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

6-6-2

ANCHORAGE REQUIREMENTS IN INTERIOR R.C. BEAM-COLUMN JOINTS

Leon, Roberto T.

Department of Civil and Mineral Engineering
University of Minnesota

SUMMARY

Current design codes vary widely in their requirements for anchorage of beam bars in interior reinforced concrete beam-column joints. To study the effect of anchorage length on beam-column joint performance four specimens with anchorage lengths of 16, 20, 24, and 28 bar diameters (d_b) were tested. The study showed that a minimum anchorage of $24d_b$ is required for the beam to reach its ultimate strength and that $28d_b$ are necessary to insure adequate hysteretic behavior.

INTRODUCTION

The problem of bond deterioration in interior reinforced concrete beam-column joints has been of interest to engineers since the introduction of the ductile moment-resisting frame concept. For design purposes the bond deterioration problem is addressed by specifying a minimum anchorage length through the joint, effectively setting a minimum column width for the structure. The anchorage requirements are usually given in terms of the number of beam bar diameters (d_b) and not as a bond stress value. The anchorage lengths suggested for design using Grade 60 (410 MPa) reinforcing bars range from $20d_b$ in the U.S. to $36d_b$ in New Zealand. The wide discrepancies stem primarily from the amount of damage permitted in the joint region and on the amount of slip permitted (or not permitted) in the beam bars crossing the joint. These, in turn, are associated with the assumption of either a strut or a panel truss mechanism as the main shear force transfer mechanism in the joint.

In most codes the requirements for minimum anchorage (l_{db}) and maximum shear stress (v_j) in the joint are treated independently (Ref. 1). This design simplification is reasonable as long as the large values of l_{db} and low values of v_j are utilized. Current U.S. specifications allow shear stresses of $20\sqrt{f'_c}$ and anchorages of 20 bar diameters (Ref. 2) that will result in significant bond deterioration with cycling. Increasing the anchorage lengths and decreasing the allowable joint shear stresses, on the other hand, will lead to very large columns and uneconomical designs.

To study the effects of anchorage length on joint performance, an experimental study has recently been completed on four half-scale interior beam-column connections (Ref. 3). The main variable in the experiments was the column depth, ranging from 16 to 28 bar diameters. The specimens were extensively instrumented to monitor the shear strain in the joint panel zone and the breakdown of bond and slip in the reinforcing bars.

FORCE TRANSFER MECHANISM

The shear force generated by the lateral displacement of a moment-resisting frame has to be transferred to the joint by a combination of bearing and bond stresses. If the anchorage lengths are long, large cracks will form due to yielding at the beam-column interface, and thus the bearing force will disappear with cycling. If the anchorage lengths are small, the bond stresses initially will be high. Upon cycling, however, significant bond deterioration will occur and the force transfer capability by bond will decrease and disappear when the bar pulls through.

Assuming a steel overstrength factor of 1.25 and that the shear in the column is about 10% of the forces in the beam bars, the joint shear stress (β_j) normalized to the $\sqrt{f'_c}$ can be calculated as:

$$\beta_j = \frac{0.883\ [N_t(d_b^+)^2 + N_b(d_b^-)^2]}{b_c\ d_c} \quad \frac{F_y}{\sqrt{f'_c}} \qquad \text{(Eq. 1)}$$

where N_t and N_b are the number of top and bottom bars, d_b^+ and d_b^- refer to the diameter of the top and bottom bars, F_y is the yield strength, and b_c and d_c are the depth and width of the column. For the case of a square column ($b_c = d_c = l_{db}$), beams with four #8 (25 mm) bars top and three #6 (19 mm) bars bottom steel, beam bar anchorage length of 20 times the maximum bar diameter, F_y of 60 ksi (410 MPa), and f'_c of 4 ksi (28 Mpa), the joint shear stress would be $12.0\sqrt{f'_c}$.

In the design of ductile moment-resisting frames, the joints are typically the last element to be designed. The areas of steel in the beams (the term contained within square brackets in Eq. 1), the material strengths (F_y and f'_c), and the size of the column (b_c and d_c) have already been determined, while the allowable joint shear strain (β_j) is dictated by the specifications. Thus the designer is left to check Eq. 1 for the case where the equal sign (=) is replaced by a less than or equal (<) sign. If this check fails the only option is to increase the size of the column, leading to substantial redesign time.

