13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1766

CONCEPT OF STORY SHEAR COEFFICIENT C_J FOR SHEAR FAILURE OF BEAM-TO-COLUMN JOINT IN R/C BUILDINGS AND ANXIETY PRESSURED FROM DISTRIBUTION OF C_J IN THE EXISTING R/C BUILDINGS

Masaya HIROSAWA[1], Jiandong ZHOU[2], Tatsuya KOMURO[3]

SUMMARY

After the 1995 Great Hanshin Earthquake where many R/C buildings suffered from distinguished shear-failure in the beam-column joints, a concept for story shear coefficient for shear failure of beam-to-column joint was proposed to explain the reason why the damages occurred.

Also after the earthquake, experimental studies on the joint with eccentric connection of beam to column and analytical studies on the scale effect of shear capacity of joint panels were carried out and their influence to the joint shear capacity was made clear not to be negligible. Further more, it was made clear that there are many existing R/C buildings with the eccentric joints.

Here in this report, the authors shows the proposals for shear capacity deterioration ratios due to the eccentric connection of beams to columns and due to the scale effect, and show also statistical data on the coefficient C_j and C_{jts} including reduction factors concerning to the existing school buildings and apartments. Due to the investigated results on the 40 existing medium- and high-rise apartments of reinforced concrete or steel framed reinforced concrete, the eccentric connections of beams to columns were recognized in all buildings and the buildings with C_{jts} smaller than 0.35 amounted more than 50% of the total.

Through the discussion, authors want to insist on the importance of the joint shear failure of reinforced concrete medium- and high-rise buildings and the importance to consider the influence of such affecting factors as eccentric connection to the joint shear capacity.

1. INTRODUCTION – LEARNING FROM THE HYOGOKEN-NANBU EARTHQUAKE

After the 1995 Hyogoken-Nanbu Earthquake, great attention was paid to the fact that the buildings that had been designed under the latest Japanese seismic codes ("Phase-III Buildings"), including relatively

[1] Professor, Kogakuin University, Tokyo, Japan. Email: dt10760@ccs.kogakuin.ac.jp
[2] Guest researcher, Kogakuin University, Tokyo, Japan. Email: zhoujiandong2002@yahoo.co.jp
[3] Graduate student, Kogakuin University, Tokyo, Japan. Email: BQB03635@nifty.ne.jp

new 5- to 11-story apartment houses, suffered significant damages to their beam-column joints. These buildings were not limited to reinforced concrete (RC) structures but included several steel reinforced concrete (SRC) structures, many of which had to be demolished. The seismic performance of beam-column joints had long been catching attention as a continuing research focus, but no general seismic codes were available for RC structures in Japan, unlike SRC structures for which such codes had been in place.

Various post-earthquake examinations into the joint damages revealed several problems about post-1981 seismic design methods [1)]. The strength reduction of eccentric beam-column joints was one of such problems identified. According to some researches, eccentric beam-column joints were mainly found in SRC apartment houses and RC school buildings, and more than 80% of all buildings had eccentric beam-column joints, many of which have been reported to get a strength reduction by almost 30% as compared with non-eccentric joints[1), 14)]. In addition, recent researches have reported that full-scale joint tests resulted in a relative reduction in joint strength because of the larger dimensions than scale models ("scale effects") [11)]. It was also gradually found that the values of joints' shear to flexural strength ratio and the story shear coefficient at joint failure calculated for apartment houses and school buildings were varied significantly in many cases, mainly due to dimensional variations including column depth, and that many buildings could not be regarded safe even when the strength reduction induced by eccentric joints or the scale effects is considered.

The researches conducted so far have found that, unlike individual beams and columns that could be given a higher strength than before through repair work even when they have undergone major damages, it is extremely difficult to perform realistic repair or reinforcement work on badly damaged beam-column joints because they tend to experience large deformations and cause severe damages affecting all stories within the system. Thus, buildings suffering such joint damages are usually considered difficult or economically unfit for reuse and usually subject to demolition. Generally speaking, increasing the sizes of columns and beams is effective in reducing the potential damage of beam-column joints. Through a rational design approach, columns and beams would not have to be much larger in size than those of existing buildings, even under the assumption that beam-column joints remain repairable under extreme seismic motions. However, because beam-column joints are given thorough seismic design, it is estimated that several tens of percent of new mid-to-high rises are being constructed with failure-prone joints every year.

Under the trend of resource saving and environmental conservation, efforts to enhance the durability and service life of buildings are inevitable. To this end, it should be pointed out that establishing better seismic design criteria for beam-column joints is extremely important to improve the seismic performance of building structures.

