REINFORCED CONCRETE BEAM-COLUMN JOINTS FOR SEISMIC LOADING

Part I - Theory

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ABSTRACT

The basic theory relating to the design of reinforced concrete beam-column joints is reviewed and extended. In particular, the theory of an alternative joint detail using bond plates is developed.

1. INTRODUCTION

Tests reported in the last decade have shown that beam-column joint zones are liable to suffer severe load and stiffness degradation when they are subjected to the load reversals typical of earthquake excitation[#]. To improve their performance under seismic loading design rules have been presented covering the proportioning and stirrup content of beam-column joint zones. As testing progressed and knowledge of the problems improved, the design rules were changed to require more joint steel and larger beams. Much of the present New Zealand practice leads to joint zones with very highly congested steel arrangements, causing practical problems with the placing of steel and concrete. The extra quantities of joint steel require beam depths to be approximately 30% greater than would have been used a few years ago.

The long term aim of the present work is to develop alternative details for beam-column joints which perform satisfactorily under seismic loading and are less restrictive in terms of joint steel requirements. This in turn should lead to reductions in member dimensions with attendant economic advantages.

The remainder of the paper is outlined as follows. Section 2 concerns basic mechanisms of shear resistance which, after reviewing shear resistance in beams, concentrates on the two important actions for joints, namely the panel truss and diagonal strut actions. Section 3 compares and evaluates current methods of design for joints and Section 4 is devoted to a description of an alternative joint detail using bond plates.

This paper and its subsequent companion (which reports experimental results) have been taken, with minor amendments, from a report⁽³⁾ of work carried out in 1976.

2. BASIC MECHANISMS OF SHEAR RESISTANCE

Before studying shear in joint zones, the basic mechanisms of shear resistance in beams will be reviewed.

2.1 Beams

Consider a portion of a reinforced concrete beam, such as that shown in Figure 1. Because it is subjected to a shear force a bending moment gradient exists along it. For all practical purposes the internal lever arm jd will be constant in this portion. Consequently, as the tension force in the flexural reinforcement is given by

$$T = M/jd, \quad (1)$$

it follows that the tension force has a gradient similar to that of the bending moment diagram. The change in tension force ΔT over a length Δx (see Figure 1(b)), generates a shear stress v_n in the concrete just above the reinforcement of

$$v_n = \Delta T/b\Delta x, \quad (2)$$

where b is the width of the beam.

In regions of the beam away from the support and load points, the resistance to the shear stresses induced by the tension force gradient is provided by two principal actions. The first of these is beam action, which is due to the flexural resistance of the concrete block between the cracks and the shear forces transmitted across them by dowel action of the flexural reinforcement and aggregate interlock⁽⁴⁾ (see Figure 1(c)). The shear resistance of beam action is $v_{pb}\Delta x$. The second contribution arises from the forces induced by the web reinforcement, where the stirrup force S (shown in Figure 1(c)), together with that part of the bond force resisted by the action of the web reinforcement ΔT_s , combine to develop a diagonal compressive force D in the concrete. To sustain each other these three forces must meet at a point. For this condition to be met the web reinforcement must be anchored around the flexural tension steel.

Longitudinal steel which is parallel to and above the flexural tension steel does not contribute significantly to the shear strength of the beam.

The shear resistance of beams loaded close to a support point arises from a different action. In this case, as is illustrated in Figure 2, a diagonal strut

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See references 3-8 in reference 1 and references 13-37, 39-44 in reference 2.

may form from the load point to the support reaction. The resulting structural action is similar to that of a bow string arch. The flexural tension force, equivalent to the tension chord of the arch, is constant between support points. Failure typically occurs at either anchorage of the tension reinforcement, or by failure of the concrete in the arch. In this situation the shear resistance of the concrete is considerably greater than when the beam is loaded away from the supports. This mechanism of shear resistance is referred to as arch action.

2.2 Beam-Column Joints

Several different mechanisms of shear resistance have been identified in reinforced concrete beam-column joints^(5,2). The first is the shear resisted by the concrete, the second is panel truss action (similar to the action of web reinforcement in beams), and the third is diagonal strut action (which is similar to arch action in beams).

The contribution of the concrete to the shear strength is certainly appreciable in joints subjected to static loading. However, results of joints tested with repeated load reversals indicate that this component decreases, and some investigators have concluded that it should be neglected in design (1,2,5).

