INTERIOR JOINTS WITH VARIABLE ANCHORAGE LENGTHS

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ABSTRACT: Four half-scale R.C. interior beam-column joints were tested to assess the validity of current anchorage provisions for interior beam-column joints. The specimens had beam bars with anchorage lengths of 16, 20, 24, and 28 bar diameters through the joint. The tests show that anchorage length is as important as shear stress in design, and that there is no experimental evidence to support lowering the current anchorage requirements of 20 bar diameters. Current design recommendations are adequate for moderate earthquakes, but would probably lead to significant stiffness and strength losses under a major earthquake. The data on bar slippage and anchorage indicate that 24 bar diameters is very close to the ideal anchorage length for interior beam-column joints.

INTRODUCTION

Ductile moment-resisting frames (DMRFs) are a very economical and efficient structural system for buildings in seismic areas. They are economical to construct because they are typically repetitive in plan, and they are attractive to building owners because of the space flexibility they offer. From the structural standpoint, ductile frames are very efficient because they can be designed so that plastic hinges form in beam regions adjacent to the joint when the frame is subjected to large lateral loads. These hinges are capable of large energy dissipation, and if properly detailed they can retain that capability through many large cycles of load reversals.

The designer of DMRFs, however, is faced with very stringent requirements to insure the proper performance of the structure. In order to preserve stability and avoid story mechanisms, the columns must be substantially stronger than the beams framing in at any joint (*Building* 1983; ACI ASCE Committee 352 1985). A second requirement is that the nominal joint shear stress must be kept low to prevent excessive joint shear deformation. A third constraint is the requirement that the anchorage length of the beam bars must be at least 20 bar diameters. Because low percentages of steel are generally desirable in seismic areas to improve the member ductility, and anchorage and shear stress are linked to the column size, large columns are the logical result of these design requirements.

For the designer the anchorage length is a problem because the size of the bars is generally not known until a preliminary design is completed. Thus the designer is often faced with the choice of: (1) Using many small bars in the beam; or (2) increasing the column size in order to satisfy this criterion. The present requirement in American design recommendations of 20 bar diameters (*Building* 1983, ACI ASCE Committee 352 1985) is substantially lower than that in the New Zealand provisions, which can require up to 35 bar diameters for anchorage (*Code* 1980). Thus any relaxation of current

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ACI design recommendations would have to be supported by extensive experimental data and a sound theoretical framework.

To study the effect of anchorage length on joint performance, four half-scale R.C. interior beam-column joints were tested. The primary objective was to assess the validity of current shear and anchorage provisions for interior beam-column joints. In particular the research was intended to determine whether the current provisions for beam bar anchorage are adequate and to improve the understanding of bond behavior under both elastic and inelastic cyclic loads.

EXPERIMENTAL PROGRAM

Four half-scale specimens were tested (Fig. 1). All had beams 8 in. (203 mm) wide by 12 in. (305 mm) deep, with four #4 (13 mm) bars at the top and four #3 (9 mm) bars at the bottom. The column width was kept constant at 10 in. (254 mm), while the column depth was increased from 8 in. (203 mm) to 14 in. (356 mm) at 2-in. (51 mm) increments. The tests were labeled BCJ1 through BCJ4 with increasing column size. Thus the top bars had anchorage lengths ranging from 16 to 28 bar diameters, while the bottom bars had anchorage lengths varying from 21.3 to 37.3 bar diameters. The nominal material properties used for design were $f'_c = 4$ ksi and $f_y = 60$ ksi, and the actual material properties did not vary substantially from these.

Details of the geometry and reinforcement of the specimens are given in Table 1. It should be noted that the specimen did not conform to present rules regarding cross ties in the columns, and that the joint shear reinforcement was about half that of current recommendations. The intent was: (1) To study the interaction between shear cracking and bond deterioration when nominal shear stresses in the joint are in the range of $12\sqrt{f'_c}$ to $15\sqrt{f'_c}$, well below the current limit of $20\sqrt{f'_c}$; and (2) to determine the behavior of connections with low joint reinforcement ratios, such as those for most frames built before 1970. Specimen BCJ1 was the first tested and did not incorporate much of the outside instrumentation used in the latter tests; therefore, only few references will be made to BCJ1, except for the data obtained from strain gages and relating to bond strength and deterioration. Due to space limitations only the data from the top bars will be discussed (Leon 1988).