2. CHARACTERISTICS OF BUILDINGS PRONE TO BEAM-COLUMN JOINT DAMAGES AND THE BACKGROUND FACTORS

2.1 Damage overview and background factors

In October 1998, the Architectural Institute of Japan published a comprehensive analytical report on beam-column joint damages in RC-group (RC and SRC) structures struck by the 1995 Hyogoken-Nanbu Earthquake [1)]. The report identified the following characteristics of the buildings that had suffered joint damages. It should be noted that though the report stated 48 buildings as the total number of damaged buildings, the actual number is unknown, which is supposed to be much more.

i. With regard to the building purpose, apartment houses (37 buildings or 77%) and school buildings (8 buildings or 17%) were found to be the major types, and all suffered damages in the longitudinal direction consisting of pure frame systems with fewer walls.

ii. With regard to the damage level, the number of buildings that collapsed, suffered major damages and suffered intermediate damages was 13 (27%), 17 (35%) and 18 (38%), respectively. Those with damaged joints suffered most severe damages.
iii. Medium rises (3 to 7 stories) and mid-to-high rises (8 to 14 stories) accounted for a half of the total (50%), which were 24. No low rises.
iv. Apart from 8 buildings whose ages of construction were not clear, many were relatively new ones: 4 (10%) were built before 1970 ("Phase-I Buildings), 10 (25%) were built between 1971 and 1980 ("Phase-II Buildings) and the rest, 26 (65%) were built after 1981 (the "Phase-III Buildings).
v. With regard to the number of spans on the short side, 18 (51%) had one, 8 (23%) had two, and 9 were with varied numbers. Thirteen buildings were excluded as their span details were not confirmed.
vi. Out of 9 buildings that allowed confirmation of joint eccentricity, 7 were found to have eccentric joints.

Based on the above data, the characteristics of the buildings prone to beam-column joint damages were identified as follows.

a) Mid-to-high rises with pure frame systems (i.e., prone to large deformations);
b) Buildings conforming to the post-1981 seismic codes (i.e., do not allow the columns to undergo shear failure easily, and as a result tend to have relatively weak joints);
c) Buildings with fewer spans (i.e., tend to have perimeter columns with relatively weak joints)
d) Buildings with eccentric beam-column joints (i.e., strength is reduced by eccentricity)

On the other hand, the following facts were identified as the background factors as to why so many RC-group structures had suffered joint damages, which had hitherto been a rare occurrence in Japan:

i. The 1981 enforcement of the latest seismic codes meant more proper calculation of design seismic load, improvement of ductility to columns and shear walls and more appropriate handling of buildings with seismically unfavorable shapes. As a result, the brittle failure of members especially the shear failure of columns and local collapses such as story collapse have significantly been reduced. On the other hand, joints became the weakest points in many structures because the design consideration is not required for the beam-column joints in RC structures.
ii. The enforcement of latest seismic codes gave rise to the design trend of providing full seismic slit for secondary walls to improve the ductility of the structure. As a result, more buildings are designed with less reserve capacity in seismic strength, allowing the flexural yielding stress of beams or columns to be inputted into joints easily.
iii. Enhancement of concrete strength meant that columns and beams could be designed with smaller section sizes.
iv. Seismic motions with long-period components struck urban areas packed with mid-to-high-rises, which were susceptible to the influence of beam-column joints' seismic performance problem.
v. There was a notable increase in the number of design applications that used joints in which the beams were placed eccentric to the column, especially for apartment houses.

2.2 Analysis of buildings with joint damages

Two buildings examined in the above report [1)] are cited here, namely a 9-story RC apartment house (*Jeunesse Rokko* [2)], Photo 1) and a 11-story SRC apartment house (*Cosmo Ashiyakawanishi* [3)], Photo 2). The values of equivalent story shear coefficient at the shear failure of joints C_j (see equation (1) described in a later section) were calculated for these buildings, which are shown in Figure 1 and Figure 2, respectively, together with the damage levels of joints observed after the earthquake (I: Minor to IV: Major) [1)]. The C_j graph also shows C_{jt} values, which take into account the shear capacity reduction rate induced by the eccentricity of beam-column joint β_{jt} (see equation (2)), as well as C_{jts} values, which take into account the shear capacity reduction rate induced both by β_{jt} and by the scale effects of column size β_{js}.

It should be noted that the 9-story *Jeunesse Rokko* and 11-story *Cosmo Ashiyakawanishi* had been subject to the concrete core compressive strength testing, which confirmed their measured strengths (28.6 to 36.8N/mm^2 and 25.7-47.3N/mm^2, respectively) were well above the design strengths (24.0N/mm^2 and 21.0-24.0N/mm^2, respectively). In addition, the 9-story *Jeunesse Rokko* had been subject to the main reinforcement tensile testing, which confirmed its measured yield strength was about 1.1 times greater than the specified value. These measured strengths were used in the calculation of C_j values, (adopting the values 1.1 greater than the specified value for the steel described above). These data seem to point to the following facts:

i. In both buildings, extreme damages to the beam-column joints were observed on lower stories, and most interior columns suffered major damages.
ii. The average strength reduction rates induced by the eccentricity of beam-column joints β_{jt} and by the scale effects of column size β_{js} are both around 0.8.
iii. When β_{jt} and β_{js} are not taken into account, the C_j values of *Jeunesse Rokko* and *Cosmo Ashiyakawanishi* are 0.6 or more and 0.4 or more, respectively, which are greater on upper stories.
vi. When β_{jt} and β_{js} are taken into account, the C_{jts} values of *Jeunesse Rokko* and *Cosmo Ashiyakawanishi* are 0.4 or more and 0.3 or more, respectively.