The panel truss and the diagonal strut actions are illustrated in Figure 3. In the first inelastic half-cycle of loading the tension steel will yield in the beams at sections adjacent to the joint (i.e., at a-a and b-b in Figure 3(a)). When the load reverses the compression force attempts to close the yield cracks opened in the previous half of the cycle. Until the cracks do close the compression force will be carried by the reinforcement. Consequently, after the first half-cycle very high bond stresses try to develop through the joint with, for example, the top bars in the beam being at yield in tension on one side and at a stress f'_s in compression on the other side, where

$$f'_s = \frac{A_{sb}}{A_{st}} f_y \neq f_y,$$

in which A_{sb} is the area of flexural steel in the bottom of the beam and A_{st} the area of steel in the top of the beam.

With panel truss action a bond gradient must be sustained through the joint. However, no bond is required in the joint zone with the diagonal strut action, although the bars must be anchored on the far side from the flexural tension force.

The shear force acting on the joint (section 1-1 in Figure 3(a)) is typical) arises from the flexural forces in the top of the beam $T_1 + C_2$ minus the shear in the column above the joint V_3 . The greatest component comes from the beam flexural forces which, after the first inelastic cycle, act predominantly in the reinforcement and, consequently, the shear from this source is provided largely by bond between the flexural steel and the concrete in the joint zone. The joint zone shear V_j is given by

$$V_j = T_1 + C_2 - V_3$$

Similar expressions exist for the other three sections.

2.2.1 Panel Truss Action

The panel truss supplies the following resisting actions to the shear V_j acting on section 1-1:

(i) Diagonal compression forces acting between the cracks (i.e., D_1 in Figure 3(b)); and

(ii) Vertical forces applied against the beam flexural reinforcement, which arise from the axial load on the column and the forces in the vertical stirrup reinforcement (i.e., P and S_v in Figure 3(b)).

Because equilibrium must be satisfied a polygon of forces may be drawn as shown in Figure 4. There T_1 and C_2 are the flexural tension and compression forces in the top of the beam at the joint, V_3 is the column shear above the joint, ΣS_v is the force supplied by the vertical stirrups, P_u is the axial load on the column (equal to $C_3 - T_3$ which, as is evident from Figure 3, is the difference between the forces in the main steel of the column above the joint) and ΣD_1 is that portion of the diagonal compression forces in the joint which apply a lateral component to the beam flexural forces (T_1 and C_2).

The direction of ΣD_1 is fixed by the angle of the cracks which have been observed to develop parallel to the joint diagonal. Thus θ , which is the angle between ΣD_1 and $T_1 + C_2$, is given by

$$\theta = \tan^{-1} \left(\frac{h_b}{h_c} \right),$$

where h_b and h_c are the distances between the flexural reinforcement in the beam and column, respectively (see Figure 3(a)).

For full shear coverage by panel truss action the area of vertical stirrups ΣA_v (with yield f'_y) is given by

$$\Sigma A_v f'_y = \left\{ (T_1 + C_2) - V_3 \right\} \frac{h_b}{h_c} - P_u,$$

For the steel one may allow a "blanket" factor α which covers strain hardening and basic overstrength (a typical value is $\alpha = 1.25$ which includes an estimate of the compensation resulting from the stirrups). In addition a capacity reduction factor ϕ may be specified (for shear it is 0.85). The vertical stirrup requirement is, therefore

$$\Sigma A_v f'_y = \frac{\alpha}{\phi} \left\{ (A_{st} + A_{sb}) f_y - V_3 \right\} \frac{h_b}{h_c} - P_u, \quad (3)$$

where, as before, A_{st} and A_{sb} are the areas of flexural steel in the top and bottom of the beam respectively.

Increasing the axial load on the column should increase the shear resisted by the truss action, provided that adequate horizontal confining steel is specified. The axial load also considerably improves the bond perform-

ance of the reinforcement⁽⁶⁾, although the deleterious effects of axial tension, which is possible in outer columns of frames under earthquake attack, should not be overlooked.

Horizontal stirrups are required to complete the truss action. The area of these stirrups is

$$\Sigma A_h f_y = \frac{\alpha}{\phi} \left\{ (A_{st} + A_{sb}) f_y - V_3 \right\} . \quad (4)$$

For satisfactory panel truss action both horizontal and vertical stirrups are necessary in the joint, and they must be anchored around the relevant flexural steel. All existing design methods overlook this and are, therefore, deficient. Nevertheless, since it is becoming more and more difficult to place the horizontal stirrups in the joint, requirements for vertical stirrups will hardly help that matter.

