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A STUDY OF BOND DETERIORATION
REINFORCED CONCRETE BEAM-COLU
JOINTS

by

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A C K N O W L E D G M E N T S

From June 1982 to July 1983, Mr. Zhu was engaged in research work at The University of Texas at Austin as a Visiting Scholar from The People's Republic of China. The work was conducted in the Phil M. Ferguson Structural Engineering Laboratory at the Balcones Research Center of The University of Texas at Austin, under the direction of Professor James O. Jirsa. Roberto T. Leon and Milind R. Joglekar, Graduate Research Assistants, were of great help in providing data and assistance on reviewing the literature.

ABSTRACT

The bond behavior and crack pattern in the joint area of reinforced concrete at different load stages are studied in order to understand the nature of deterioration of bond strength.

It is difficult to relate bond deterioration to lateral drift of frame structures under earthquake loadings. Most design recommendations and codes require the calculated inelastic interstory drift be less than 0.01 to 0.02 of the floor height. In this paper a relative interstory drift of 0.03 is suggested for evaluating the performance of beam-column joint test specimens. Using this criterion, tests conducted at the University of Michigan , The University of Texas at Austin, the University of California-Berkeley, and the University of Canterbury, New Zealand are reevaluated.

From the reevaluation of the previous test results, design recommendations for the ratio of column width to beam bar diameter are made. Similar suggestions are made for the ratio of beam depth to column bar diameter.

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CHAPTER I

INTRODUCTION

1.1 Background

The ability of many multistory reinforced concrete frames to resist strong earthquakes has been demonstrated in Chile (1960) [1], Yugoslavia (1963) [2], Alaska (1964) [3], Caracas (1967) [4, 5], Tokachi-Oki (1968) [6], and San Fernando (1971) [7]. However, damage to frame structures in severe earthquakes has been observed also. In order to withstand a severe earthquake a frame structure should be able to dissipate the input earthquake energy through large inelastic deformation reversals. The sources of potential brittle failure must be eliminated. Thus, it is necessary to prevent premature crushing and shearing of concrete, as well as sudden loss of bond and anchorage. Intensive research activities have been conducted since 1967. Much of the research work has concentrated on ensuring the strength and ductility of the frame through careful detailing of members and joints. Design rules and details for beam-column joints have been presented in a series of recommendations and in building codes [8, 9, 10].

Although it is widely recognized that anchorage conditions are adversely influenced by load reversals, quantitative design rules are lacking in most recommendations. The ACI 352 report [8] and ACI building code [10] contain provisions for hooked bars in exterior joints, but give no specific recommendation for continuous bars through interior joints, except in a commentary form which states that small diameter bars are desirable. To reduce bond deterioration, the New Zealand Standard [9] requires that the diameter of longitudinal beam or column bars passing through the joint should be less than 1/20 of the column width for Grade 40 reinforcement and 1/25 for Grade 60. Based on "push-pull" tests of bars, researchers at the University of California-Berkeley suggest that an anchorage length of approximately 25 d_b or 35 d_b is necessary for satisfactory performance of Grade 40 or Grade 60 deformed bars respectively [11]. The difference between design suggestions for the beam bar anchorage lengths and the lack of specific guidelines in some codes reflects the lack of test data and different design philosophies.

Any specimen will fail under large loads or deformations. In the case of inelastic load reversals, the magnitude of deformation becomes a critical consideration. However, there are wide differences in the deformation histories used in tests. Many beam-column joint tests have been evaluated using a ductility factor with the beam or column end displacement reaching very high values. Under large deformation histories, beam bars pull through the column in almost all the tests. However, from reports of post-earthquake inspections, there is little correlation between severe structural damage and poor performance of joints. Therefore, the deformation history used in previous beam column joint tests must be questioned.

1.2 Objective

It is difficult to determine the maximum lateral drift of a frame structure expected under a severe earthquake. However, most design recommendations and codes require the calculated inelastic interstory drift be less than 0.01 to 0.02 of the floor height. In this paper a relative interstory drift of 0.03 is suggested for evaluating the performance of beam column joint test specimens. Using this criterion, tests conducted at the

University of Michigan [17], The University of Texas of Austin [15, 18, 19, 20, 26, 27, 28], the University of California-Berkeley [11,13], and the University of Canterbury, New Zealand [12] are reevaluated.

The bond behavior and crack pattern in the joint area at different load stages are studied in order to understand the nature of deterioration of bond strength. From the reevaluation of the previous test results, design recommendations for the ratio of column width to beam bar diameter are made. Similar suggestions are made for the ratio of beam depth to column bar diameter.

CHAPTER II

BOND DETERIORATION IN AN INTERIOR BEAM-COLUMN JOINT

2.1 Deformation History of a Beam Column Joint Under Load Reversal

Figures 1-3 show the deformed shape of a beam column joint at various load stages. At load stage A the beam bar has yielded (Fig. 1). A large crack is generally found at the interface between the column and beam, but the beam concrete compression zone is still in contact with the column. The crack width, ΔA, is produced by beam bar slippage or yielding. If the column width is sufficient in this case, the embedded bar will be anchored within the column width and the beam steel will be in tension at one side of the column and in compression at the other side. Since the beam concrete is still in contact with the column, the compression force is carried mostly by the beam concrete, and the compressive stress in the beam steel at the column face is very low (Fig. 4). This situation has been observed in several tests, especially when beam end deflections are small [12]. If the column width is not sufficient, the bar will be anchored not in the column but in the beam at the other side of the column. This situation has been observed in almost all tests with large beam end deflection and small column width-beam bar diameter ratio. In this case the beam steel is in tension on both sides of the column. Figure 4 shows that at first yield, the top steel is in tension on both sides of the column while the bottom steel is in tension at one side and in compression at the other side. Because the beam bar can be anchored either in the joint or in the beam on the other side of the column, total bond damage or beam bar pull through is impossible. However, local bond damage will occur.

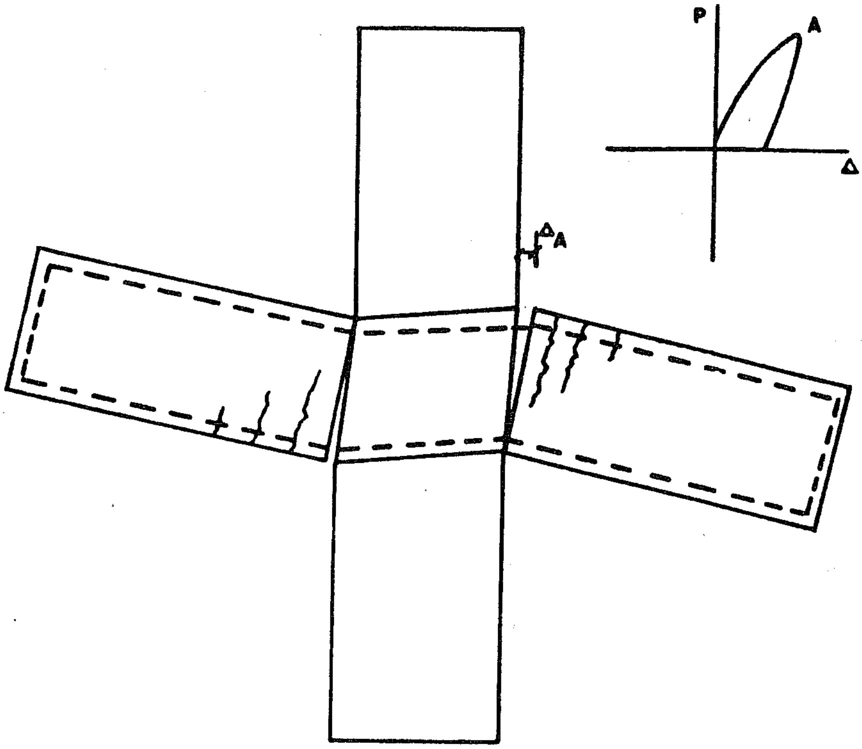


Fig. 1 Deformation at load stage A.