a) Panorama view

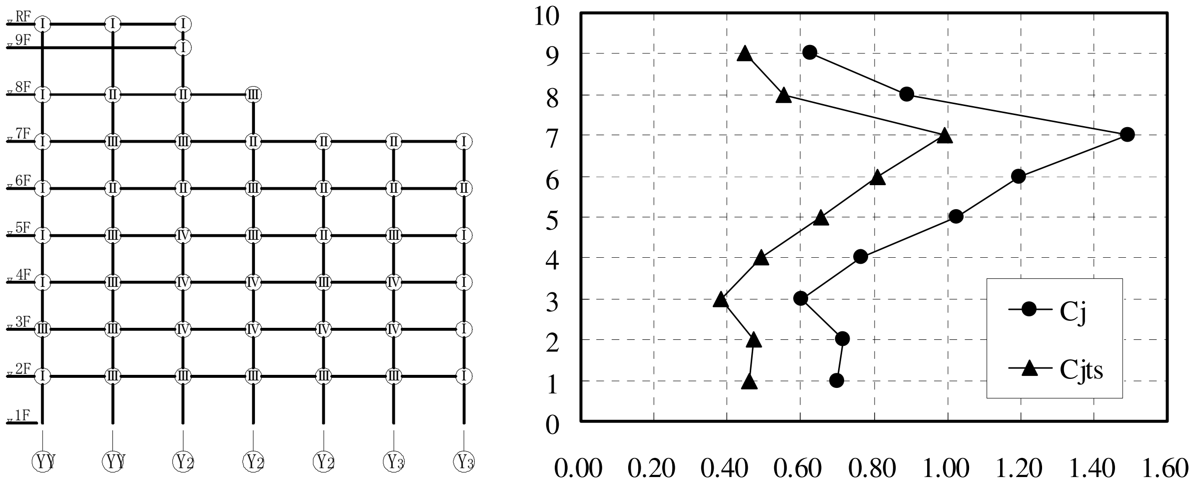
b) Damage on an east side lower story part

Photo 1 Damage on the beam-column joints occurred in the RC apartment house of 9 story

a) Beam-column joint on the 2nd Floor

b) Concrete crush observed in a concrete core drawn out from the damaged joint

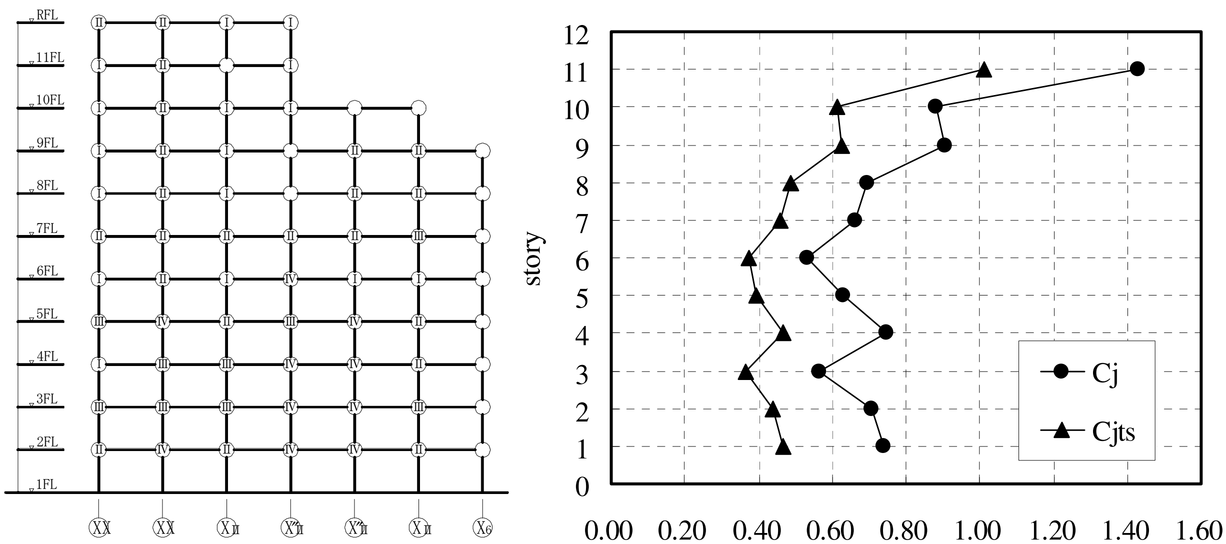
Photo 2 Damage on the beam-column joints occurred in the SRC apartment house of 11 story



a) Damage levels of beam-column joints

b) Distribution of C_j and C_{jts}

Figure 1 Damage level and C_j of the RC apartment house of 9 story (*Jeunesse Rokko*)



a) Damage levels of beam-column joints

b) Distribution of C_j and C_{jts}

Figure 2 Damage level and C_j of the RC apartment house of 11 story (*Cosmo Ashiyakawanishi*)

The analysis of beam-column joint damages has thus shown that, for the purpose of evaluating the seismic performance of joints, it is reasonable to consider the influence of joint eccentricity and scale effects, and that the application of equivalent story shear coefficient at the shear failure of joints is a valid concept.

