial stress in joint

b_j , D_j - same definition as in Eq. 2

The average shear stress at joint cracking is expressed in Eq. 31 similar to joint shear strength in Eq. 21.

$$\tau_{jc} = V_{jsc}/(b_c \cdot D_c) = F_t\sqrt{1+\sigma_0/F_t} \cdot 0.5(1+p) \cdot \alpha \quad (31)$$

And converting this value into average shear strength of column the following equation can be obtained.

$$\tau_{cjc} = f_j \cdot \tau_{jc} \quad (32)$$

Using the values σ_B=225 kg/cm 2 , p=0.5, σ_0 =0, 0.2 σ_B, 0.4 σ_B, and f_j =0.25 in the calculation of τ_{cjc} in Eq. 32 and ratio of τ_{jc}/τ_{ju} of Eqs. 21 and 31, the results are tabulated in Tables 4 and 5. From Table 4, the values of shear joint cracking stress increases from 3～10 kg/cm 2 as the average axial stress in column increases. As a result, joint panel cracking occurs easily under strong lateral forces because this value is smaller than the typical value at flexural mechanism. However based on the values of Table 5, especially in interior joint even the shear cracking has been reached there still a residual capacity that can be expected.

Table 4. The simplified values of average shear cracking stress of column, τ_{cjc} at joint shear cracking

σ_0 / σ_B	0	0.2	0.4
┼ -Type	4.5	7.6	9.8
├ -Type (a=0.7)	3.2	5.3	6.9

Table 5. The ratio of shear cracking strength to shear ultimate strength of the joint panel (τ_{jc} / τ_{ju})

σ_0 / σ_B	0	0.2	0.4
┼ -Type	0.32	0.55	0.70
├ -Type	0.55	0.93	1.21

SIMPLIFIED EVALUATION METHOD OF BEAM-COLUMN JOINT

Application in Design of New Building

Eqs.20 and 21 can be applied to determine the beam section to assure that no joint failure will occur by assuming appropriate column section and building deformation ability coefficient, D_s.
Investigation of the baem-column joint in new buildings are proceeded as follows.

(a) Determine span L, story height h, and number of story N.
(b) Assume concrete strength F_c.
(c) Estimate column load.
(d) Assume D_s ,F_{es} :horizontal capacity increasing coefficient due to irregular horizontal rigidity distribution in height and in plan.
--->Calculate Q_m at column shear yield mechanism.
(e) Determine column section b_c x D_c.
(f) Determine beam depth ratio $p=b_b/D_c$.
(g) Calculate required beam depth ratio q or q'.
(h) If the assumed beam depth is less than calculated depth.
--->OK (joint shear failure will not occur).
(i) If D_g is larger than the designed value, using any of the following suggestions or in combination the investigation shall continue to satisfy the condition.
1. Increase the beam width
2. Use hunch beam
3. Increase F_e
4. Increase the column section

Application to Existing Building Seismic Diagnosis

In Japan, a seismic screening procedure from the simple first level to a complicated third level is applied for seismic diagnosis of existing buildings. In the first level, average unit shear stress referring to shear failure and flexural failure of members are used to estimate the capacity of columns. Here, a lower boundary shear stress of columns at joint failure calculated from Eqs. 20 and 21 will be used as one of the critical ultimate values compairing it with the above-mentioned critical values.

Estimation of Seismic Performance of Damaged Buildings with Joint Failure

Concerning the seismic performance evaluation of old buildings damaged by an earthquake, there are many cases with few structural information. In this cases, Eqs. 24 or 25 may be used to estimate shear coefficient of the building at joint failure C_j based on fundamental structural dimensions. And based on the calculated C_j, estimation of the acting earthquake motion may become possible.

CONCLUSION

Beam-column joint must be safeguarded against the brittle failure in order to improve the seismic performance of the buildings. In this paper, according to the recent research results, simplified estimation method of column stress at joint panel failure and calcula-ted results, and application of this simplified equations are described based on the equations to estimate the ultimate joint shear capacity. The results are as follows:
a. Comparing the mean shear stress of joints τ_j to mean shear stress of columns τ_c will result in about 1/2~1/10 in a typical moment resisting frames.

b. The ratio of $\tau c / \tau j$ is strongly influenced by beam to story height ratio, q'. At q'<0.2 the ratio $\tau c / \tau j$ =1.2q' ~ 1.4q'.
c. For this purpose, τc at joint failure is 0.01Fc ~0.04Fc. When Fc is low, the value of τc become fairly lower than 10 kg/cm 2.
d. The possibility of damage in SRC joint panel is possibly larger than RC structures under strong earthquake.
e. Mean shear stress of column at joint cracking τjc increases, the axial stress σo is almost larger by around 3 ~ 10 kg/cm 2 in average.
As discussed above, joint failure controls the seismic performance of the SRC or RC buildings with slender columns and beams. Also in order to prevent shear failure the related equations described above is recommended to be applied. Therefore, this simplified method can be applied in the preliminary sizing of members in design of new buildings and evaluation of existing buildings. And, it is evident that the lateral strength capacity of buildings with damaged joint can be predicted.

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