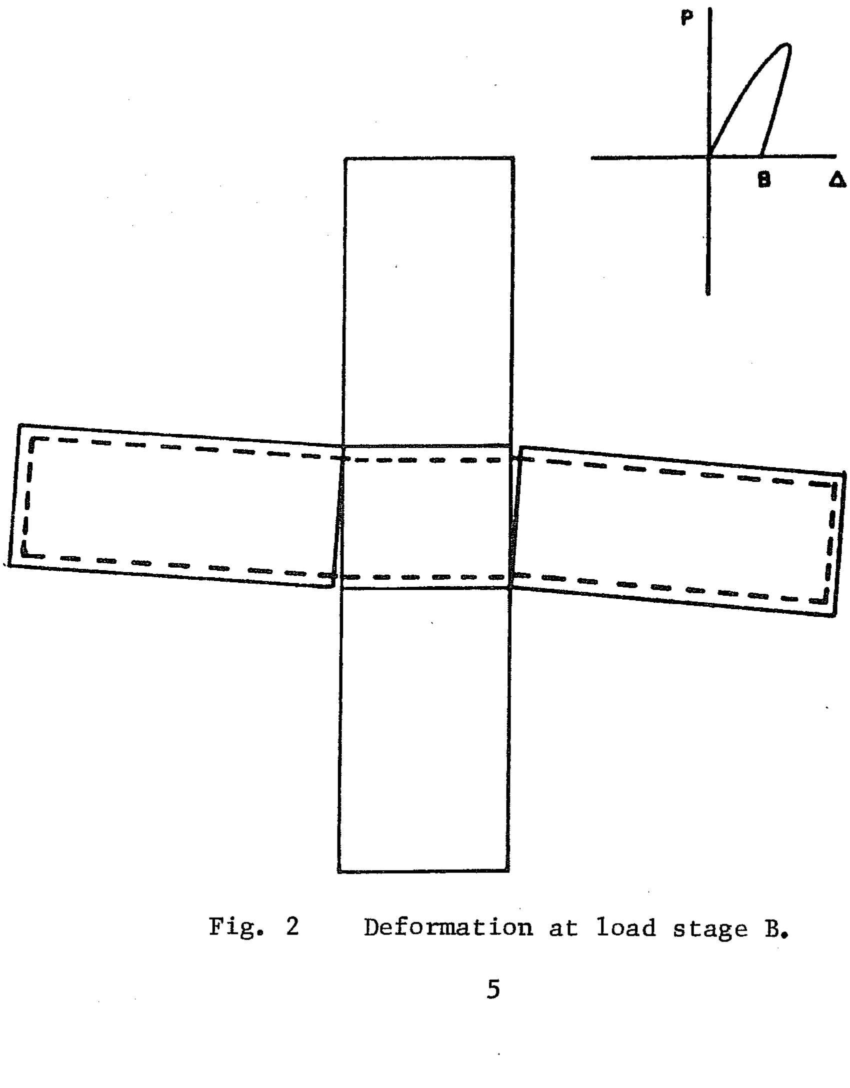


Fig. 2 Deformation at load stage B.

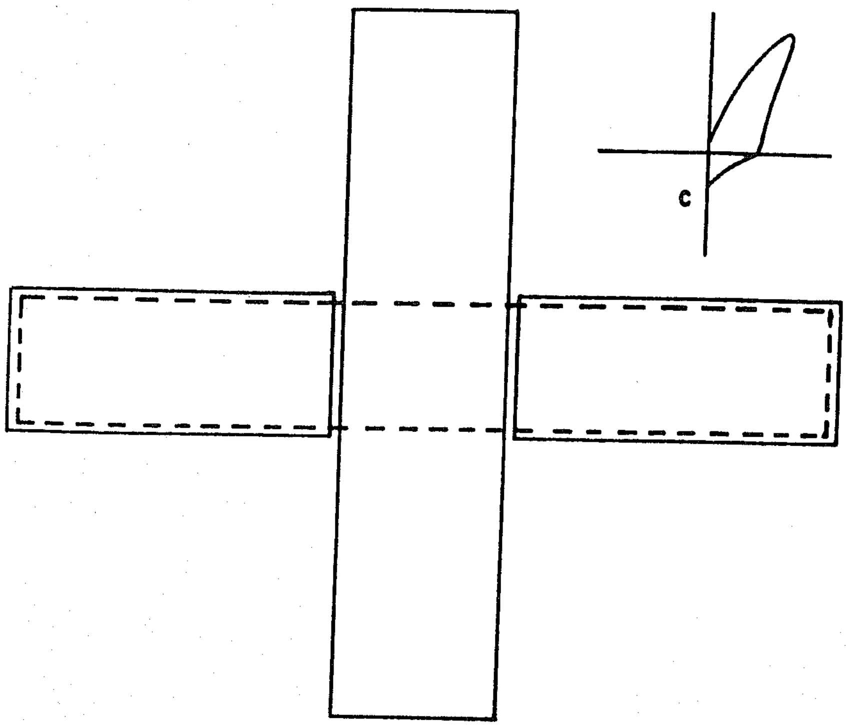


Fig. 3 Deformation at load stage C.

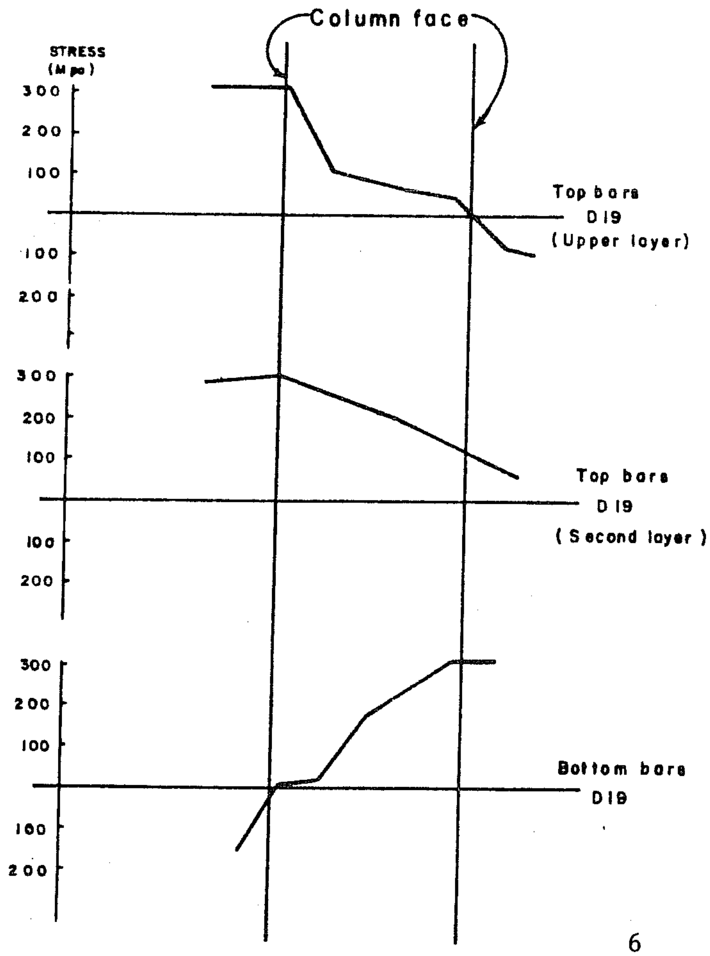


Fig. 4 Beam steel stresses across joint at first yield (Beckingsale, Park and Paulay [12]