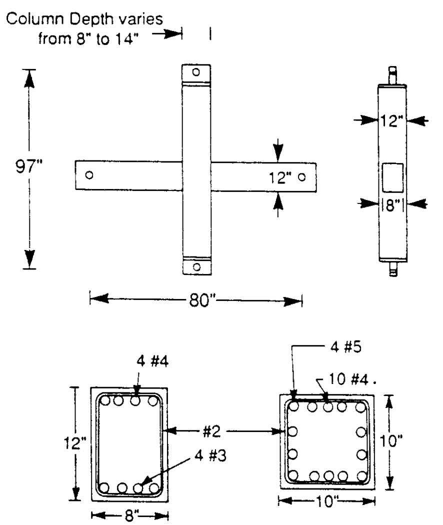

FIG. 1. Specimen Details (BCJ2)

TABLE 1. Specimen Details

Specimen (1)	BCJ1 (2)	BCJ2 (3)	BCJ3 (4)	BCJ4 (5)
Beam	8 in. × 12 in.	8 in. × 12 in.	8 in. × 12 in.	8 in. × 12 in.
Top beam reinf.	4 #4	4 #4	4 #4	4 #4
Bottom beam reinf.	4 #3	4 #3	4 #3	4 #3
Transverse reinf.[a]	#2 at 2 in.	#2 at 2 in.	#2 at 2 in.	#2 at 2 in.
Beam length[b]	44 in.	42 in.	40 in.	38 in.
Column	8 in. × 10 in.	10 in. × 10 in.	12 in. × 10 ft	14 × 10 in.
Column reinfor.				
(face)	4 #5, 6 #4	10 #4	10 #4	8 #4
(side)	4 #4	4 #4	4 #4	4 #4
Joint transv. reinf.	4 #2	4 #2	4 #2	4 #2

[a]#2 deformed rebar was obtained from PCA.
[b]Distance from strut to column face.

An overall view of the test setup is given in Fig. 2. In order to develop design guidelines for bar anchorages, two sets of instrumentation were used. The first was used to monitor loads, deflections, rotations, and shear strains in the specimen. The displacement at the top and bottom of the column, and

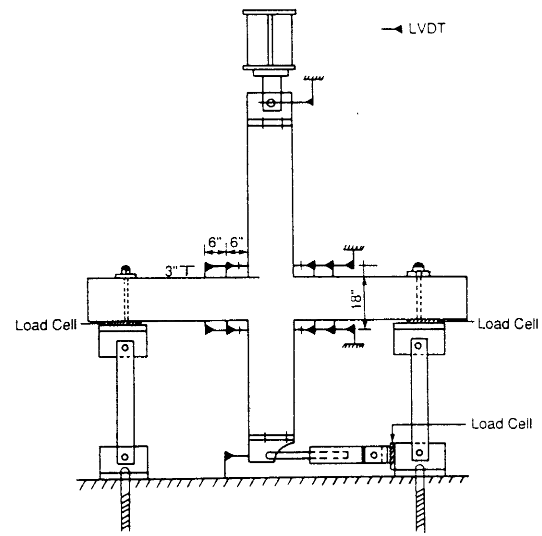

FIG. 2. Test Setup and LVDT Layout

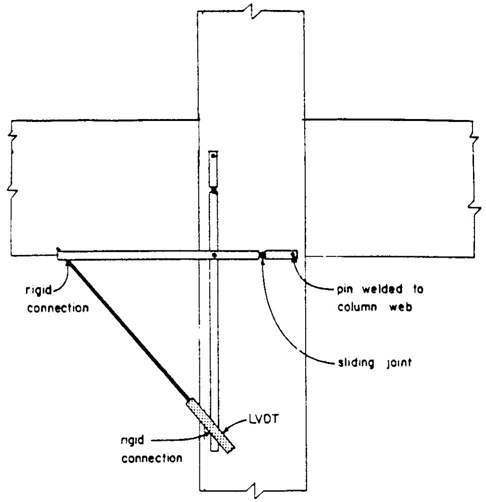

FIG. 3. Shear Strain Measurement Device

the rotation of the beams with respect to the column, measured over lengths of one-half and one beam depth, were monitored using LVDTs. Another absolute joint rotation measurement was made to an external reference frame and the shear strain was monitored with a frame shown in Fig. 3. These measurement techniques were originally developed at the University of Texas at Austin by Jirsa and Burguieres (Leon 1983), and further refined by Leon (1986). They measure the rotation between two rods attached to three reference inserts at the joint corners. Each rod is actually made of two pieces, one inside another, to allow freedom of horizontal movement between the three measurement points.

The second set of instrumentation was intended to provide data on the local state of stress and slip of the reinforcement. There were 30 gages on the beam bars and 16 on the column bars. Small gages were used to minimize the disturbance to the bond. For the same reason, the wires were routed away from the bars as near as possible to the gage.