Equation 1 shows that if b_c is replaced by l_{db} the shear and bond stresses are related by the bar diameter. Thus the larger the anchorage length is, the lower the joint shear stresses. If the designer wants to optimize the design and use the minimum anchorage length and the maximum shear stress, Equation 1 can be reworked for any combination of material properties to give a maximum ratio of total steel in the beam (A_{st}) divided by the depth of the column:

$$\alpha_j = \frac{[N_t d_b^{+2} + N_b d_b^{-2}]}{d_c} = \frac{\beta_j\ (\sqrt{f'_c})\ (l_{db})}{F_y} = \frac{A_{st}}{d_c} \qquad \text{(Eq. 2)}$$

For the case of $\beta_j = 20\ \sqrt{f'_c}$, $l_{db} = 20\ d_b$, $f'_c = 4$ ksi (28 MPa), $F_y = 60$ ksi (410 MPa) and #8 bars (1 in or 25 mm), the limit this ratio will be $\alpha_j = 0.421$. The designer will save time by computing Eq. 2 as soon as the beams have been designed by assuming the limit values for anchorage and shear stress, and a reasonable maximum bar diameter. This will allow the designer to compute a value for the width of the column; if this value appears reasonable (less than or equal to 20 d_b, for example) then the design can proceed without problems.

While the current specifications set limits for the allowable joint shear stresses and the minimum anchorage lengths, utilizing these minimum values will probably result in significant damage and softening with cycling. This research intends to establish a better set of limits based on linking the maximum shear stress with the minimum anchorage length. This will lead not only to better joint performance but also to time savings in the design process.

EXPERIMENTAL STUDY

The four specimens tested were labelled BCJ1 (column depth of 16 d_b), BCJ2 (20d_b), BCJ3 (24d_b), and BCJ4 (28d_b). All specimens had column widths of 10 in (254 mm), and 12 in by 8 in (305 mm by 203 mm) beams. The reinforcement in the columns was changed from specimen to specimen to maintain a reasonable ratio of column overstrength, but the beam reinforcement was maintained at four #4 (13 mm) bars at the top and four #3 (9.5 mm) bars at the bottom. All the joints were reinforced with four #2 (6 mm) closed ties. This gave a volumetric reinforcement ratio of 83% of that required by the code, and thus the joints were slightly under-reinforced.

Assuming nominal material properties (i.e., no understrength factors) and gross areas, the joint shear stresses in the specimens would range from 10.5 to 18.3 $\sqrt{f'_c}$. Utilizing current American specifications (3), the design joint shear stress values were 11.5 $\sqrt{f'_c}$ for BCJ4, 13.4 $\sqrt{f'_c}$ for BCJ3, 15.6 $\sqrt{f'_c}$ for BCJ2, and 18.1 $\sqrt{f'_c}$ for BCJ1. These values were well below the 20 $\sqrt{f'_c}$ currently allowed, and thus shear should not be a major factor in the joint performance.

The specimens were instrumented to monitor all beam and column deformations and rotations, as well as the joint shear strain, and the slip of two of the top bars and one of the bottom bars. One of the bars was extensively instrumented with strain gages to obtain the strain profiles in the beams and across the column.

The tests were carried out by slowly displacing the column by 0.1 in (2.5 mm) for two full reversal cycles, 0.25 in (6.2 mm) for two more cycles and then at increments of 0.25 (6.2 mm) until yield was reached. The specimen was then cycled for two cycles at displacement ductilities of 1.0, 1.5, 2.0, 2.5, 3.00 and then to the ultimate displacement allowed by the test setup.

TEST RESULTS

Only the performance of the specimens with anchorages of 20, 24 and 28 bar diameters was deemed satisfactory for design, as BCJ1 (l_{db} = 16 d_b) did not reach the yield capacity of the beams due to shear cracking in the joint area and inelastic bending in the column due to poor anchorage of the column bars. This occurred in spite of the fact that the column was designed with a 1.4 flexural overstrength factor over the nominal beam capacities.

Figure 1 shows the column loads vs. joint shear strain for BCJ2. The very fast increase in shear strain after a load of 10 kips (4.45 kN) was reached is indicative of the shear cracking damage to the joint. Yielding of the beams had been predicted at a column load of 12.8 kips (56.9 kN) based on conventional analysis and nominal material properties. The ultimate capacity had been estimated at 15.1 kips (67.2 kN) of column load, based on confined section and non-linear material properties. BCJ2 did not achieve the yield strength predicted; the nominal shear stress at yield was about 925 psi or 13.9 $\sqrt{f'c}$ (6.38 MPa). The corresponding values at yield were 866 psi or 13.7 $\sqrt{f'c}$ (5.97 MPa) for BCJ3 and 772 psi or 12.3 $\sqrt{f'c}$ (5.32 MPa) for BCJ4. These are based on the first yield observed in the reinforcing bars at the beam-column intersection. Corresponding values based on confined area are 1001 psi (6.90 MPa) for BCJ2, 1103 psi (7.60 MPa) for BCJ3 and 969 psi (6.68 MPa) for BCJ4.. Up to the yield point the shear levels were about the same for all specimens. The differences in behavior can be attributed primarily to the differences in anchorage length and its interaction with the shear cracking.