When the beam bar is in tension on both sides of the column, the compressive stresses in the concrete will be increased and may cause crushing of the concrete in the beam at the face of the column. Figure 5 shows the equilibrium of forces in the joint and beam and the location of the crushed area in the beam concrete. With crushing of beam concrete, the stiffness of the whole test assemblage is reduced. Measured beam end deflections include the effect of concrete crushing.

Figure 2 shows the deformation at load stage B. Although there is no load acting on the beam end, the cracks between beam and column are not closed. This is the result of beam bar slippage, plastic deformation of the embedded bar and crushing of the concrete compression zone during load stage A. It is hard to assess the contribution of each factor on the beam residual deflection. The crack opening is usually attributed to slippage of the embedded bar. Since the stress and strain situation in the joint area is so complicated, it should be noted that a test procedure using an element smaller than the beam-column joint assemblage must simulate the boundary condition of the real joint. Otherwise, direct comparisons cannot be made.

Figure 3 shows the deformed shape at load stage C. In this case the beam end deflection is zero. There is a small space between the beam and column. Only the top steel and bottom steel connect the beam to the column. The forces in the beam bars can be determined from equilibrium of the forces acting on the beam. Figure 6 shows a freebody of the beam at load stage C.

Taking moments about o, $P = \frac{F_1 h_o}{L_o}$. By summing forces in the horizontal direction, $\Sigma x = 0$, $F_1 = -F_2$. If the area of the top reinforcement is equal to that of bottom reinforcement the stress in the top and bottom steel is the

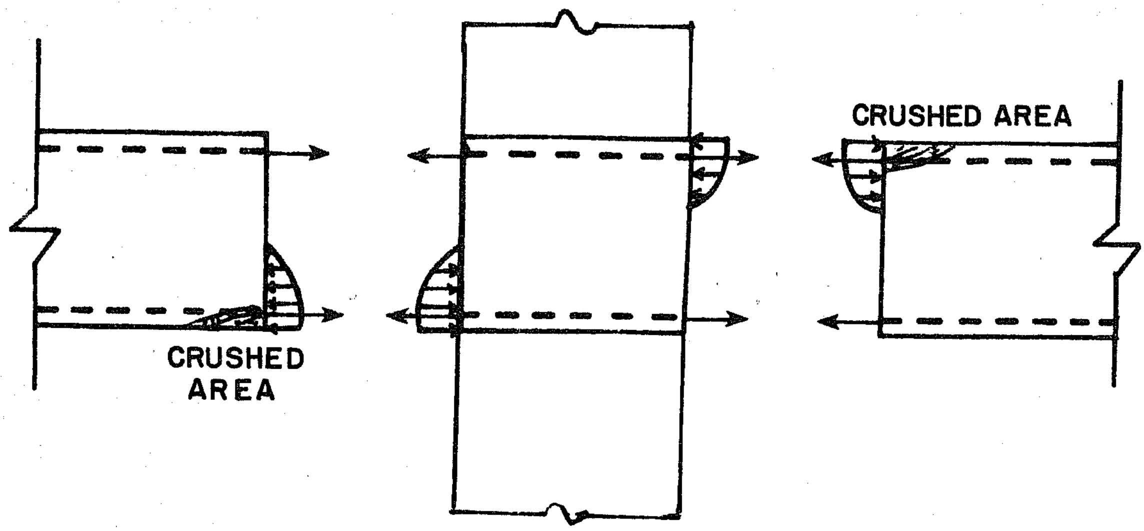


Fig. 5 Equilibrium of forces in the beam and joint.

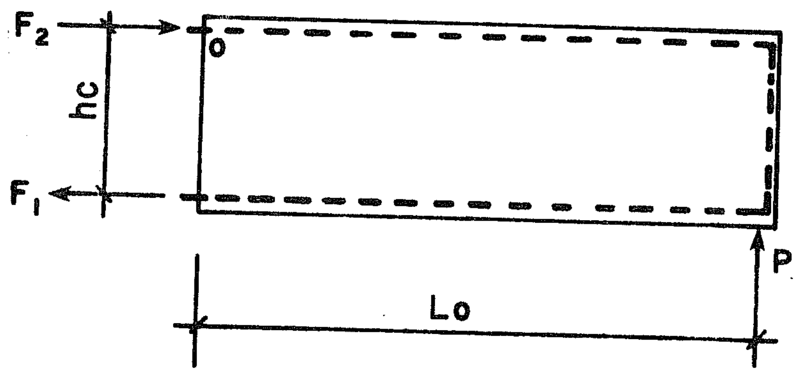


Fig. 6 Equilibrium of forces in beam at load stage C.

same. However, the area of top steel usually is greater than that in the bottom. As a result, the stress and strain in the bottom steel are greater than that in top steel. Because the bottom bars are more highly stressed, they are likely to lose anchorage first. This has been observed in tests at the University of California-Berkeley [13].

Because only two layers of steel connect the beam to the column, the flexural stiffness and shear stiffness of the section is lower than when concrete is carrying some compression. The loss of stiffness leads to pinching of the load deflection curves. After the beam touches the column face the compression force is shifted from the steel to the concrete, and the stiffness of the section is increased. If the joint shear strength is still greater than the joint shear force and the beam bars are still anchored, the hysteretic curve exhibits "hardening" (Fig. 7).

Stage C is critical for bond deterioration because the beam bar is in tension on one side of the column and in compression on the other side (see Fig. 8). Since the beam concrete is not in contact with the column, the compressive force is totally carried by the beam steel. As a result, the stress in the beam steel may be very high. Bond damage usually occurs in this load stage. Under continued load the concrete of the compression zone in the beam eventually touches the column face as shown in Fig. 9.