The slips were monitored using "slip wires." This instrumentation was also originally developed at UT-Austin (Leon 1983). It essentially consists of a very stiff wire firmly anchored to the bar at one end, and to a spring-loaded LVDT at the other. The wire is housed in a lubricated plastic casing to insure minimum friction, and is allowed about 0.50 in. (13 mm) of free movement at the bar end. By using very sensitive LVDTs and proper signal conditioning, a very high resolution (0.0002 in.) can be obtained. By attaching one of these at each side of the joint and using the data from the strain gages, an approximation to the bar slip can be made.

It should be noted that bar slippage, as usually defined, refers to the movement of the bar with respect to the surrounding concrete at a point. It is made up of elastic and inelastic deformations and rigid motion of the bar. In the strictest sense, only the latter should be considered slip, but it is extremely difficult to isolate these components.

Steel Strains in Beam Bars

Since monitoring of the steel stresses in the beam bars was considered very important in assessing the performance of the specimens, one of the longitudinal steel bars in each specimen was extensively instrumented. Nine strain gages were installed on a bar—one at the column centerline, two at the beam-column interface, and three on each beam at 2-in. (51-mm) intervals.

One of the primary differences between these tests and others is the large number of cycles imposed below the yield limit. After an initial cycle at an inter-story displacement of 0.10 in. (2.5 mm) to determine the initial stiffness of the system, the specimens were cycled at increments of 0.25 in. (6.3 mm) of inter-story displacement until yield was achieved. Afterwards, cycles at 1.5, 2.0, and 3.0 times the yield deflection were imposed. Two cycles at each deflection level were imposed to assess the amount of cumulative damage from subsequent cycles.

Comparison of the stress profiles for the four specimens at levels roughly corresponding to 0.5 f_y, 0.75 f_y, and yield are shown in Figs. 4 through 7. In an ideal joint working at the yield level the bars would go from yield in tension on one side to some compression on the other. This would imply that all the bond forces are being transferred within the joint and that negligible bond deterioration with cycling would occur below the yield level.

The effect of anchorage length even when cycling in the elastic range can

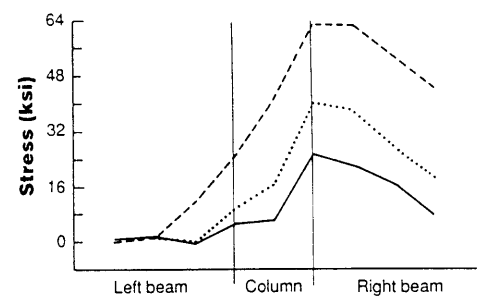

FIG. 4. Stresses in BCJ1 (16d_b, Negative Direction)

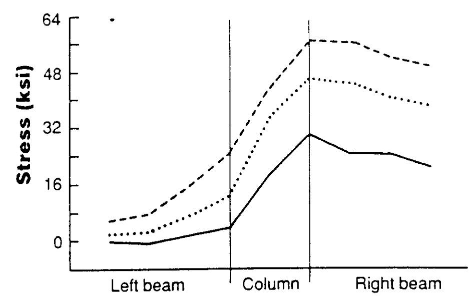

FIG. 5. Stresses in BCJ2 (20d_b, Negative Direction)

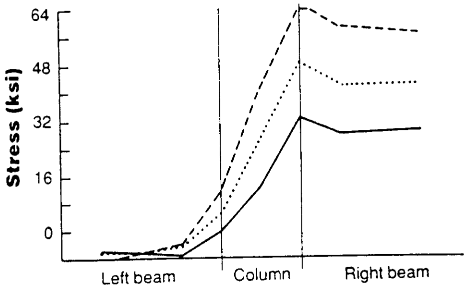

FIG. 6. Stresses in BCJ3 ($24d_b$, Negative Direction)

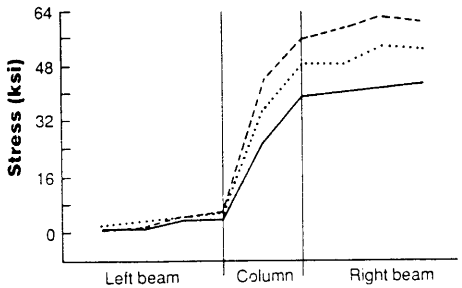

FIG. 7. Stresses in CBJ4 ($28d_b$ Negative Direction)

be seen for BCJ1 and BCJ2 (Figs. 4 and 5), where large shifts in the tensile strains occured with cycling below yield. BCJ3 (Fig. 6) showed somewhat better behavior, but still showed a perceptible upward shift of the tensile strains with cycling in the elastic range. Only the specimen with $28d_b$ (BCJ4) was able to withstand the cycling without undergoing significant bond deterioration. Data from all the tests showed no noticeable deterioration with cycling at the same deflection level. Fig. 8 shows the strains measured for BCJ2 at different levels up to yield. It seems that most of the damage is concentrated in the initial cycle of loading, regardless of the anchorage length.