Column bars located between the main column flexural steel in the joint may act in lieu of vertical stirrups. However, they are unlikely to be as efficient as stirrups because they do not bend around and bear against the beam steel, thereby provided a clamping force which improves bond. Nevertheless, some experimental work⁽⁵⁾ has shown that intermediate column bars improve performance.

2.2.2 Diagonal Strut Action

This mechanism is similar to the shear resistance associated with arch action in a beam loaded close to its supports (see Figure 2), and is illustrated in Figure 3(c).

The flexural compression forces at the corner of a joint, together with the flexural tension forces from the members on the other side, are balanced by an inclined compression force which develops across the joint diagonal. Thus, with reference to Figure 3(c), the flexural tension forces T_1 , T_2 , T_3 and T_4 are anchored on the opposite side from where they enter the joint. For example, the force T_2 is anchored at or near point A. No bond stresses develop between point B and the zone where the column compression force C_4 bears against the bar but, at this location, high bond stresses may develop because the bar is imbedded in concrete which is subject to compression. Under load reversal the tension force is now anchored on the other side of the joint at B. Yielding of the beam bars, due to plastic hinging at the column face, causes the steel between A and B to grow in length. However, because of the presence of stirrups in the column and the joint, the column cannot expand appreciably. Consequently, the extension can only be taken up by slip of the bar through the joint.

With the high cyclic bond stresses associated with diagonal strut action, the bond capacity of the concrete is progressively destroyed causing the anchorage points of the bars to drop back further into the beams. The inelastic extension and consequent slip of the bars does not directly affect strength in the initial load cycles, but it does cause a major reduction in joint stiffness. With a large number of load reversals well into the inelastic range, and the breaking up of the concrete due to alternate opening and

closing of wide cracks across the joint diagonals, the ability of the concrete to sustain the diagonal compression force may be severely curtailed, leading to an eventual reduction in load capacity of the joint.

Theoretically, diagonal strut action requires no stirrups for shear resistance. However, some steel is required in the joint zone to control the width of the diagonal cracks. The diagonal compression force creates a splitting force perpendicular to it in very much the same way as splitting is generated in the split cylinder, or cube test (see Figure 5).

The splitting force P_t generated by a line load acting across the diameter of an elastic cylinder (or cube) is given by

$$P_t = 2D/\pi , \quad (5)$$

where D is the diagonal compression force. If the angle of inclination is 45° then, ignoring the small influence of the column shear,

$$D = \sqrt{2} \left(T_1 + C_2 \right) .$$

Now equation (5) may be modified to allow for the distribution of the diagonal compression force over a width a . From Figure 5 it is seen that the dispersion of the diagonal force into the joint generates a bending moment in a free body consisting of one quarter of the joint which, from studies, can be shown to be proportional to $(1 - a/d)$. As a first approximation P_t will be proportional to this bending moment and, including this allowance, the equation for P_t becomes

$$P_t = 2D(1 - a/d)/\pi ,$$

and, assuming $a \approx 0.2d$, finally

$$P_t \approx 0.5D . \quad (6)$$

If an orthogonal mesh of reinforcement is used in the joint with horizontal and vertical steel areas of A_h and A_v , respectively, then the requirements are given by

$$\left(A_v + A_h \right) f'_y \approx \frac{\sqrt{2}D}{2} \approx \left(T_1 + C_2 \right) ,$$

or

$$\left(A_v + A_h \right) f'_y \approx \left(A_{st} + A_{sb} \right) f_y , \quad (7)$$

where f'_y is the yield stress of the stirrups and f_y is the corresponding value for the flexural reinforcement.

Equation (7) is limited to joints that are approximately square and is based on an elastic analysis of the joint. Once diagonal cracks form the tension stiffness across the diagonal decreases and, consequently, the force necessary to hold the two sides together must be less than that given above. The steel area specified by equation (7) is too high, probably by at least a factor of two, so that, provided the steel is evenly dispersed (to give adequate crack control), a more realistic assessment of joint steel necessary to control the splitting associated with diagonal strut action is

$$\left(A_v + A_h \right) f'_y = \gamma \left(A_{st} + A_{sb} \right) f_y \quad (8)$$

where γ is likely to be substantially less than unity.