Once the beam is in contact with the column, beam bar anchorage is no longer as serious a problem. However, there may still be loss of stiffness because the bar is elongating over a considerable length. In this case, if the column width is sufficient the embedded bar can be anchored in the column. If the column width is insufficient the beam bars can be anchored in both

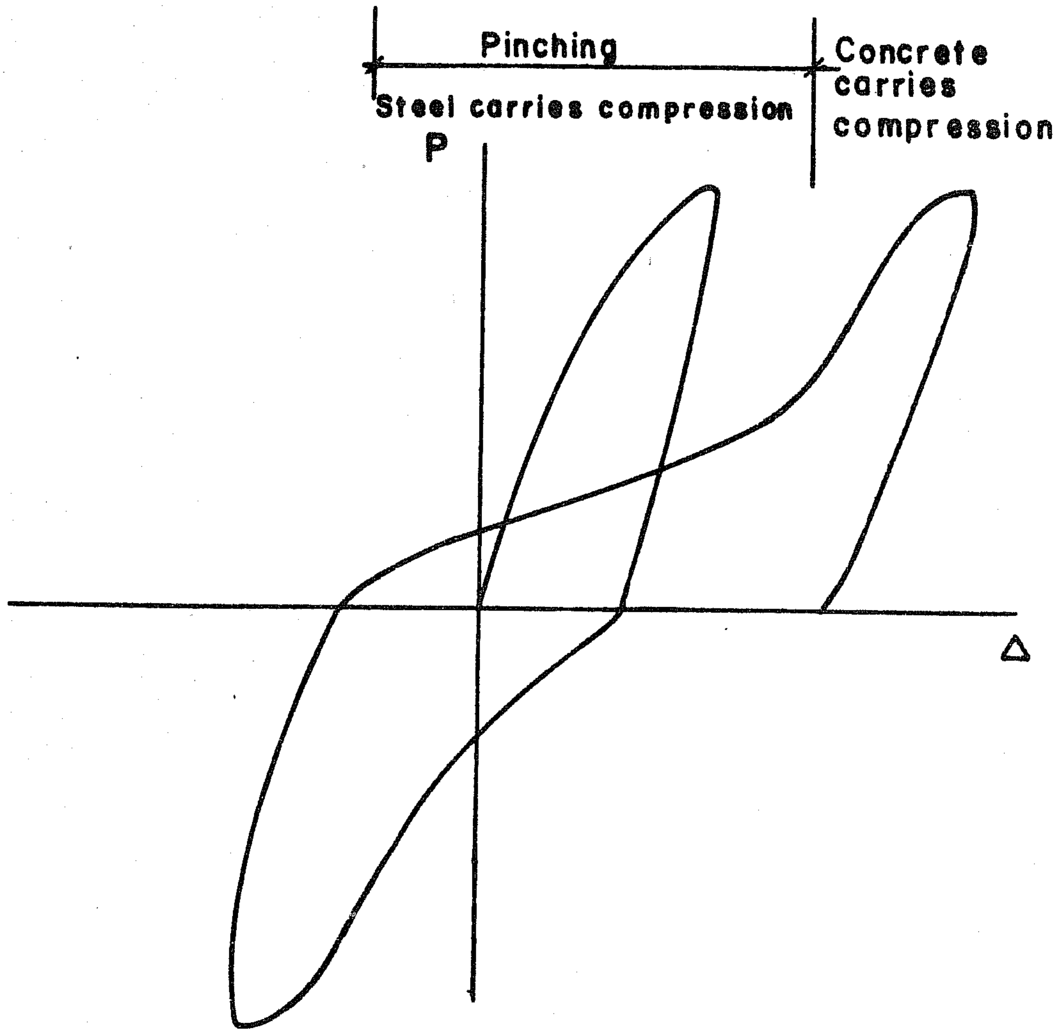


Fig. 7 Pinching of the hysteretic curve and change of slope after the beam touches the column.

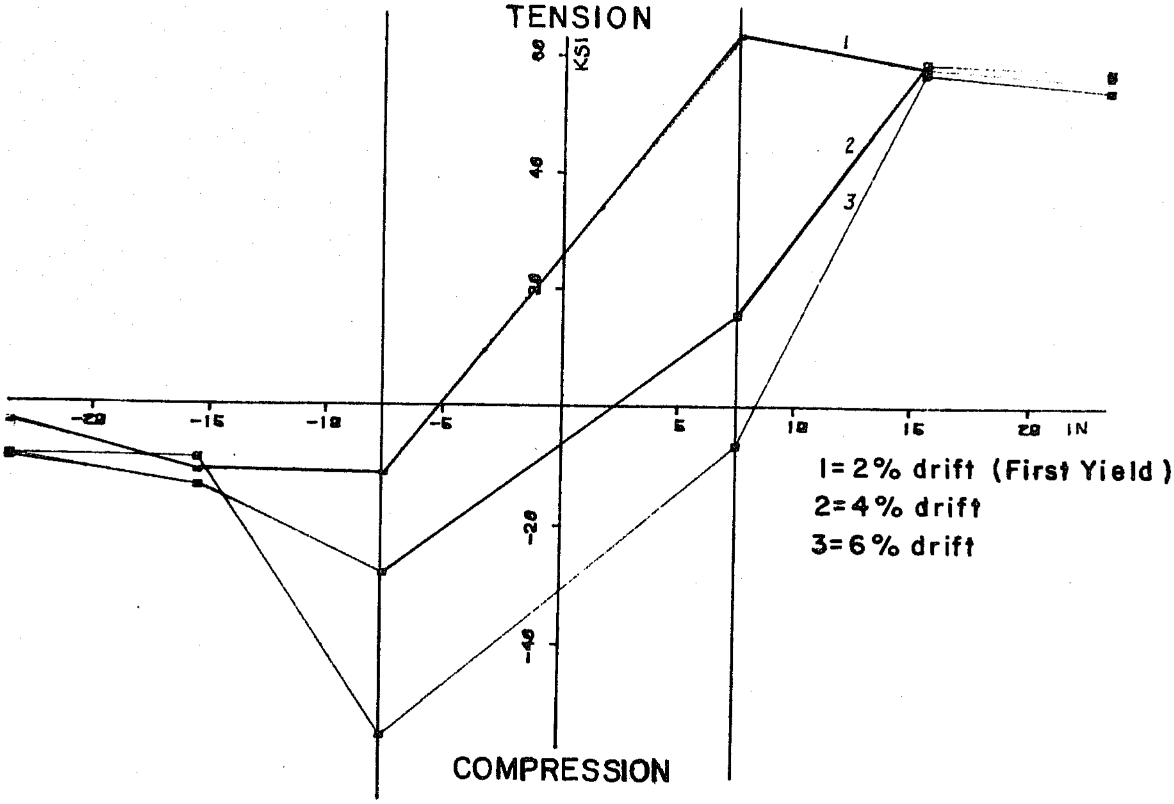


Fig. 8 Stress distribution of #6 bottom steel bar across the joint at different load stages (Leon [26]).

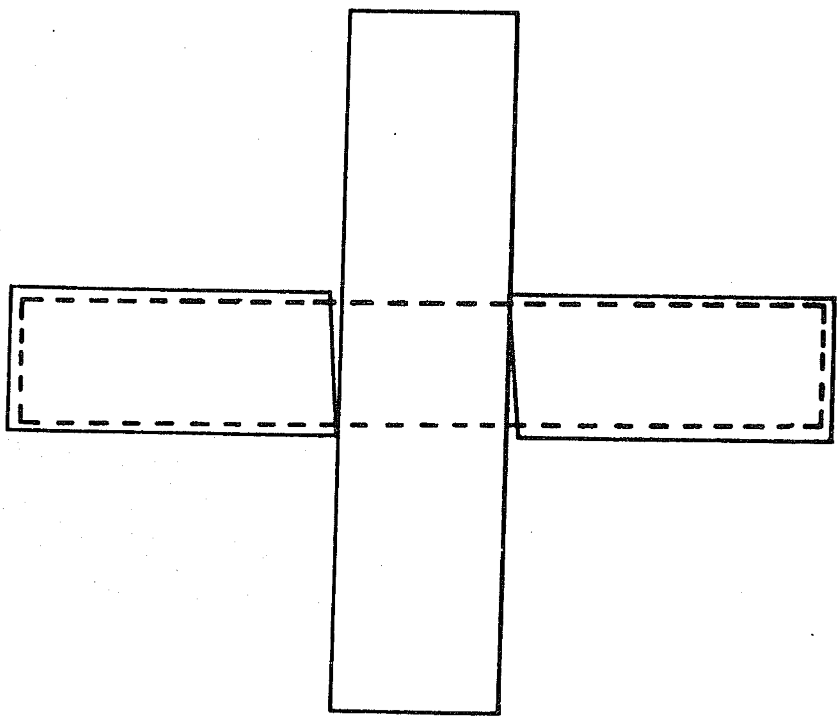


Fig. 9 Beam concrete in contact with the column after load stage C.

column and beam, as in load stage A. If the bond strength is exceeded, bond damage will occur in this load stage. Once bond damage occurs, continued loading will result in the beam compression zone being in contact with the column. With loss of bond, there is stress redistribution between concrete and steel. In this case, the steel will carry tension at both faces of the column. If in a beam column joint test tensile stress at both faces of the column is measured, bond damage has already occurred. This phenomenon was observed and reported in almost all tests [15, 17, 26].