Fig. 4 shows that for very short anchorage lengths ($16d_b$, BCJ1) the full force of the bar could not be transmitted to the column even at half of the yield stress. When the steel yielded in tension on one side of the joint, the other side had about 22 ksi (152 MPa) of tensile stress, rather than the compressive stress required by the weak beam-strong column mechanism. The situation did not improve much for the BCJ2 ($20d_b$) which also had about 23 ksi (158 MPa) in tension at that stage (Fig. 5). The behavior began to improve as the anchorage length reached $24d_b$ (BCJ3), when the tensile stress had decreased to about 12 ksi (83 MPa), as seen in Fig. 6. Behavior close to that idealized in design was only noted when the anchorage length was increased to $28d_b$ (BCJ4, Fig. 7), when the tensile stress was a negligible 4 ksi (27 MPa).

Stress profiles for loading in the opposite direction indicated only a slight effect of the original direction of loading. The stress in tension at the far

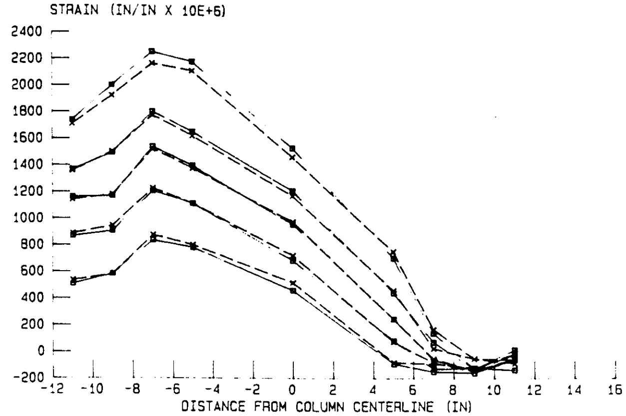

FIG. 8. Strain Profiles for BCJ2 ($20d_b$, Negative Direction)

end of the joint for BCJ2 was about 24 ksi (165 MPa) at $0.5 f_y$, comparable to that shown in Fig. 5 for the original direction of loading. The major change in this tensile force occurred as the cycling was increased from 0.5 to 0.75 of the yield load, as opposed to the other direction of loading, when the major change occurred as the load was increased from 0.75 to 1.00 of the yield load. This change in tension was also very large for BCJ3 as the load changed from 0.75 to 1.00 of the yield stress, with a jump from 10 to 22 ksi (69 to 152 MPa). Once again, BCJ4 was the only specimen to show relatively low tensile stresses (about 8 ksi, or 55 MPa) and no significant increase in tensile stress as the load increased from 0.75 to 1.00 of the yield stress.

To quantify the effect of anchorage length, it was decided to use the concept of "bond efficiency." Bond efficiency was defined as the average bond stress, measured by the strain gages at the beam-column interfaces and center of the column, divided by the bond stress required to bring the bar from yielding (or whatever force was present in the beam) in tension on one side to zero force at the other side. Since there was a gage at the middle of the joint, it was possible to compute a different bond efficiency for the front half of the anchorage length, where the largest tension should be present, and the back half. The results are tabulated in Table 2 for the peak at which first yield was attained by all specimens.

For an average length of $16d_c$ the average bond efficiency was only 64%.

TABLE 2. Bond Efficiency

Anchorage length (bar diameter) (1)	Full bond (psi) (2)	Front bond (%) (3)	Back bond (%) (4)	Efficiency (%) (5)
16	954	55	73	64
20	764	80	63	71
24	636	73	100	86
28	545	103	70	97

More important, the front part was carrying less bond stress than the back part due to bond deterioration with cycling in the elastic range. Neither of the two halves, moreover, was carrying anywhere near its desired bond capacity. The front was carrying only 55% and the back 73%. The increase in bond efficiency was relatively small for the next specimen, BCJ2, with $20d_b$ of anchorage. Significant gains were made when the anchorage reached $24d_b$, with a bond efficiency of 86%. The best behavior was evidenced by BCJ4, with $28d_b$, where the bond efficiency reached 97%, and the front half of the bar was carrying more than the nominal required bond stress even after a large number of cycles.