The anchorage performance of the bars was monitored by small strain gages located at the beam-column joint interface, at the center of the joint, and at 2 in (5mm) intervals on the beams. The bond deterioration studies centered on measuring the rate of bond stress deterioration with cycling by monitoring strain gages at these critical sections. In the joint region the "bond efficiency" (β) was quantified by dividing the actual bond stress by the nominal bond stress (Γ_n) if the bar were to go from yielding in tension on one side to no stress on the other. This is considered the minimum acceptable bond performance of the joint. Thus a value of 100% or more indicates that the joint was performing as expected. The value can exceed 100% because for a long anchorage it is conceivable that the far end of the connection could be in compression.

Table 1 summarizes the bond calculations. The nominal design bond demand (Γ_n) is based on nominal properties and $\alpha = 1.0$. The design bond stress (Γ_u) is the maximum anticipated bond stress assuming $\alpha = 1.25$. The maximum bond stress achieved during the tests was Γ_m, and was generally reached in the deflection level prior to yielding. As can be seen, only the specimen with 28 d_b exceeded the desired limits, while the specimen with 24 d_b reached its design capacity but was not able to maintain it. It is well known that bond stresses above 600 psi will lead to significant deterioration with cycling; the poor performances of BCJ1 through BCJ3 was not unexpected.

The remainder of the table shows the bond stresses measured over two gage lengths normalized by the design bond stress. The stresses were measured over the front half of the joint (end where the bar was being pulled) and back half (side where the bar should be at zero tension), for the first cycle at yield. For an anchorage of $16d_b$ the bars were capable of transferring only an average of 64% of the required bond stress; for the $20d_b$ case the situation did not improve much with only a 71% efficiency; the $24d_b$, transferring 86% of the required bond, begins to approach the desired limit; and the $28d_b$, transferring 97%, indicates a good limit for design.

The bar slips were monitored using "slip wires" (Ref. 4), and data such as that shown in Fig. 2 were developed. The bond stress shown in Fig. 2 is the average bond stress over the front part of the joint, while the slip was measured at the critical section. Since the bond stress shown represents an average over at least ten bar diameters, the curve does not show a sharp drop after the maximum bond stress is reached; there is a drop, but it is gradual (Ref. 4). Envelopes for the data from the bond stress vs, slip curves were computed resulting in curves such as the one shown in Fig. 3. These curves showed a significant influence of the direction of loading, since the maximum values in the original direction (positive values for this case) were typically 10 to 20 percent higher than in the opposite direction. Also, the slope of the curves seemed steeper for the second direction of loading; this is probably due to the smaller bond demand. The sharp drop in bond stress was evident only in the original direction of loading.

TABLE 1 - BOND PERFORMANCE

Specimen	Anchorage Length (in)	Design Bond (Γ_n) (psi)	Ultimate Bond (Γ_u) (psi)	Maximum Bond (Γ_m) (psi)	Front Bond (β)	Back Bond (β)	Total Bond (β)
BCJ1	16	1082	1352	----	55	73	64
BCJ2	20	866	1082	572	80	63	71
BCJ3	24	721	901	703	73	100	76
BCJ4	28	618	773	1002	103	90	97

The resulting curves were averaged and smoothed to obtain the curves shown in Fig. 4. The data indicated that initial slip can be related to an ultimate average value of bond stress of approximately 600 psi, and that the magnitudes of slip are directly related to the anchorage length. For BCJ2 there was a sharp drop of bond stress with cycling after it reached 572 psi, while for BCJ3 the peak of the curve was reached at 703 psi. For BCJ4 the maximum value was probably not reached in the test, since the bond stress exceeded 1000 psi without deteriorating. The strain gages at the critical sections underwent severe straining due to local yielding and it was not possible to track the strains much beyond the first yield stage. Maximum values of slip measured varied from 0.1 in (2.5 mm) for $20d_b$ down to 0.05 in (1.2 mm) for $28d_b$; the maximum slip for $28d_b$ was measured at ductilities roughly twice that of the $20d_b$ one.

SUMMARY

The study showed that a minimum anchorage of $24d_b$ is required for the beam to reach its ultimate strength and that $28d_b$ are necessary to insure adequate hysteretic behavior. A large amount of shear cracking and spalling was observed in the joints with $16d_b$ and $20d_b$, leading to a very rapid deterioration of behavior. The study also showed that the ACI 318-86 Appendix A requirement, that the moment capacity of the column be at least 20% greater than the sum of the beam moment capacities, is inadequate to insure ductile failure and the formation of a strong column-weak beam mechanism. A ratio of moment capacities of at least 1.6 to 1.8 is necessary for uniaxially loaded joints, and 2.4 to 2.8 for biaxially loaded joints (Ref. 5).