2.2 Crack Pattern in Joint Area

2.2.1 Side Face of the Joint. The forces acting on an interior joint are shown in Fig. 10. The stress situation in the joint area is very complicated. Basically, a compression strut is formed under these forces (see Fig. 11). According to the strut theory, there is compression in the joint in one diagonal direction and tension in the orthogonal direction. Under the reversed load, cracks on the side face of the joint usually are parallel to both of the diagonal lines of the joint as shown in Fig. 12. Cracks in the joint area parallel to the beam bars have not been observed in most tests.

2.2.2 Beam and Column Interface. Test results demonstrate that when beam bars yield, transverse cracks appear not only at the interface between the beam and column but also form at the face of the confined core [15], as shown in Fig. 13. Because the joint core area is confined by the column steel and joint transverse steel, flexural cracking generally is limited to the unconfined concrete. Cracks usually develop along the plane represented by the longitudinal steel in the column, as shown in Fig. 14.

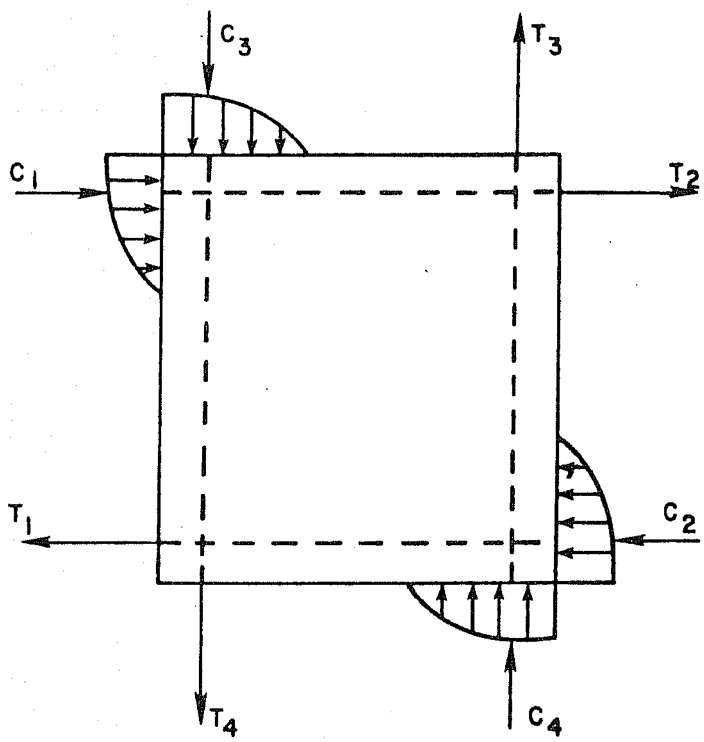


Fig. 10 Force acting on a joint.

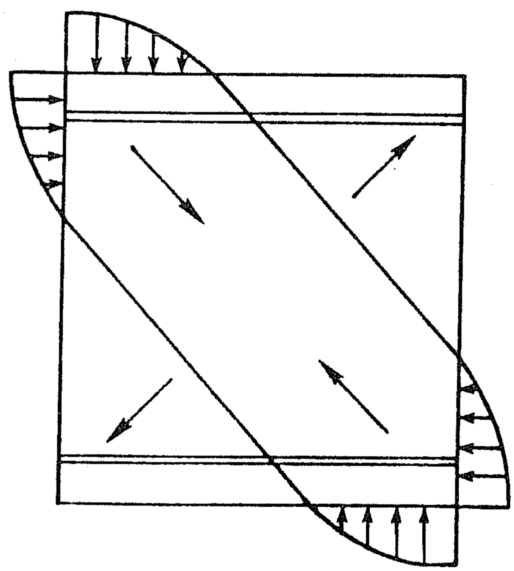


Fig. 11 General outline of the inclined compression strut in a joint.

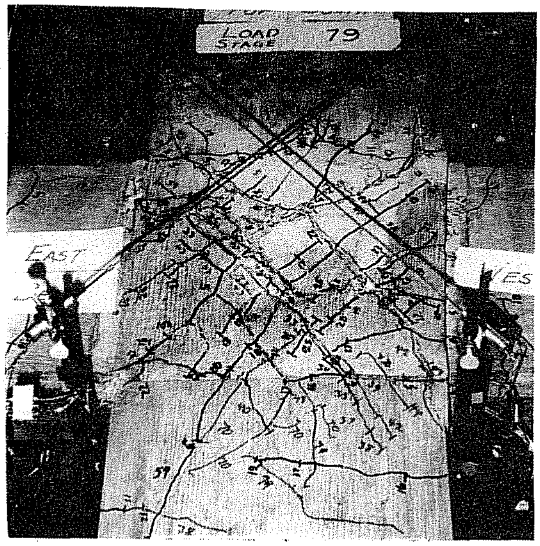


Fig. 12 Joint cracking pattern of specimen V (Meinheit and Jirsa [15]).

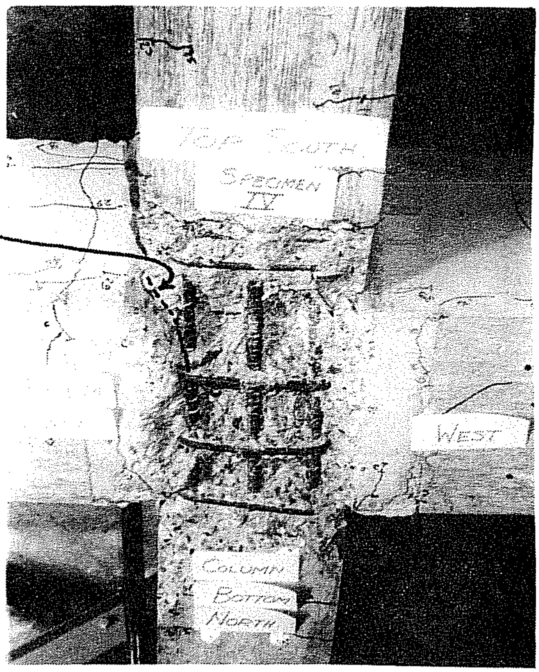


Fig. 13 Main beam flexural cracks opening within the column cross section (Meinheit and Jirsa [15]).