Another important conclusion reached from the strain gage data is that the bars at the compressive side of the joint only picked up minimal amounts of compression for the case of BCJ4. The shear is carried by the joint either through a strut or a panel truss mechanism, depending on the anchorage conditions. The lack of any compression strains in these bars indicates that a panel truss mechanism did not seem to be active for these specimens. Most of the load was transferred by a strut mechanism even when the anchorage lengths reached 28 bar diameters. Given the performance of BCJ4, it is not justifiable to ask for larger anchorage lengths than $28d_b$ or to require large amounts of joint transverse steel to sustain a hypothetical truss mechanism.

Slippage of Reinforcement

The slippage of the reinforcement was monitored at six locations with the aid of the "slip wire" instrumentation. Four of the measurements were on the top #4 (13 mm) bars, and two on the bottom #3 (9 mm) bars. The measurements were made with 1-in. (25.4-mm) LVDTs calibrated for a full output of +/− 0.25 in. (6.3 mm) and yielded consistent and reliable data.

The slip data are shown plotted versus the beam moments (Figs. 9 and 10). The beam moments are linearly proportional to the forces in the steel bars, and thus are directly proportional to the bond stress if a linear variation of bond stress across the column is assumed. Therefore, a graph of the slip versus moment will have the same general shape as that of a graph of slip versus average bond stress.

Fig. 9 shows a typical slip versus moment curve for a top bar, while Fig. 10 shows a typical one for a bottom bar in BCJ2. The envelopes of slip for three of the top bars are shown in Fig. 11 and seem to be divided into four zones, as shown in Figs. 12. Zone 1 is an area where no slip seems to occur with initial loading, and is associated with: (1) The bond due to chemical adhesion between the bar and the surrounding concrete; and (2) the uncracked concrete carrying tension. This component disappears after the initial cycle of loading. Although visible cracks were not evident at this point, it can be hypothesized that small cracks had occurred, as evidenced by the jump in the strain readings at this level. Zone 2 is associated with a slow and linear deterioration of behavior in the elastic range. It is thought that Zone 2 is associated with: (1) The progressive crushing taking place in the front of the lug as the loads are cycled; and (2) the growth of splitting cracks from the top of the lugs. Zone 3 is an area of rapid bond deterioration with cycling, when large portions of the concrete in front of the lug have been crushed. This zone is also associated with opening of the radial cracks, and the shearing of the concrete in front of the lug. Zone 4, or an area of plas-

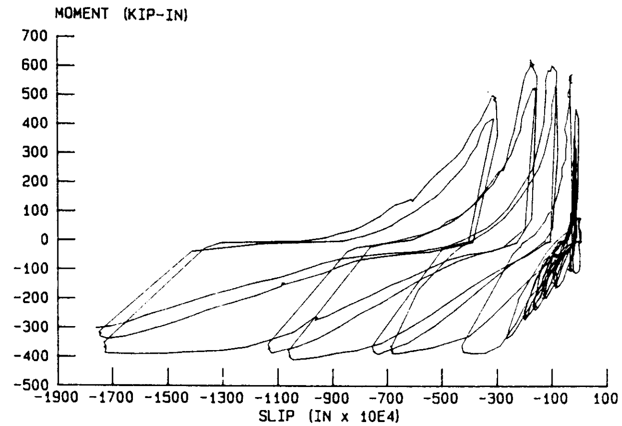

FIG. 9. Typical Slip versus Beam Moment Curve for Top Bar of BCJ2

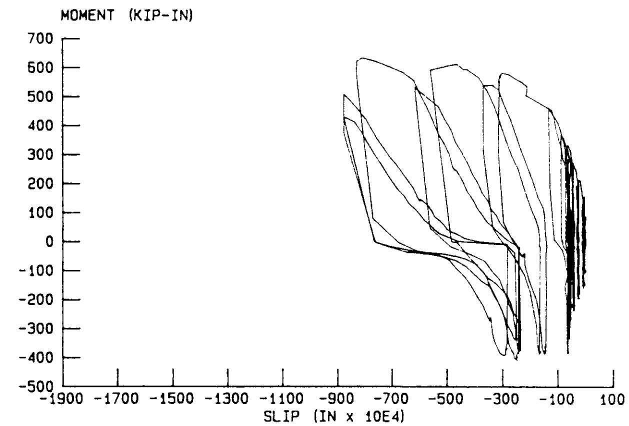

FIG. 10. Typical Slip versus Beam Moment Curve for Bottom Bar of BCJ2

tification and slip of the reinforcement, is the very flat and long portion of the curve reached when the member achieves its full moment capacity.

