Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.IV)

6-6-6

DESIGN CRITERIA FOR REINFORCED CONCRETE INTERIOR BEAM-COLUMN CONNECTIONS

Hiroyuki AOYAMA[1], Shunsuke OTANI[1], and Kazuhiro KITAYAMA[2]

[1]Department of Architecture, University of Tokyo,
Bunkyo-ku, Tokyo, Japan
[2]Department of Architecture, Utsunomiya University,
Utsunomiya-shi, Tochigi, Japan

SUMMARY

A set of earthquake resistant design criteria are proposed for reinforced concrete interior beam-column joints taking into account the acceptable deformation limits of a frame structure. Some bond deterioration along the beam reinforcement within a joint is permitted, the criteria of which were determined on the basis of nonlinear earthquake response analysis of buildings exhibiting good and poor hysteresis energy dissipation. The joint shear stress is assumed to be transferred by concrete diagonal compression strut mechanism, and its level is restricted in proportion to concrete compressive strength. The role of the lateral reinforcement is considered to confine the joint rather than to resist shear.

INTRODUCTION

A reinforced concrete (R/C) building in Japan has traditionally been designed for a large earthquake load, which normally resulted in wide columns. Hence, the damage to the beam-column joint was scarcely observed in the past earthquakes, joint shear stresses being limited at a low level. Therefore, the design of R/C beam-column joints has not been required. However, the advancement of the design calculation and the use of higher strength materials might may reduce the column dimensions, especially by the adoption of an ultimate strength design procedure relying on the ductility. Then beam-column joints may become a weak link of the chain, and design provisions may become necessary for R/C beam-column joints in Japan.

A beam-column joint, in principle, should not fail during a strong earthquake because (a) the gravity load must be sustained in the joint, (b) a large ductility and energy dissipation can not be expected in the joint, and (c) a joint is difficult to repair after an earthquake. Then the shear failure of a joint and at the same time, significant slippage of beam bars within a joint should be prevented up to a usable limit of structural deformation, which this paper arbitrarily defines as a beam ductility of 4.0 or a story drift angle of 1/50, whichever is smaller.

SHEAR MECHANISMS IN BEAM-COLUMN JOINT

Shear transfer mechanisms in a joint have been described by Paulay et al.(Ref.1) as shown in Figs.1.a and 1.b. These are called "main strut mechanism"

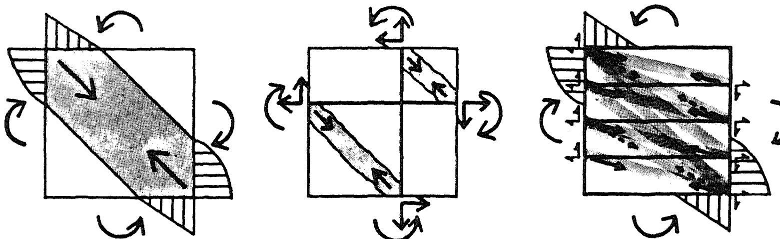


Fig. 1 Shear Transfer Mechanisms in Joint

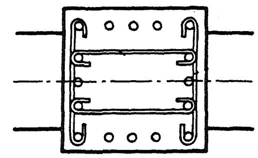


Fig. 2 Joint Reinforcement

Table 1 Properties of Specimen B1

Beam (300x200 mm)		Column (300x300 mm)		Joint	
Top Bars	8-D13	Total Bars	16-D16		
Bot.Bars	8-D13	Load(tonf)	18.0		
f_y(kgf/cm^2)	3780	f_y(kgf/cm^2)	3580	Hoops	2-R6
Stirrups	2-R6	Hoops	4-R6	sets	4
@(cm)	5.0	@(cm)	5.0	p_w(%)	0.35
f_y(kgf/cm^2)	4980	f_y(kgf/cm^2)	4980	f_y(<-)	2400

Note R6 : Round bar with the nominal diameter of 6 mm
f_y : Yield stress of the reinforcing bar
p_w : Lateral reinforcement ratio in a joint