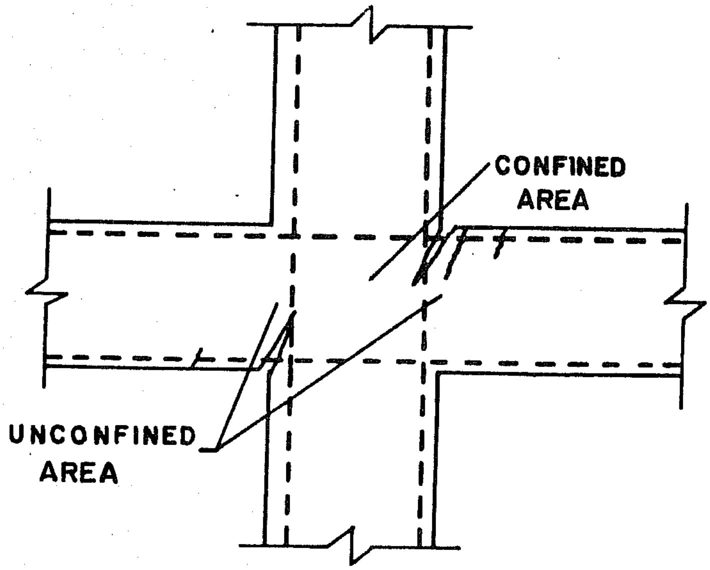
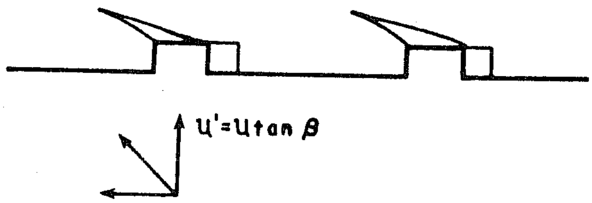
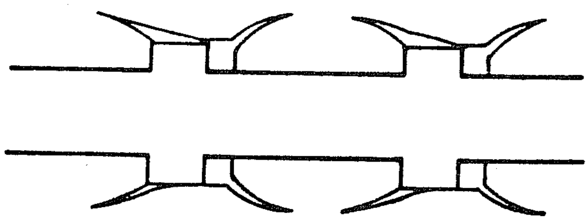


Fig. 14 Confined area and unconfined area in a joint.



a. Tangential and radial component of the force exerted by the lug on the surrounding concrete.



b. Axisymmetric cracks under load reversal.

Fig. 15 Splitting cracks in the concrete surrounding longitudinal bars in the joint.

Because flexural cracks penetrate into the joint, only the confined area of the joint can be considered to be effective for anchorage of the beam bars. The cracks are very close to the column steel and substantially reduce anchorage between the concrete and the column steel; as a result, the column bars slip. Because the strong column-weak beam principle is used in design, similar cracks are not anticipated to occur in the column at the top and bottom of the beam.

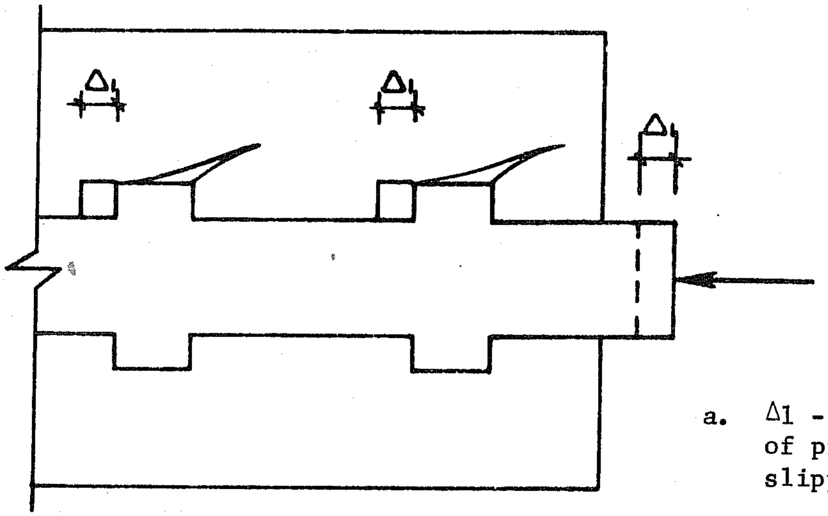
2.2.3 Cracks along the Beam Bar. Stress from a deformed bar is transferred to the concrete mainly by mechanical locking of the lugs with the surrounding concrete. The resultant force exerted by the lug on the concrete is inclined with the axis of the bar as shown in Fig. 15a. This force can be resolved into two components. One is the radial component which causes splitting of the surrounding concrete and the other is the shear force which is parallel to the rebar axis. Because the concrete is well confined in the core area, the splitting failure caused by the radial force is not likely. The main failure mechanism involves shearing the concrete between lugs. Because of load reversals, cracks in the concrete surrounding the beam rebar are axisymmetric about the lugs (see Fig. 15b).

2.3 Mechanism of Slip and Bond Deterioration

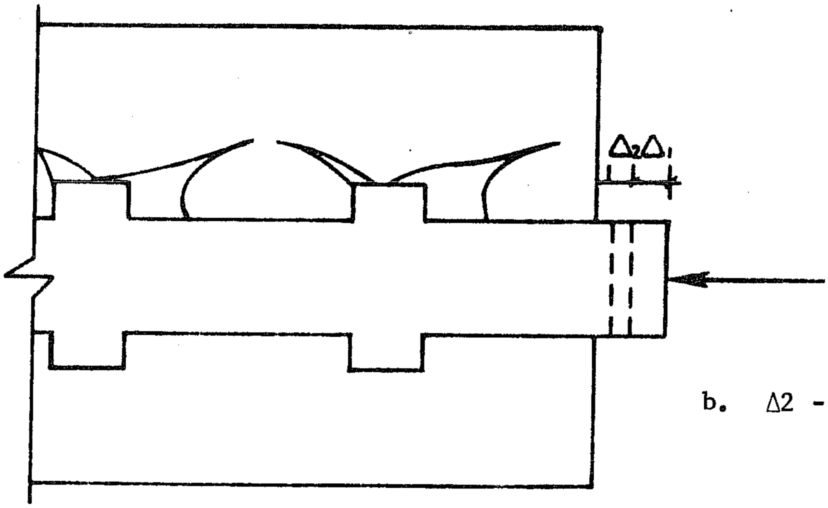
As shown in Fig. 3, there is an open crack between the beam and column in load stage C. The beam steel is in compression on one side and in tension on the other side of the column. Because the beam concrete is not in contact with the column the compressive force in the beam bars may be very large. Under large "push-pull" forces the concrete between the steel lugs is gradually crushed. The bond strength deteriorates and the stiffness of the joint gradually decreases. With continued cycling the concrete at the interface progressively deteriorates and bond failure will finally occur. However, this "push-pull" mechanism exists primarily while the flexural crack is open, before the beam touches the column.

Under continued loading, the concrete compressive zone will eventually touch the column. There are three components related to the elimination of the open crack between the beam and column. First, after reaching the last peak deflection there is a small gap behind the lugs, $\Delta 1$, in Fig. 16a. Under load reversal the beam bar can move through a slip $\Delta 1$, without much resistance. However, if the joint area is in compression due to axial load or due to the development of a diagonal compression strut there may be a friction force between the concrete and the bar.