3. PROPOSAL FOR THE EVALUATION OF BEAM-COLUMN JOINTS' SHEAR PERFORMANCE

3.1 Concept of story shear coefficient at the shear failure of beam-column joints C_j

The concept proposed here is to roughly calculate the seismic story shear force induced at the shear failure of each story's beam-column joints, obtain its ratio to the design input seismic load equivalent to gravity acceleration, and thereby to define it as the equivalent shear coefficient C_j to be used as an index of building's susceptibility to joint shear failure at each story. Under the assumption that all joints on a given story undergo shear failure at once, C_j can be given as a function of such terms as the ratio of the total column area to the total floor area a_c and the unit weight of the building w_i, as shown in equation (1).

Given the strength reduction induced by the eccentric joints or scale effects, the C_j values of an ordinary building should be above the lower limit of structural characteristics factor D_s (representing the shear coefficient equivalent to the necessary horizontal load-carrying capacity) of a pure frame system by a relevant safety factor. Depending on the presence or the extent of eccentricity, they would need to be around 0.4 to 0.45 or more.

According to this approach, the necessary value of ac is around 40 to 50 cm^2/m^2 when the C_j value is set at 0.45 and the beam depth at values 0.25 to 0.3 times of the story height. However, in existing buildings, the values of a_c are often found to be below this level.

Equivalent story shear coefficient at the shear failure of joints C_{jui}

$$C_j = \frac{\sigma_B^{0.7} \cdot \overline{f_j} \cdot \overline{\alpha_j} \cdot a_{ci}}{w_i \cdot A_i} \qquad (1)$$

$\overline{f_j}$: average f_j of the joints on the i-th story

$\overline{\alpha_j}$: average joint strength coefficient α_j on the i-th story; $\alpha_j=1.6\kappa \cdot \varphi \cdot (b_j/b_c) \cdot (D_j/D_c)$

a_{ci} : ratio of the total column area to the total floor area, given by:

$a_{ci}= \Sigma A_{ci}/ \Sigma A_{fi}(cm^2/m^2)$

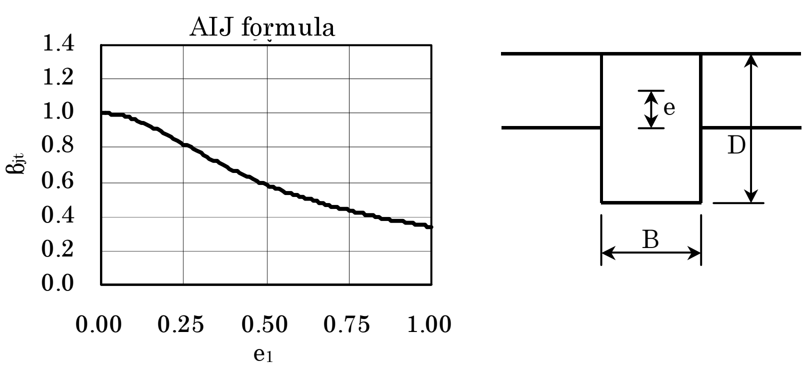
where $\Sigma A_{ci}(cm^2)$ is the total column section area on the story, and $\Sigma A_{fi}(m^2)$ is the total area of all floors up to the story.

A_i : seismic story shear distribution coefficient of the i-th story

w_i : unit weight of the building calculated for the part above the i-th story (kg/m^2)

3.2 Shear capacity reduction rate at eccentric beam-column joints

The seismic damage analysis revealed that there were numerous cases where buildings suffered damages to joints in which the beams were placed eccentric to the columns. The results of the past tests on eccentric joints also showed that the greater the extent of eccentricity, the lower the strength. As a means of evaluating the influence of eccentricity, equation (2) has been proposed to give the strength reduction rate induced by eccentricity β_{jt} [1]. In Figure 3, β_{jt} is given as a function of eccentricity e_1, where the average shearing stress at joint shear failure K_{ju} and the moment coefficient at torsional failure K_{to} are determined under the standard conditions. According to the figure, under the assumption that $B=D$ and beam width is 0.5D, e_1 and β_{jt} become 0.25 and 0.8, respectively.



$$\beta_{jt} = \left\{ 1 + \left(\frac{e_1 \cdot K_{ju}}{K_{to}} \right)^2 \right\}^{-0.5}$$

$$e_1 = e / \mathrm{Min}(b_c, D_c)$$

$$K_{ju} = V_{ju} / (B \cdot D)$$

$$K_{to} = M_{to} / (B^2 \cdot D)$$

Figure 3 Strength reduction rate induced by eccentric jointing of beam and column β_{jt}

Figure 4 shows the percentage of eccentric joints in existing school buildings in Japan, which implies they are on a gradual increase. On the other hand, the current joint shear design codes are confined to qualitative descriptions only with regard to eccentric joints, and there is no mention of how to evaluate their influence quantitatively.