3. CURRENT METHODS FOR DESIGN OF JOINT ZONES

Several different proposals have been made for proportioning beam-column joints. The only relevant code covering their design is that of ACI 318-71. However, test programmes completed in the last few years have indicated that these provisions do not ensure adequate performance^(2,5). In Table 1 (see Figure 6 for a definition of symbols used) the provisions of the ACI code are compared with other proposals. These include the tentative recommendations of May 1976 for the revision of the N.Z. Ministry of Works and Development code of practice for the design of public buildings (PW 81/10/1), and the recommendations of the ACI-ASCE Committee 352 contained in reference 1.

Stirrup reinforcement is required in joint zones for two reasons. The first is to increase the shear strength, and the second is to provide confinement of the column (although this is necessary only if the ultimate axial load exceeds 0.4 of the axial load under balanced conditions by the ACI-318-71 code). To simplify the comparison, Table 1 is restricted to two dimensional beam-column joints with column loads less than 0.4 of the balanced axial loads.

The various methods follow the same basic approach. The joint shear is calculated from the flexural forces in the beam and the column shear. In most cases allowance is made for the strength of the beam steel being greater than the specified minimum and for work-hardening of the steel. The concrete is assumed to contribute to the shear strength in the ACI code and Committee 352 recommendations, but this contribution, if any, is neglected in the other proposals when a condition of low axial load applies. The remainder of the shear is carried by stirrups placed around the column bars. Two of the proposals require the main column steel to be distributed right around the section to assist with confinement and shear. However, there are no vertical stirrups specified in terms of joint shears. Thus, the provisions summarized in Table 1 cannot be interpreted in terms of panel truss action (see section 2.2.1).

The problem of high bond stresses through the joint zone is recognised in the MWD proposals and limits are placed on bar diameters in an attempt to control bond failure.

Existing joint design procedures are capable of considerable improvement because, by and large, they fail to recognise adequately the means by which forces must be transferred through the joint. Design procedures based on panel truss and diagonal strut actions similar to those described in the present paper have recently been proposed by the Frame Study Group of the N.Z.N.S.E.E. Joint shears are resisted principally by a combination of panel truss action and diagonal strut action. However, in conventional

joints the absence of vertical stirrups means that the panel truss may be incomplete, and insufficient anchorage is available for strut action so that excessive stiffness degradation may result.

In the next section a joint concept is outlined which relies almost solely on diagonal strut action.

4. BOND PLATE CONCEPT

The basic idea behind the bond plate detail is that all the joint shear is transmitted by diagonal strut action. No bond transfer is assumed for the main flexural steel in the joint zone and, as a result, the complete change in force in the bar, from compression on one side to tension on the other, is taken out on bond plates which are fixed to the flexural steel on the perimeter of the joint, and which sustain the diagonal compression force on the concrete (see Figure 3(c)). This concept will be discussed in greater detail in a later paper, together with test results.

Since diagonal strut action is the principal method of load transfer in the joint, joint shear reinforcement requirements are greatly reduced with some steel necessary to control the splitting effects associated with the diagonal compression force (see equation (8)).

5. CONCLUSIONS

The basic mechanisms of shear resistance in joints have been discussed and formulae derived to account for the joint steel requirements associated with both panel truss action and diagonal strut action. A review of existing joint design methods showed that none could be interpreted rationally in terms of either panel truss action or diagonal strut action.

A concept for an alternative detail using bond plates was briefly described; the method of load transfer being almost solely due to diagonal strut action. A subsequent paper will report on experiments which indicated the substantial improvement in ductile performance that the bond plate detail provided over that obtained from conventional joints.