The curves shown here refer to an average bond stress rather than a localized bond stress; the latter would have a sharp decreasing slope immediately after its maximum value was achieved. In the curves shown here the beam moment is assumed to be proportional to the average bond stress across the entire joint.

The most interesting portion of the cyclic curves shown (Figs. 9 and 10), however, refers to the flat portions near the zero moment region. These indicate actual rigid body motion of the bar as the stress reversal occurs. Ideally true slip would be associated with movement with no change in load. Due to the brittle nature of concrete, some small grains are likely to be broken off with cycling, allowing the bar to pick up some load by friction.

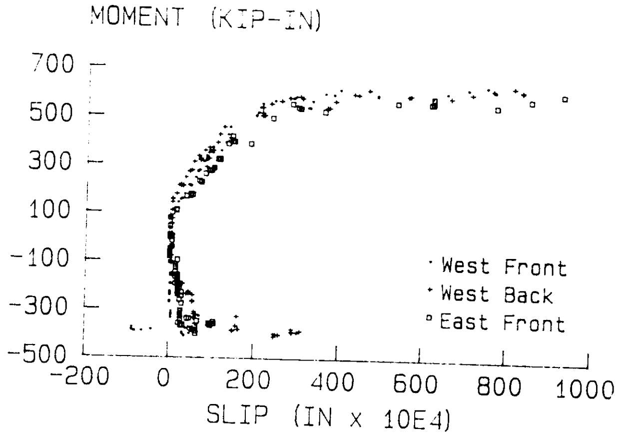

FIG. 11. Envelopes of Slip for Top Bars

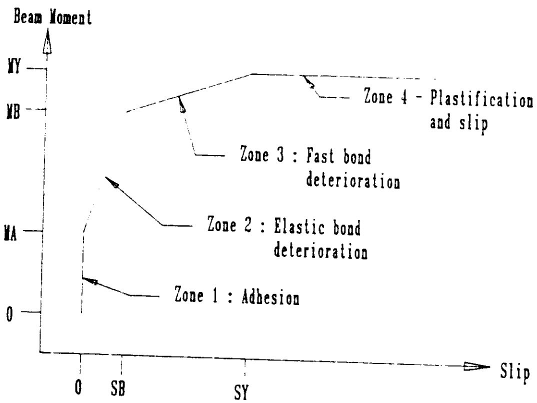

FIG. 12. General Regions of Slip versus Bond Stress Curves

Thus the slip will occur with some minor change in stress.

From the envelopes shown, as well as from those derived for the other slip wires, the values at which the slopes change can be calculated. The moment at which initial adhesion is overcome is labeled MA (Fig. 12), and for BCJ2 it corresponds to a moment between 90 and 105 kip-in. (10.1 to 11.9 kN-m), and a nominal bond stress between 135 and 160 psi (0.93 to 1.10 MPa). No slip could be detected at this stage. The moment at which rapid deterioration begins (MB) is 450 kip-in. (50.8 kN-m) and is associated with a slip (SB) of 0.012 in. (0.3 mm) and a nominal bond strength of 675 psi (4.65 MPa). Yield occurred at a moment (MY) of about 600 kip-in. (67 kN-m), a slip (SY) of 0.04 in. (1.0 mm), and a nominal bond stress of 900 psi (6.20 MPa). The data for the beam loaded in the original direction were used to compute these values. For the only curve available in the other direction, the values tended to be 8% to 10% lower.

The values for the adhesion limit for both BCJ3 and BCJ4 were found to be very similar to those of BCJ2, averaging 100 kip-in. (11.3 kN-m). The moment at which rapid deterioration began (MB) for BCJ3 was about 500 kip-in. (56.5 kN-m), with a slip (SB) of about 0.017 in. (0.43 mm), and a nominal bond stress of 625 psi (4.31 MPa). For the MB for BCJ4, the corresponding values were 560 kip-in. (63.3 kN-m) for moment, with a slip (SY) of 0.026 in. (0.66 mm), and a nominal bond stress of 600 psi (4.17 MPa). The yield moment (MY) for BCJ3 was 600 kip-in. (67.8 kN-m) at a SY of 0.035 in. (0.90 mm) and a nominal bond stress of 750 psi (5.17 MPa). Corresponding values are 670 kip-in. (75.7 kN-m) (MY) for CBJ4, with a slip of 0.046 in. (1.17 mm) and a nominal bond stress of 720 psi (4.96 MPa).