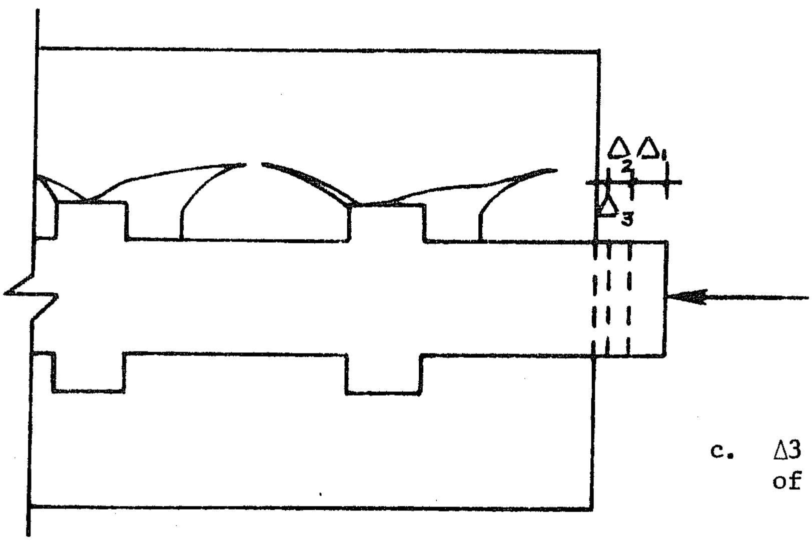
The second component of the movement is the slip of the beam bar, $\Delta 2$, under load reversal, as shown in Fig. 16b. The new slip, $\Delta 2$, is dependent on the loading history and the ratio of bond stress to bond strength. If the bond stress reaches the bond strength significant damage to the anchorage or beam bar pull through could be produced. In order to predict the slip $\Delta 2$ some researchers have attempted to define the bond slip relationship under load reversal [11]. Since the bond slip relationship is dependent on the loading history, it is likely that any relationship obtained from a special test will suit only the loading history under which it was obtained. Another problem is that in real beam column joints the bond stress is changed with the loading history. Before the beam touches the column, the beam bar is in compression at one end and in tension at the other. Bond stress-slip relationships usually are obtained from such deformation histories.



a. $\Delta 1$ - the result of previous slippage.



b. $\Delta 2$ - new slippage.



c. $\Delta 3$ - compression of the beam bar.

Fig. 16 Deformation components producing bar slip.

After the beam touches the column the compressive force in the beam bar is shifted to the concrete, stress redistributes in the joint as does the bond stress. It is difficult to know exactly when the beam comes in contact with the column and to determine the stress redistribution. The bond stress-slip relationship obtained from one case may no longer be useful in a different situation.

The third component is shortening $\Delta 3$ of the embedded bar under the compressive force (Fig. 16c). $\Delta 3$ is related to $\Delta 2$. The higher the ratio of bond strength to bond stress the smaller the slip $\Delta 2$, thus the higher the shortening $\Delta 3$. If this ratio is high, the beam steel may yield at the face of the column under compressive force. In Fig. 17 the beam steel is in compression at one side and in tension at the other and at yields on both sides [12]. The stresses in Fig. 17 are for a test with small bars, large column depth and low steel strength (Grade 40). However, for Grade 60 steel, recorded steel stress in most tests shows that steel compressive stress is usually below the yield stress. Before the steel reaches the compressive yield stress, the beam compressive zone touches the column. Stress redistribution occurs. This situation could be seen in Fig. 8. According to the above analysis, the maximum bond stress along the embedded bar would occur under the condition shown in Fig. 18. According to a review of crack distribution, only the confined area could be considered effective for anchorages. If the distance between outer layers of column steel is H_c and the diameter of the beam bar is d, the average bond stress μ is calculated as follows:

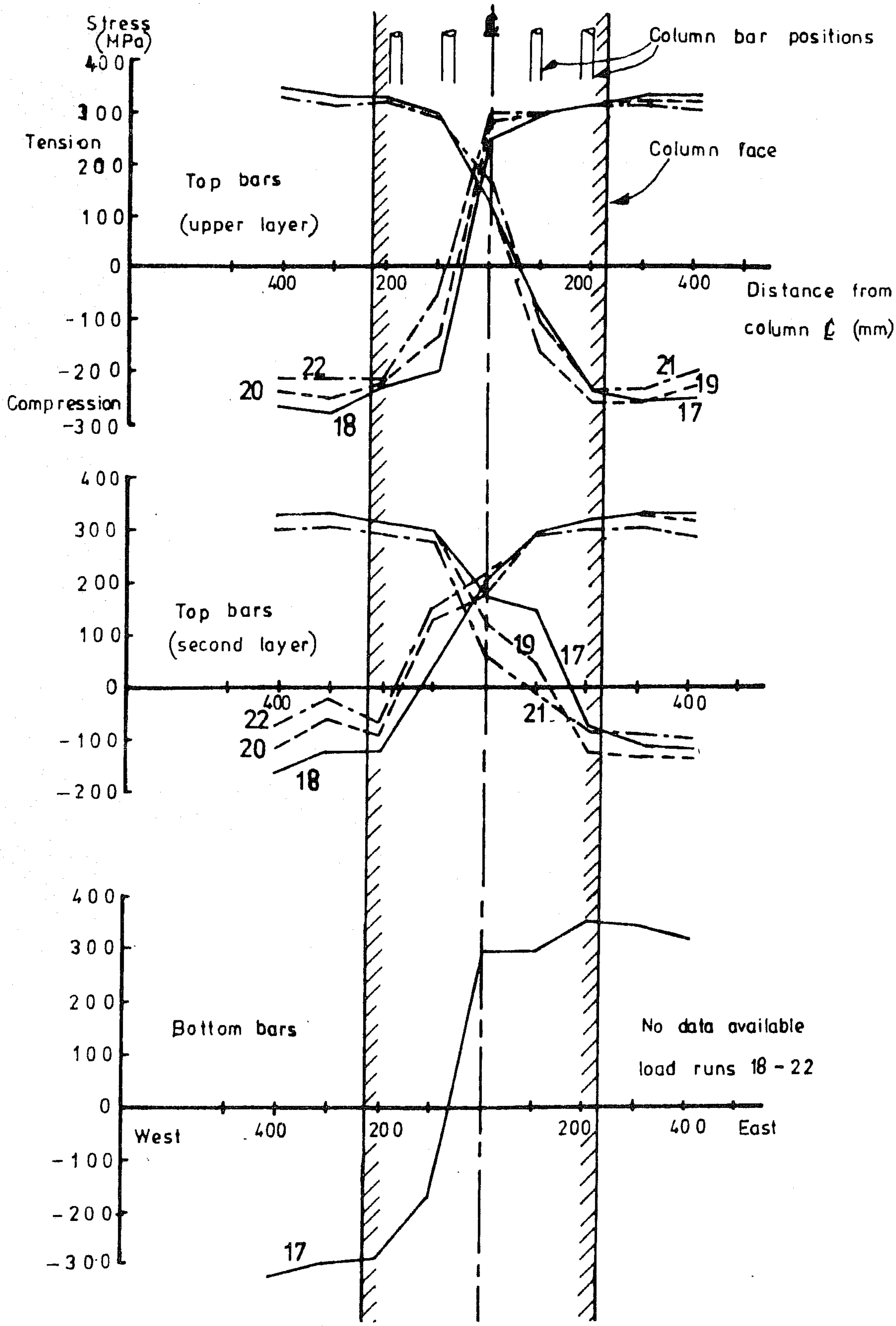


Fig. 17 Beam steel stresses across joint (Beckingsale and Park [12]).