To detail joints to dissipate large amounts of energy without excessive joint shear cracking and bond deterioration, the following new limits are proposed:

$$N_{bd} > \alpha_j + 10 \qquad \text{(Eq. 3)}$$

where N_{db} is the number of bar diameters required for anchorage. This will force the use of large anchorage lengths if high shear stresses are desired for design.

CONCLUSIONS

The data indicate that joints in moment-resisting frames designed with only $20d_b$ of anchorage length can expect to suffer significant shear damage and slip in the joint region, and that $24d_b$ should be considered a minimum design value. Good energy dissipation and formation of a strong column-weak beam mechanism can be insured only by a large moment ratio and anchorage lengths of $28d_b$ or higher.

REFERENCES

[1] American Concrete Institute, "Building Code Requirements for Reinforced Concrete (ACI 318-83)," ACI, Detroit, 1983.

[2] ACI-ASCE Committee 352, "Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures," ACI Journal, Vol. 82, No. 3, May-June 1985, pp. 266-284.

[3] Leon, R.T., "Behavior of Interior Beam-Column Joints with Variable Anchorage," Tech. Report 88-03, Dept. of Civil Engineering, U. of Minnesota, Minneapolis, May 1988.

[4] Fillipou, F.C., Popov, E.P., and Bertero, V.V., "Analytical Studies of Hysteretic Behavior of R/C Joints," J. of Structural Eng., ASCE, Vol. 111, No. 7, July 1986, pp. 1605-1622.

[5] Leon, R.T., "Bidirectional Loading of R.C. Beam-Column Joints," Earthquake Spectra, EERI, Vol. 2, No. 3, 1986, pp. 537-564.

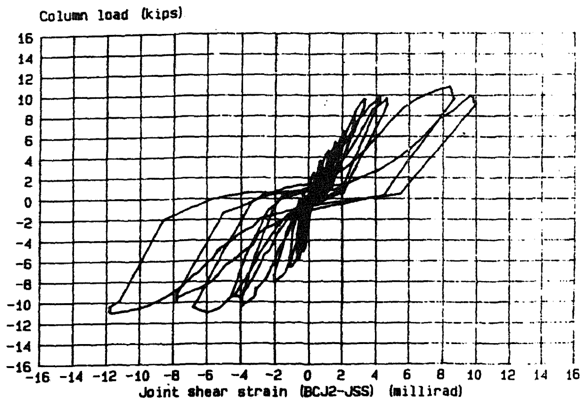

Figure 1 - Joint shear strain vs. column load for specimen BCJ2.

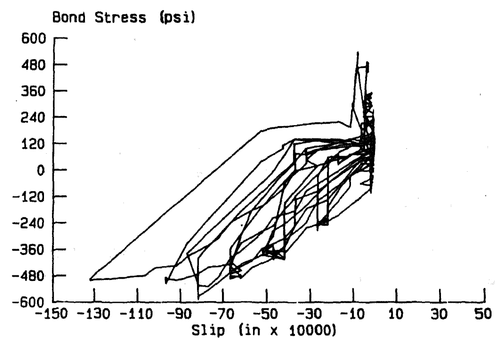

Figure 2 - Total slip vs. bond stress for top bar (#4) of BCJ2.

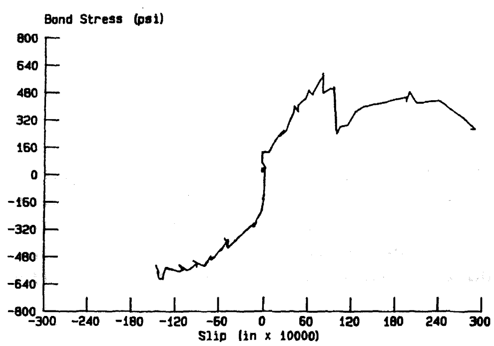

Figure 3 - Envelope of total slip vs. average bond stress for top bars of BCJ2. Positive slip is for the right beam and negative slip is for left beam.

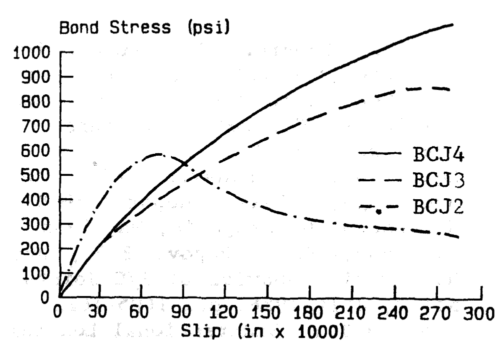

Figure 4 - Smoothed curves for average bond stress vs. slip for tests with 20, 24, and 28 bar diameters of anchorage.