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7. REFERENCES

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TABLE 1
DESIGN METHODS FOR BEAM-COLUMN JOINTS IN 2D-FRAMES SUBJECTED TO SEISMIC LOADING
(UNDER LOW AXIAL LOADS)

Action \ Method	ACI-318-71	Recommendations of ACI-ASCE Committee ¹ 352	Draft Revision PW 81/10/1 - May 1976	Park and Paulay ² 1975
Calculation of Joint Shear Force V_j	$V_j = \alpha f_y (A_{st} + A_{sb}) - V_{col}$ $\alpha = 1.0$	$V_j = \alpha f_y (A_{st} + A_{sb}) - V_{col}$ $\alpha = 1.25$	$V_j = \alpha (f_y (A_{st} + A_{sb}) - V_{col})$ $\alpha = 1.20$	$V_j = \alpha (f_y (A_{st} + A_{sb}) - V_{col})$ $\alpha = 1.25$
Ult. shear stress in joint " v_u " $\phi = 0.85$	$v_u = \frac{V_j}{\phi b d_c}$	$v_u = \frac{V_j}{\phi b' d_c}$	$v_u = \frac{V_j}{\phi b d_c}$	$v_u = \frac{V_j}{\phi b d_c}$
Shear stress carried by concrete v_c	$v_c = 0.166 \sqrt{f'_c} \left(1.0 + 0.0725 \frac{N_u}{A_g} \right)$ $\nless 0.291 \sqrt{f'_c} \sqrt{1.0 + 0.29 \frac{N_u}{A_g}}$	$v_c = 0.291 \sqrt{f'_c} (1 + 0.29 \frac{N_u}{A_g})$ N_u is positive for axial compression $v_c = 0$ for axial tension	$v_c = 0$	$v_c = 0$ for $N_u \leq 0.12 f'_c A_g$ For $N_u > 0.12 f'_c A_g$ there are no definite recommendations. See p. 746-756, ref. 2.
Shear carried by stirrups parallel to flexural steel A_v^*	$v_s = v_u - v_c$ $\frac{A_v}{s} = \frac{v_s b}{f_y}$	$v_s = v_u - v_c \nless v_u/3$ $\frac{A_v}{s} = \frac{v_s b'}{f_y}$	$V_s = V_j / \phi$ $\frac{A_v}{s} = \frac{V_s}{d - d'}$	$\frac{A_v}{s} = \frac{V_s}{(d - d')}$ interior cols. $\frac{A_v}{s} = \frac{1.5 V_s}{(d - d')}$ exterior cols. $v_s = \frac{V_j}{\phi} - v_c \frac{b d_c}{b d_c}$
Max. permissible shear on joint	No limit stated	$v_s \nless 1.25 \sqrt{f'_c}$ $v_u \nless 1.66 \sqrt{f'_c}$	$v_u \nless 1.41 \sqrt{f'_c}$	Further research required to establish limit
Bond for flexural steel	No limit stated	No limit stated	$f_y = 275$ Beams $d_i < D_1/25$ Cols. $d_i < D_2/25$ $f_y = 380$ Cols. $d_i < D_2/35$	No limit stated
Vertical Reinforcement or stirrups in joint	None required	None required	None specified. Vert. col. bars should be distributed round perimeter to take vertical shear component	None specified. Comment given as for PW 81/10/1 May 1976

NOTES: * A_v in this table conforms to ACI usage and refers to joint shear steel placed parallel to beam flexural steel. In the rest of this paper A_v refers to stirrups perpendicular to beam flexural steel and A_h refers to stirrups parallel to beam steel. Requirements for stirrup confinement of concrete through joint zone are not included. See Figure 6 for definition of symbols.

(a) REINFORCED CONCRETE BEAM SUBJECTED TO MOMENT AND SHEAR.

(b) BENDING MOMENT AND TENSION FORCE DIAGRAM FOR REINFORCEMENT.

(i) BEAM ACTION (ii) STIRRUP ACTION (iii) POLYGON OF FORCES
(c) BEAM AND WEB REINFORCEMENT ACTIONS IN A BEAM

(a) CRACK PATTERN AND JOINT FORCES.

(b) PANEL TRUSS ACTION IN JOINTS.

(c) DIAGONAL STRUT ACTION IN JOINT.

FIGURE 1:
MECHANISMS OF SHEAR RESISTANCE
IN REINFORCED CONCRETE BEAMS.

FIGURE 3:
MECHANISMS OF SHEAR RESISTANCE
IN BEAM-COLUMN JOINTS.

FIGURE 2: ARCH ACTION IN A REINFORCED CONCRETE BEAM.

FIGURE 4: PANEL TRUSS ACTION POLYGON OF FORCES FOR TOP OF JOINT

FIGURE 6: DIMENSIONS USED TO DESIGN JOINT SHEAR REINFORCEMENT.

FIGURE 5:
SPLITTING FORCE IN A REINFORCED
CONCRETE JOINT UNDER DIAGONAL
STRUT ACTION.