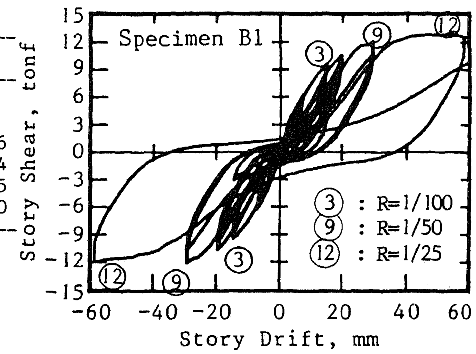


Fig. 3 Story Shear-Drift Relation

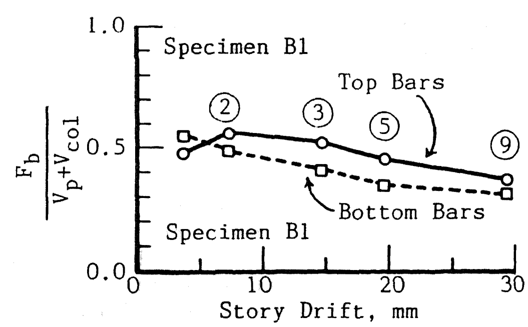


Fig. 4 Contribution of Bond Forces to Joint Shear

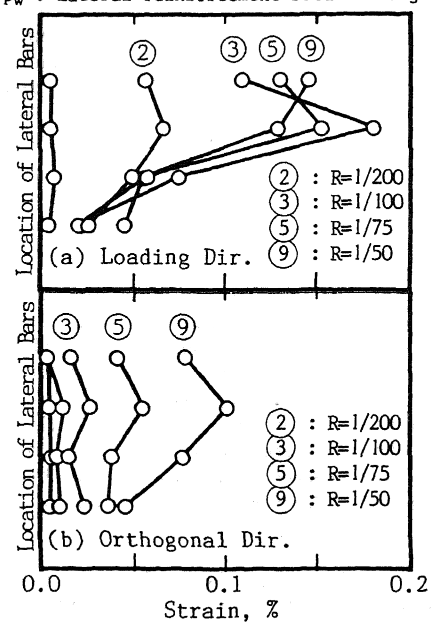


Fig. 5 Strains in Joint Lateral Reinforcement

Table 2 Properties of Sub-structures

	Height (m)	Fundamental Period(sec.)	Base Shear Coefficient
4-story	14.0	0.28	0.30
7-story	25.0	0.50	0.28
16-story	56.0	1.12	0.22

and "sub-strut mechanism" in this paper. The main compression strut is formed along the main diagonal of the joint panel as the resultant of the horizontal and vertical compression stresses acting at the beam and column critical sections. Note that the main strut exists irrespective of the bond characteristics of beam bars within the joint. The sub-strut mechanism requires good bond along the beam and column bars, formed with diagonal compression stresses distributed uniformly within the panel region. The diagonal strut stresses must balance with the tensile stress in the vertical and horizontal reinforcement and the bond stresses acting along the beam and column exterior bars. There may be another transfer mechanism ("truss mechanism") as shown in Fig.1.c. This is a truss mechanism formed by the lateral reinforcement, diagonal concrete struts and the column exterior reinforcement, and without the contribution of column intermediate reinforcement.

Note that the sub-strut mechanism is developed only when a good bond stress transfer is maintained along the beam and column reinforcement. However, the bond deterioration along the beam reinforcement is inevitable, especially after beam flexural yielding. With a bond deterioration along the beam reinforcement, the sub-strut mechanism starts to diminish. Consequently, the main strut mechanism carries the dominant part of joint shear, increasing the magnitude of compression stresses in the main strut. Because the strut concrete is weakened by the reversed cyclic loading and because the compressive strength is reduced by the increasing tensile strain perpendicular to the direction of the main strut, the shear capacity of the main strut decreases and eventually fails in shear compression. The principal role of the lateral reinforcement in this case is to confine the cracked joint core concrete. Note that the shear transfer mechanism in a beam-column joint changes with the bond deterioration along the beam bars.

ROLE OF JOINT LATERAL REINFORCEMENT

The test results of a half-scale plane beam-column joint (Specimen B1 tested at the University of Tokyo) are summarized to demonstrate some features of the shear transfer mechanisms of a joint. The properties of the specimen are listed in Table 1. The amount of the beam bars was made large to develop joint shear stress as high as 0.32 f_c' at beam yielding (f_c': concrete compressive strength of 250 kgf/cm^2), and to cause the bond stress along the beam reinforcement to be severe within a joint; the average bond stress u_b expressed by Eq.(1) was 80 kgf/cm^2. Legged ties shown in Fig.2 were used as the lateral reinforcement within a joint to discriminate between strains caused by carrying shear and those developed by the confinement of the core concrete.