Strength reduction rate induced by eccentric joints β_{jt}

$$\beta_{jt} = \left\{ 1 + \left(\frac{e_1 \cdot K_{ju}}{K_T} \right)^2 \right\}^{-0.5} \qquad (2)$$

$e_1 = e/Min(b_c, D_c)$
$K_{ju} = V_{ju}/(b_c \cdot D_c)$
$K_T = T_j/Min(b_c \cdot D_c^2、 b_c^2 \cdot D_c) = 0.80\sqrt{\sigma_B} + 0.45 p_j \cdot \sigma_{jy}$

e, e_1: eccentric distance and eccentric factor
b_c, D_c: column width, column depth
p_j: ratio of joint shear reinforcement
T_j: torsional strength of joints
V_{ju}: shear capacity of joints
σ_{jy}: yielding point of joint shear reinforcement

Equivalent story shear coefficient at the failure of eccentric joints C_{jt}

$$C_{jt} = \beta_{jt} \cdot C_{jui} \qquad (3)$$

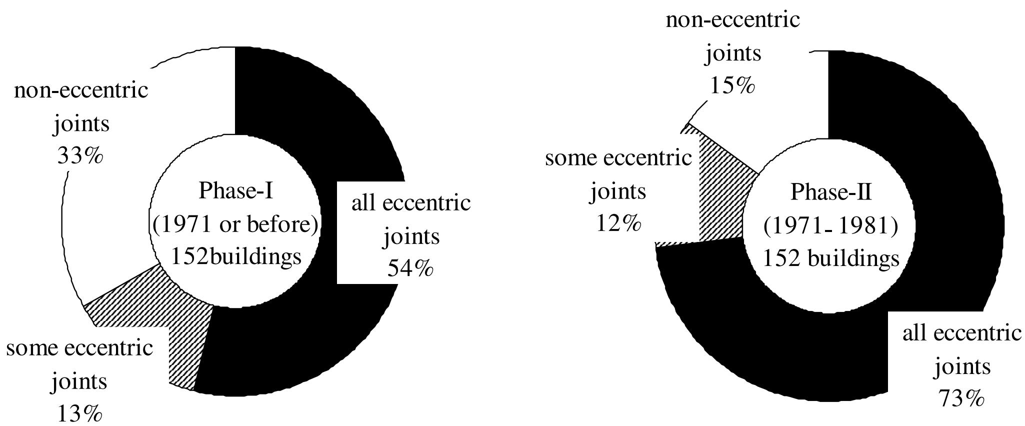


Figure 4 Percentage of eccentric joints in 152 existing RC school buildings in Japan

3.3 Average shearing stress of beam-column joints and columns at the shear failure of joints τ_{ju}, τ_{cju}

This section describes the conversion of beam-column joints' shear capacity into the average shearing stress per total section area of the columns τ_{ju}, and then into columns' average shearing stress at joint shear failure τ_{cju}. Under the assumption that concrete strength is F_c = 20N/mm^2, the beam width is a half of the column width and transverse beams are placed on one side, then τ_{ju} is given by:

$\tau_{ju} = 0.20F_c$ (┼-shaped section) to $0.08F_c$ (L-shaped section) = 4.0 to 1.6N/mm^2

On the other hand, the ratio of the shear force of columns V_c to that of joints V_j is generally given by $V_c \fallingdotseq 0.25V_j$. It means, given the value of τ_{ju} above, that the average shearing stress of columns at joint shear failure τ_{cju} is given by:

τ_{cju} = 1.0 (┼-shaped section) to 0.4 (L-shaped section) N/mm^2

While enhancement of joint shear reinforcement does not increase the value of τ_{cju}, enhancement of column shear reinforcement can increase the shear capacity of individual columns by about 2.0N/mm^2, as is generally known. These results show that the shear failure of beam-column joints is more likely to occur than the yielding of beams or the failure of columns if the average shearing stress of columns is high and the yielding of beams occurs first.

3.4 Scale effects on the shear capacity of beam-column joints

Conventionally, the shear testing of beam-column joints in RC structures had been performed using 1/2 to 1/3 scale models. However, full-scale testing was implemented because the analysis of beam-column joint damages in the Hyogoken-Nanbu Earthquake implied the scale effects on joint shear capacity. As a result, it was reported [11)] that the measured strengths were below the calculation values derived from the design criteria [6)]. In addition to this finding, another research reported on the regression analysis of scale effects on joint shear capacity based on the past conventional test results [12)].