Mechanisms Contributing to Drift

To determine the accuracy of the assumptions of full joint rigidity and no column inelastic action used in DMRF design, the contribution of different mechanisms to the stiffness of the subassemblage was calculated. The four mechanisms correspond to the elastic deformation of the beams and columns, the inelastic rotation of the beams and columns, and the joint shear strain.

Elastic Deformations

The elastic bending of the beam and column can be calculated by using the forces measured at the end of the members. Using elastic analysis, the deflections and rotations at the joint face can be computed. A major problem in this procedure is the choice of an appropriate moment of inertia of the section, as the actual moment of inertia will depend on the load level and the amount of previous cracking. For all calculations the cracked section properties were used for both the beam and column, as there was no applied axial load. The cracked moment was approximated as one-fourth of the uncracked moment; these values were very close to those predicted by an exact analysis. Elastic components contribute appreciably to the inter-story drift below yield. As the deformations increase the percentage of contribution begins to decrease because the member end loads cannot increase significantly past the yield point.

Beam Inelastic Rotation

In a properly designed joint, the intent is to form a weak beam-strong column mechanism. Thus, the majority of the inelastic rotation should concentrate in the beam adjacent to the column face as soon as the yield load is reached. The rotations measured between the beam and the column include beam elastic deformations, bar slippage, and inelastic elongation of the rebars. The beam rotations were measured using the LVDTs shown in Fig. 2.

Joint Shear Strain

As discussed in a previous paper (Fillipou 1986), joint shear cracking can be an important factor in joint behavior. A considerable amount of the total deformation can be traced to joint flexibility if shear stresses are large and anchorage lengths are short. This mechanism is ignored in the usual analysis

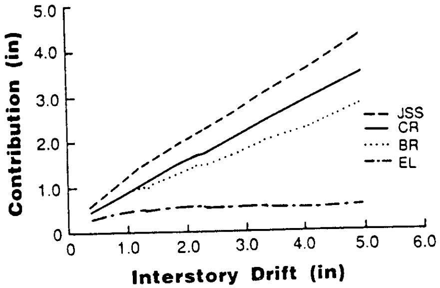

FIG. 13. Contributions to Interstory Drift (BCJ2)

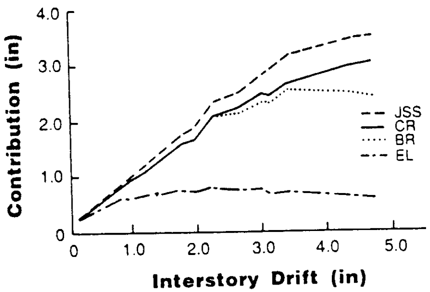

FIG. 14. Contributions to Interstory Drift (BDJ3)

procedures, which assume the joint to be rigid. The joint shear strain was calculated with the apparatus shown in Fig. 3.

Inelastic Column Rotation

Theoretically this component should be zero, since the moment capacity of the column always exceeds that of the beams. In practice, however, some local yielding and slip of the bars can occur in the column, leading to a nonzero component. The column rotations were calculated as the difference between the absolute rotation of the joint region minus the contribution of beam rotations, joint shear, and elastic deformations.

The contributions of the four mechanisms to the beam end deflections are shown in Figs. 13–15. The lines in these graphs include the contribution of all mechanisms below that line. Thus, for BCJ2 at a deflection of 1.00 in. (Fig. 13), the elastic mechanisms (EL) contributed about 0.40 in. (10 mm), and the beam rotation (BR) about 0.35 in. (9.0 mm). The inelastic column rotation (CR) was negligible, and the joint shear strain (JSS) contributed about 0.25 in (6.4 mm). For the ideal case the only contributions should be from elastic deformations and inelastic beam rotations. Thus the curves for column rotation (CR) and shear strain (JSS) should ideally plot on top of the curves for inelastic beam rotation (BR).

Fig. 16 shows a typical comparison of the calculated (solid line) to the measured (dashed line) interstory displacement. For most of the load his-

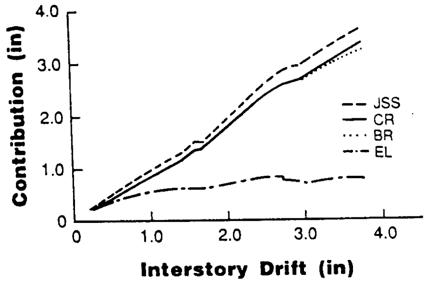

FIG. 15. Contributions to Interstory Drift (BCJ4)

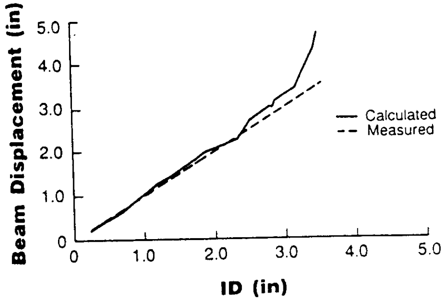

FIG. 16. Comparison of Calculated versus Measured Interstory Drift (BCJ2)

tories the agreement was excellent, while for BCJ2 (case shown) the calculated and measured interstory displacements diverged at the end due to the joint shear failure of the specimen.