The beam-column joint did not fail in shear up to the story drift angle of 1/50. The contribution of the joint shear deformation was 40 % of the total story drift, comparable to that of the beam deflections. Therefore, the beam yielding was delayed to a story drift angle of 1/50. The damage, however, concentrated on the joint panel region due to high shear after a story drift angle of 1/25. Story shear-drift relation is shown in Fig.3. The pinched hysteresis shape was caused by both the shear distress of the joint core concrete and the bond deterioration along beam bars.

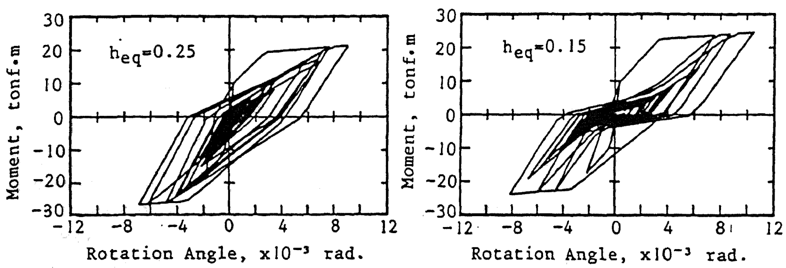
The ratio of the forces F_b transferred into the joint by the bond stresses along the beam reinforcement to the total joint shear V_p added to the story shear V_{col} is shown in Fig.4. The contribution of the bond forces to the joint shear decreased with the increase in the story drift, indicating that the bond along beam bars deteriorated gradually, and reached about 30 % of total joint shear plus story shear at a story drift angle of 1/50.

The strains in lateral reinforcement within the joint are shown in Fig.5 under such bond conditions of beam bars. Strains parallel to the loading direction became almost constant at story drift angle greater than 1/100, and did not reach the yield strain(0.2 %). Hence the contribution of the sub-strut mechanism to joint shear resistance decreased with bond deterioration along the beam reinforcement. On the other hand, strains orthogonal to the loading direction increased with the story drift, but the ties perpendicular to the loading direction did not yield up to the story drift angle of 1/50. Therefore, the amount of the lateral reinforcement provided in Specimen B1, i.e., 0.35 % is sufficient to confine the joint core concrete.

EFFECT OF BOND DETERIORATION ON RESPONSE

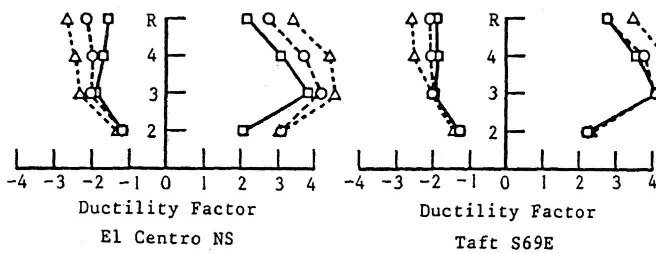
The bond deterioration of beam bars within a joint is not desirable because (a) the energy dissipation at beam ends is reduced by pinching in a hysteresis shape, (b) the diagonal compressive stresses increase with a change in joint shear transfer mechanism after beam yielding, and (c) the beam deformation increases due to the bar slip within a joint. The influence of the energy dissipation capability at the beam ends on earthquake responses is studied to discuss the permissibility of the beam bar slip within a joint.

The earthquake response analyses were carried out representing each member by a one-component model, in which inelastic rotational springs were placed at member ends. Beam-column joints were assumed to be rigid. The hysteresis models placed at beam ends were selected to simulate the pinching behavior caused by the bond deterioration along the beam reinforcement (Takeda-Slip hysteresis model). Takeda model was used to simulate a good bond situation with a spindle-shape hysteresis shown in Fig.6. The skeleton curves of both models were common, but