Figure 5 shows the results of this research, together with the strength reduction rate proposed for various types of RC member strength, such as the shear capacity of RC beams. While the design criteria [6)] derived a formula from the results of testing that had been performed on scale models having a column depth of about 250 to 300mm, the strength of a full-scale column with a depth of about 700mm was found to be reduced by about 0.7 as compared with that of a scale model. Now, the strength reduction rate induced by scale effects is defined as β_{jts} and multiplied by βjt, another reduction rate induced by eccentricity. The resultant value, namely β_{jts}, is as large as between 0.5 and 0.6. Note that the C_{jts} values of damaged buildings described in section 2.2 were given by $C_{jts} = \beta_{jts} \cdot C_j$, which took into account the strength reduction rate induced by eccentric joints and scale effects, namely β_{jts}.

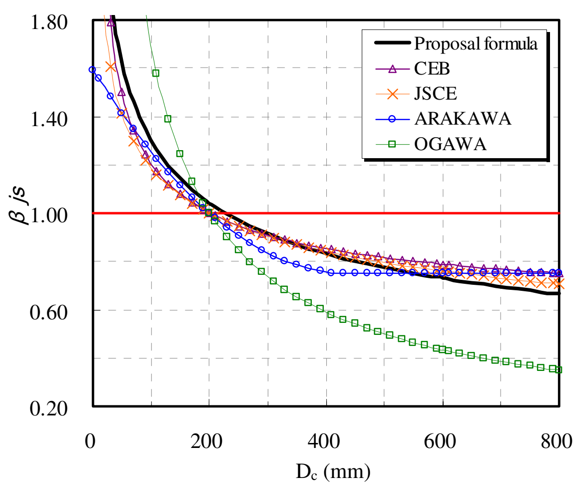


Figure 5 Strength reduction rate by scale effect

4. SAFETY LEVELS OF BEAM-COLUMN JOINTS IN EXISTING BUILDINGS AND THEIR DISPERSION

4.1 Buildings prone to beam-column joint damages

Because the shear failure of beam-column joints often occurs when large horizontal deformations are induced during earthquake, vulnerable buildings are those having pure frame systems in one direction, which are typically used as schools, hospitals and apartment houses. In particular, many apartment houses have two planes of structure in the longitudinal direction consisting of pure frame systems, mostly with eccentric beam-column joints and only a single transverse beam. Furthermore, limited by the low story height, the beam depth is often around 700mm, even in the cases of 5- to 6-story buildings. The main reinforcement ratio of beams is also greater on lower stories. The number of stories of school buildings is around 4. However, the ratio of column shear force to joint shear force f_j tends to be small due to the large story height, which gives the cause of susceptibility to joint damages. Based on these results, the following section looks at the seismic performance of joints in existing school buildings and apartment houses.

4.2 Safety levels of beam-column joints in existing school buildings and apartment houses

There have been reports on 147 existing school buildings that studied the typical story shear coefficient at joint failure C_j in the longitudinal direction and the dispersion of impact factors. Figure 6 is a part of the results, namely the frequency distributions of C_j. Figure 7 shows the average values of compressive strength of concrete cores extracted from the buildings, classified by age of construction. Figure 8 shows the average values and dispersion of the ratio of total column area to total floor area a_c on the 1st floor, classified by the number of stories.

Figures 9, 10 and 11 show similar data on the joints in 40 RC public apartment houses[13)]. Figure 9 shows the eccentricity rate of lower stories e_l on lower stories (1st to 3rd stories), and Figures 10 and 11 show the frequency distributions of different impact factors and results, those are the frequency distributions of C_j and C_{jts} and the ratio of total column area to total floor area a_c on the 1st floor, respectively. These data seem to point to the following facts with regard to the seismic safety of beam-column joints in existing school buildings and apartment houses. Note that the design concrete strength F_c (18 or 21N/mm^2) is adopted if the measured concrete strength exceeds it, which is in turn adopted if it is below the design strength.

i. The average C_j value exceeds D_s=0.3 when the eccentricity and scale effects are not considered, that is 0.5 and 0.36 for school buildings and apartment houses, respectively. On the other hand, when the eccentricity and scale effects are taken into account, the average C_j value is reduced to 0.25 for apartment houses. As expected, C_{jts} values are varied significantly, and most C_{jts} values were 0.3 or less in the case of apartment houses.

ii. The C_j and β_j values of each building are varied significantly, but they become smaller for larger story heights or the joints of lower stories. Such dispersion would be attributable to the variations in the ratio of total column area to total floor area a_c, dimensions of beam section b_g x D_g and concrete strength σ_B. It means that mid-rises are more prone to beam-column damages than low-rises and that mid-to-high-rises with smaller beams and columns having low concrete strength are likewise more prone to beam-column damages.