For BCJ2, the data indicate that both the joint shear strain and inelastic column rotation contributed significantly to the overall deflection. Since the elastic contribution does not increase once the yield deflection has been reached, the three other mechanisms have to increase their percentage contribution to accommodate the inelastic deformations. As one would expect, beam inelastic rotations are the largest contributors after yielding, but joint shear strain began to contribute almost from the beginning of the load history. The column inelastic rotation began to contribute significantly after the beam deflections exceeded one inch (just below the first yield peak).

BCJ3, on the other hand, shows substantial contributions of column rotation only after the beam end deflections exceeded three and a half inches. Moreover the contribution of joint shear strain was small up to deflections of about two inches. Up to a beam end deflection of 3.5 in (89 mm) BCJ3 behaved very well, with the vast majority of the deflection coming from elastic deformations and inelastic beam rotations. After the beam end deflection exceeded 3.5 in (89 mm) the column began to contribute appreciably due to increased cracking and crushing of the column corners.

BCJ4 performed very well. Although shear strain contributed to the overall deformation from the start, its percentage contribution was below 10%

for most of the load history. Moreover, the contribution of inelastic column rotation was negligible. Fig. 15 shows the type of behavior that would justify the assumption of a weak beam-strong column mechanism, and the ignoring of joint shear strains contributions to frame deflections.

Limitations of Investigation

The data generated in this investigation need to be evaluated with respect to two important assumptions. The first assumption is that the half scale of the specimens provides an adequate model of a full-scale prototype. Thus it is assumed that the #4 (13 mm) bars used adequately model a #8 (25 mm) bar insofar as bond behavior is concerned. Every effort was made to maintain uniform rib areas from specimen to specimen and to use bars with typical rib geometries and spacing. The data generated correlate very well with that obtained by the writer on full-scale specimens (Leon 1983, 1986). It should not be inferred, however, that a linear scaling of bond properties is assumed. Bond is a complex phenomenon that depends on material properties, cover and spacing provided, related rib area, and the state of stress around the bar. This investigation, and the review and comparison of other research efforts (Abrahms 1987; Fillipou 1986), indicate that the data obtained can be properly extrapolated up to #9 (28 mm) bars.

The second assumption regards the end conditions assumed for the specimen. These are no different from the vast majority of the tests performed on beam-column joints, in that the ends of the beams were restrained only vertically. It has been argued that a large amount of the slip measured in laboratory tests is a direct result of this lack of restraint. In a continuous structure such slip would not be likely, unless all the joints at a particular story had deteriorated sufficiently as to allow slip across all joints simultaneously. Moreover, the amount of slip needs to be scaled appropriately (Fillipou 1986).

Conclusions

The data described in this paper lead to the following conclusions.

1. An anchorage length of 28 bar diameters is necessary to insure that a weak girder-strong column mechanism can be maintained through a severe load history.
2. Most of the force transfer is accomplished by a strut mechanism, since the lack of joint reinforcement precludes a panel truss mechanism from carrying a substantial portion of the shear. For a connection with large anchorage lengths (equal or greater than 24 bar diameters) only a minimum amount of transverse steel should be required in the joint region. The minimum required for column confinement seems a reasonable choice for minimum transverse steel in the joint.
3. The absolute magnitude of slip measured in all three tests was very similar. However, the value at which the slip initiated varied substantially. This latter value seems to be a better predictor of slip behavior. From the flat portions of the load-deformation curves, it is seen that the bars in BCJ2 were slipping more than those of BCJ4. On the other hand, the amount of inelastic yielding of the bars was substantially greater for BCJ4 than for BCJ2. Unfortunately, there is not a clear way of separating the contributions of these two mechanisms.

4. The contribution of inelastic column rotation was proportional to the moment ratio. For BCJ2, with a moment ratio of only 1.14, the column rotation became a significant factor after yield was achieved. For BCJ3 column rotation became significant only after ductilities higher than two were imposed. For BCJ4, on the other hand, there was very little contribution, if any, of inelastic column rotation. This observation leads to the conclusion that an increase in moment ratio could be a good design solution in order to decrease the anchorage requirements of the column bars.

Appendix. References

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