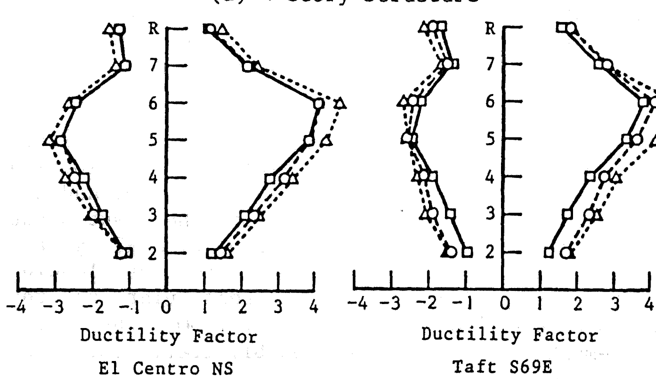


(a) Takeda Model (b) Takeda-Slip Model

Fig. 6 Hysteresis Models



(a) 4-Story Structure



(b) 7-Story Structure

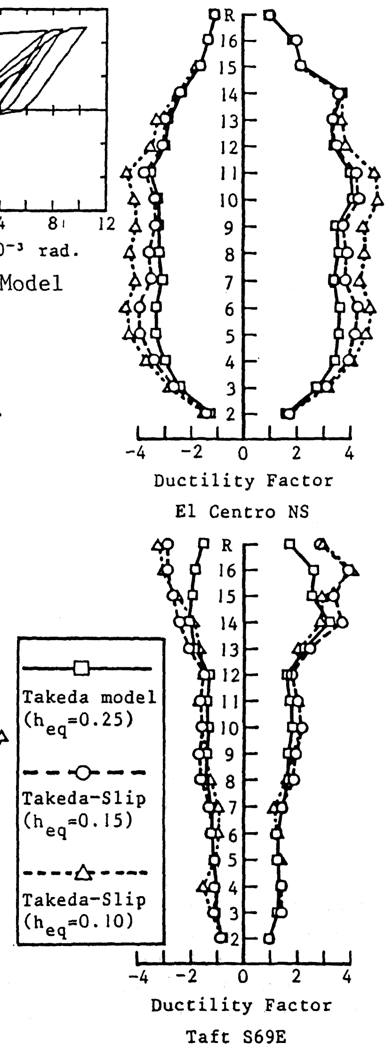


Fig. 7 Attained Ductility Factors at Beam Ends (c) 16-Story Structure

the values of parameters for the hysteresis shape were varied to study the effect of decrease in hysteretic area on the response; an equivalent viscous damping ratio h_{eq}, ratio of the dissipated energy within half a cycle to 2π times the strain energy at peak of an equivalent linearly elastic system, was 0.25 for a Takeda model, and 0.15 and 0.10 for Takeda-Slip models at a ductility factor of 4.0. The additional deformation caused by the pull-out of beam bars from a joint is not considered here.

Four-, seven- and sixteen-story sub-structures, an interior column with beams at the opposite column faces, were designed to form weak-beam strong-column frame structures. The properties of these sub-structures are listed in Table 2. Input earthquake motions were the 1940 El Centro (NS) record and the 1952 Taft (S69E) record. The intensities of ground motions were selected so that the maximum member ductility factors were approximately 4.0 at beam ends for the structures using the Takeda model.

The attained ductility factors at beam ends are shown in Fig.7. The distribution of beam-end ductilities of a structure with the Takeda model is similar to that with the Takeda-Slip model (h_{eq}=0.15). The change in the h_{eq} value of the Takeda-Slip model from 0.15 to 0.10 did not affect the ductility demand at beam ends. The attained maximum response drifts were comparable for the three structures, however the larger response drifts developed frequently in the structures with the Takeda-Slip models than that with the Takeda model. From the results of earthquake response analyses, the effect of hysteresis energy dissipating capacity on the response was found relatively small for a range of equivalent viscous damping ratio from 0.10 to 0.25 at ductility factor of 4.0. Therefore, some bond deterioration of beam bars within a joint may be tolerable.

LIMITATION OF BEAM BAR BOND INDEX

The average bond stress u_b over the column width for simultaneous yielding of the beam reinforcement in tension and compression at the two faces of the joint divided by the square root of the concrete compressive strength, called beam bar bond index, is used to indicate the possibility of bond degradation along the beam reinforcement;

$$u_b / \sqrt{f_c'} = f_y \ (d_b \ / \ h_c \ ) \ / \ 2 \ / \sqrt{f_c'} \qquad (1)$$

where f_y: yield strength of beam bars in kgf/cm^2, d_b: diameter of beam bars, h_c: column width and f_c': concrete compressive strength in kgf/cm^2.