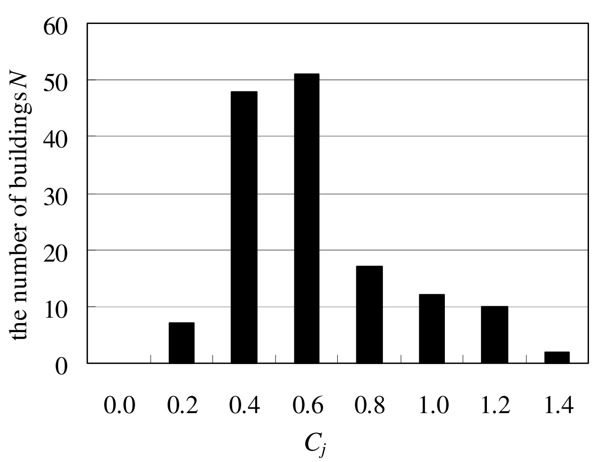


Figure 6 Frequency distributions of C_j on the existing RC school buildings

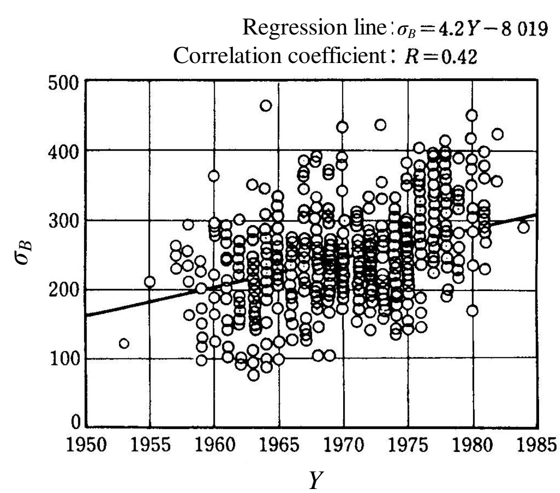


Figure 7 Relationship between the average values of compressive strength σ_B and age of construction Y on the existing RC school buildings

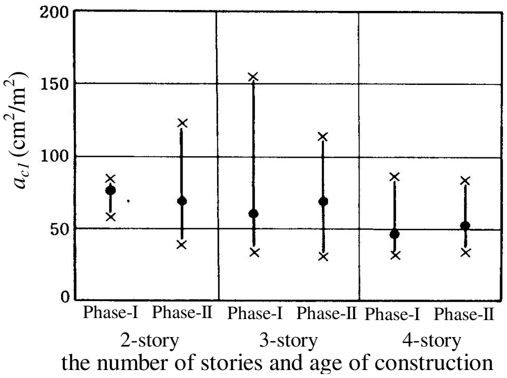


Figure 8 Ratio of total column area to total floor area a_c on the 1st floor, classified by the number of stories and age of construction of the existing RC school buildings

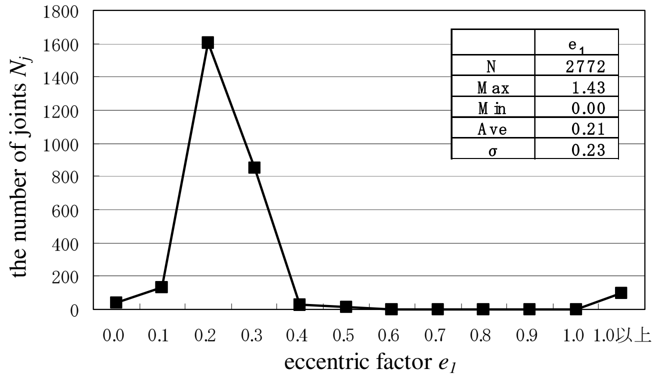


Figure 9 Frequency distribution of eccentric factor *e1* on the joints in 40 RC mid-rise apartment houses

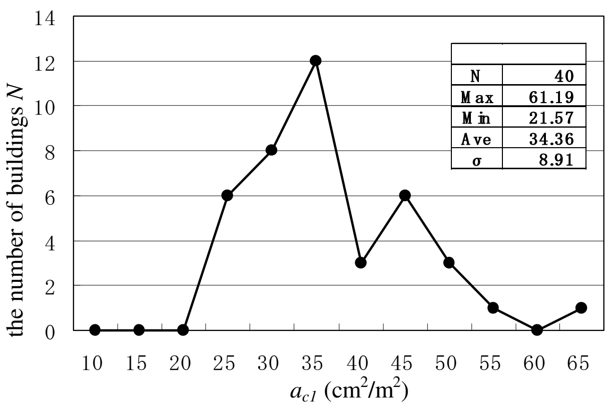


Figure 10 Frequency distribution of the ratio of total column area to total floor area a_c on the 1st floor in 40 RC mid-rise apartment houses

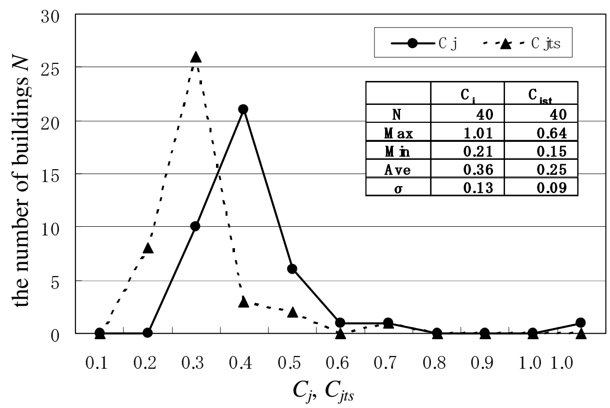


Figure 11 Frequency distribution of C_j and C_{jts} in 40 RC mid-rise apartment houses

5. CONCLUSIONS

The above examination of the seismic performance of beam-column joints in RC buildings has offered the following insights:

i. In the Hyogoken-Nanbu Earthquake, beam-column joints suffered notable damages, mainly in relatively new mid-to-high-rise RC-group rigid frame structures used as apartment houses etc. While the latest joint design method failed to explain the cause of such damages, the problem of a possible strength reduction due to the eccentricity of beam-column joints and scale effects has been pointed out.

ii. Recent experimental studies and analyses have been shedding light on the quantitative influence of eccentricity and scale effects on the shear capacity reduction of beam-column joints. For example, if a 350mm wide beam is placed eccentric to a column of 700mm wide and deep, it is expected that the joint strength be reduced by 20% due to eccentricity and also by scale effects; the total reduction would be about 1/3 in this case.

iii. As a result of examination of beam-column joints in existing pre-1981 RC school buildings and apartment houses, it was revealed that most of the joints were eccentric, that the equivalent story shear coefficient at joint shear failure C_j were varied significantly, and that the C_{jts} values of the majority of the buildings were smaller than 0.4 when the eccentricity and scale effects were taken into account.

iv. The above findings point to the fact that because the beams and columns of post-1981 buildings were given higher flexural and shear capacity under the current seismic codes, the strength of beam-column joints has relatively been reduced, which as a result has become more damage prone. It is unlikely to be able to obtain an adequate safety level even when seismic considerations are given to the joints.

v. To go to the root of this problem, a national research project should be launched as soon as possible, and there is now a pressing need to establish relevant seismic codes on strength reductions induced by eccentric joints and scale effects. Note that it is the beam-column joints in RC structures only that have been discussed here, but joints in SRC structures [14)] and highway bridges [15)] have been reported to be more or less under the same condition.

REFERENCES

1. AIJ (Architectural Institute of Japan), “Recommendation to RC Structural Design after Hanshin-Awaji Earthquake Disaster – Cause of particularly notice damages and corresponding RC structural design detail –”, 1998
2. Technical Branch, Arai-Gumi Co. Ltd., “Technical Research Report No.8-Special Issue – Investigation Report on Seismic Damages Caused by the 1995 Hyogoken-Nanbu-Oki Earthquake”, 1995
3. S. SUZUKI，M. HIROSAWA, “Discussion on the earthquake damage to beam-column joint of steel reinforced concrete building – In case of the 11-storied apartment house which suffered from the 1995 Hyogo Ken-Nanbu earthquake–”, RESERCH REPORTS OF KOGAKUIN UNIVERSITY No.88, pp143~151, 2000
4. The Building Center of Japan, “Structural Code for Buildings”, 1997
5. AIJ, “AIJ Standard for Structural Calculation of Reinforced Concrete Structures”, 1996
6. AIJ, “Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept (Draft)”, 1997
7. AIJ, “Ultimate Strength and Deformation Capacity of Buildings in Seismic Design (1990)”, 1990
8. AIJ, “Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept”, 1990
9. JBDPA (The Japan Building Disaster Prevention Association), “Standard for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings (2001 Revised Edition)”, 2001
10. Japan Association for Building Research Promotion and Structural Research Consulting Association, “Technical Manual for Seismic Diagnosis and Seismic Retrofit Design of Existing Buildings (2000 edition)”, 2000
11. S. FUJII, T. AKADA, K. ISHIDA, K. SHIMA et al. ”An Experimental Study on Seismic Capacity of RC Actual-Size Beam-Column Joints (part1~3)”, SUMMARIES OF TECHNICAL PAPERS OF ANNUAL MEETING AIJ, pp189~194, 2001
12. R. KUROSAWA, T. KOMURO, M. HIROSAWA, “Verification of Seismic Performance of Beam-Column Joints of RC and SRC Building Structures (Discussion on Scale Effect to Shear Capacity Joint Panel)”, SUMMARIES OF TECHNICAL PAPERS OF ANNUAL MEETING AIJ, pp197~198, 2001
13. T. OSADA, “Seismic Performance of Beam-Column Joint of RC system Apartment Houses and Influence by Arrangement of Shear Walls”, Master’s thesis of Kogakuin University, 2003
14. T. Komuro, S. SUZUKI, M. HIROSAWA, “Study on Seismic Performance of Beam-Column Joints of Existing RC & SRC Mid-to-high-rise Apartment Houses”, Proceeding of the Japan Concrete Institute Vol.23 No.3，pp421~426，2001
15. S. OHWA，M. HIROSAWA, “Study on Seismic Behavior of Beam-Column Joints in Bridge Structure of Reinforced Concrete“, RESERCH REPORTS OF KOGAKUIN UNIVERSITY No.89, pp181~188, 2000