The beam bar bond index $u_b/\sqrt{f_c'}$ and the equivalent viscous damping ratio h_{eq} at a story drift angle of 1/50 are compared for the plane beam-column joints tested previously in Japan in Fig.8. The solid line was derived from the least squares method to fit the data. Concrete compressive strength f_c' was greater than 270 kgf/cm^2 for specimens with open symbols. The h_{eq} values tend to decrease with an increasing $u_b/\sqrt{f_c'}$ value. If an allowable deformation level of R/C frame structures is taken to be a story drift angle of 1/50, the $u_b/\sqrt{f_c'}$ value should satisfy Eq.(2) to ensure the equivalent viscous damping ratio of 0.10, *as indicated in the earthquake* response analyses.

$$u_b / \sqrt{f_c'} \leq 4.5 \qquad (2)$$

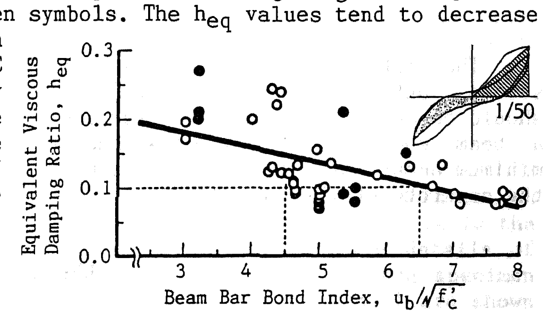


Fig. 8 Equivalent Viscous Damping Ratio-Beam Bar Bond Index Relation

Substituting u_b in Eq.(1) into Eq.(2), the following expression is obtained.

$$h_c \ / \ d_b \geq f_y \ / \ (9\sqrt{f_c'}) \tag{3}$$

LIMITATION OF INPUT SHEAR INTO JOINT

After some bond deterioration takes place along the beam reinforcement in a joint, the shear stress within a panel region is carried dominantly by the main strut mechanism. The shear compression failure in the main strut mechanism may be avoided by restricting the level of shear stress.

The joint lateral reinforcement ratio is compared with the value v_u/f_c' for plane beam-column joint specimens larger than the half-scale, tested in Japan and U.S., shown in Fig.9 in which v_u is the maximum joint shear stress observed in the tests. The effective joint area to resist shear is defined as the column depth multiplied by the average of the beam and column widths. The lateral reinforcement ratio was defined as the total cross-sectional area of lateral reinforcement within a joint divided by the column width and the distance of (7/8)d, d: effective depth of beam critical section. The figure indicates that the joint shear stress v_u must be limited as given in Eq. (4) to prevent shear failure after beam flexural yielding;

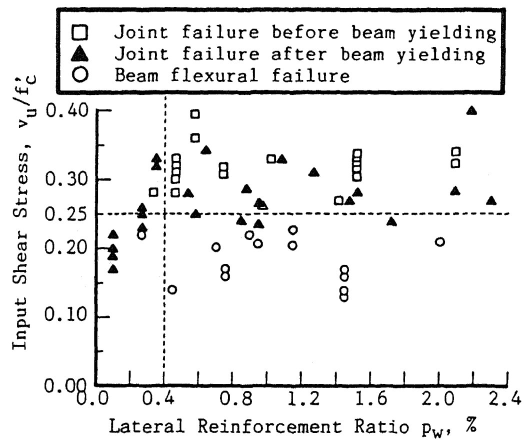


Fig. 9 Joint Lateral Reinforcement Ratio-Normalized Joint Shear Stress Relation

$$v_u \ / \ f_c' \leq 0.25 \tag{4}$$

The shear failure in a joint occurred at a story drift angle greater than 1/25 regardless of the amount of lateral reinforcement if the input shear stress v_u is greater than 0.25 f_c'. On the contrary, the lateral reinforcement ratio of 0.27 % was sufficient to prevent shear failure when v_u is less than 0.25 f_c'. From these test results and those of specimen B1 with lateral reinforcement of 0.35 %, a minimum lateral reinforcement ratio of 0.4 % is recommended. The required lateral reinforcement ratio of 0.4 % may be reduced if the joint shear stress is sufficiently lower than 0.25 f_c'. The lateral reinforcement within a joint is expected to confine the panel concrete.

CONCLUDING REMARKS

The ratio of the column width to the beam bar diameter and the joint shear stress must be limited by Eqs.(2) to (4), under the assumption that buildings should be designed for the permissible maximum story drift angle of 1/50, or for a beam ductility factor of 4.0 under the most severe earthquake motion. A minimum amount of lateral reinforcement must be placed within a joint to confine the concrete of the main strut.

REFERENCES

1. Paulay, T., Park, R. and Priestley, M. J. N., "Reinforced Concrete Beam-Column Joints Under Seismic Actions," ACI Journal Vol.75, 585-593, (1978).