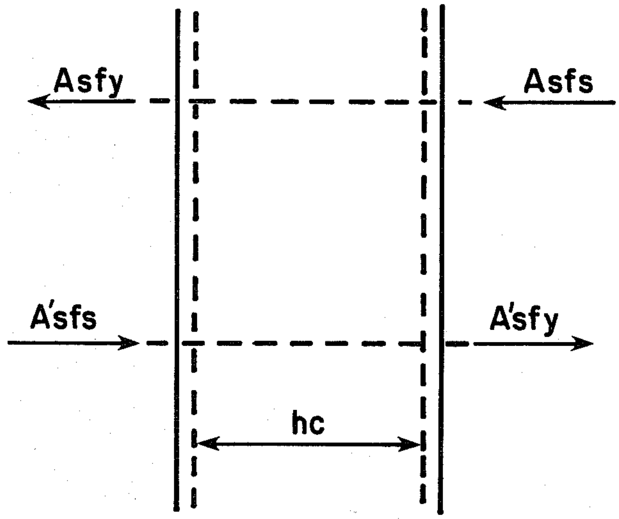


Fig. 18 Bar forces producing maximum bond stress at load stage C.

$$\mu = \frac{A_s f_y + A_s f_s}{\pi d H_c} \tag{1}$$

Bond damage will only occur during this load stage if the bond stress exceeds the bond strength. However, bond strength is not a constant under cyclic loading. Under cyclic "push-pull" loading the concrete between steel lugs is progressively crushed. A soft layer of concrete is formed close to the steel bar. It causes deterioration of the bond strength between the steel and concrete. The maximum external force remained either unchanged or decreased with increase in number of cycles. Therefore, the inelastic deformation history of the beam steel caused deterioration of the bond strength. As far as bond deterioration is concerned imposed inelastic deformation history is more important than imposed force. Because of the uncertainty of the bond strength under reversed load it is impractical to use bond strength to set the required embedment length of the beam steel in the column confined area.

Actually, bond damage can occur regardless of anchorage length provided that force and deformation are sufficient. The problem is that the imposed force and deformation history should be representative of the conditions to be expected during an earthquake. Under reasonable force and deformation history, bond deterioration will occur, but bond damage can be avoided. Since the stress and bond conditions of steel bars in a beam column joint are very complicated, the bond situation in a beam must be reevaluated for each beam column joint test directly.

C H A P T E R   I I I

REVIEW OF PREVIOUS RESEARCH

3.1 Comments on Current Beam Column Test Programs

To understand the behavior of the joint and for economy, nearly all test programs have utilized an isolated beam column joint, i.e., all members cantilever out from the joint. In most cases, the columns were pin supported at the ends and the beams were loaded vertically. In a few programs, the beams were pin supported at the ends and the columns were loaded laterally. Almost all the design recommendations and code requirements for beam-column joints are based on the results of such tests. Most beam column joint test specimens have been subjected to displacements which could be considered equivalent to an interstory drift exceeding 4 to 5% of the floor height. Under these large deformations, the joint has frequently been reported to be inadequate and the weakest component of a frame system. In nearly all the tests bond deterioration was observed with the bars pulling through the joint after large beam end deflections were reached. However, it is somewhat paradoxical that joint distress in frame structures has not been reported as a major factor in structural failure in recent destructive earthquakes. Therefore, a question has arisen as to the credibility of the deformation histories used in test programs. If the performance of the isolated joint is evaluated in terms of a maximum "credible" beam deflection, strength and stiffness degradation problems are eased considerably and the requirements for shear and anchorage found in current design recommendations might be eased without sacrificing overall structural performance [21].

The general philosophy of earthquake resistant design is 1) to prevent nonstructural damage in minor earthquakes which may occur at intervals in the service life of the structure, 2) to prevent structural damage and minimize nonstructural damage in moderate earthquakes, and 3) to avoid collapse or serious damage in a major earthquake which may occur rarely [22]. The most important factor influencing the level of damage is the level of deformation. To prevent nonstructural and structural damage many building codes and recommendations stipulate a permissible interfloor response drift as a design criterion. Table 1 shows the interstory drift limitations from some codes and recommendations [22, 23, 24, 25]. It should be noted that maximum nonlinear displacement is less than 2%. Drift limitations reflect engineering judgment based on available experience and experimental data. At drift values where nonlinear response is expected, severe damage to nonstructural elements is likely; however, the structure may experience only minor damage.

The frame should have the capacity to survive even large deformation but such deformation may be accompanied by severe damage to some elements and local collapse is possible. When a moment resisting frame is designed and calculated interstory drift is larger than the code drift limitation, the lateral stiffness of the frame structure is considered to be insufficient and a redesign is required. As a result, the interstory drift of a well designed frame structure under a "credible" earthquake is likely to be in the vicinity of 0.015. In general, test requirements should exceed design requirements. However, it has not been possible to define a value of the maximum beam end deflection or drift at which to evaluate the test results of the beam column joints. For this study, a drift of 0.03 was selected to evaluate beam column

TABLE 1. CODE INTERSTORY DRIFT LIMITATION*

	1976 UBC	Mexico	Japan	NBC	N.A.
Linear response displacement	0.005	0.008	0.003	0.005	0.006
Nonlinear response displacement	0.01	0.016	0.01	0.015	0.01

*Drift Angle $= \frac{\text{lateral deflection}}{\text{story height}}$

joint response. If a beam-column joint assemblage can survive drift of 0.03 without substantial loss of strength and bond damage, the detailing of the beam column joints should be satisfactory. Using this criterion, test results obtained by different researchers have been reevaluated.

3.2 Modeling Beam Column Joints

A substantial difference between the test joint and the real frame structure is that in a test only the beam and joint absorbs and dissipates energy. In a real structure not only the beam and joint but also the slab and lateral beam (through torsion) absorbs and dissipates the input energy. When beam bars yield, the stiffness of the beam section drops rapidly. In this case, moment redistribution between beam and slab occurs. In design of frame structures under lateral load, only part of the slab is considered effective in resisting the lateral load. The "equivalent slab" method is based on the results of elastic analysis. However, when the frame experiences large inelastic deformations, not only the "equivalent" slab but the entire slab may absorb and dissipate energy. Therefore, the real structure can experience large inelastic deformation without substantial loss of strength and bond damage as was observed in joint tests with slab and lateral beams [17].

The isolated beam-column test specimen is very sensitive to the deformation history. Because the continuous structure can redistribute forces to less severely stressed regions, the beam column joint in a real structure is not as sensitive to the deformation history as a bare beam column joint. Isolated beam column joint test results are generally conservative. If an isolated test assemblage can withstand a certain level of inelastic deformation reversals, say to a drift of 0.03, the real framed structure with lateral beams and floor slab should exhibit better performance than the test assemblage under the same deformation history.

3.3 University of Michigan

Durrani and Wight [17] studied the effect of the level of shear stress in the joint and the effect of confinement of the joint on the performance of the beam-column subassemblies. To achieve these objectives, six interior beam column subassemblages (Fig. 19, 20) were tested. Three of the specimens had lateral beams and a slab to study the effect of confinement as in a real structure. The other three specimens were bare connections. The other parameters were the amount of joint reinforcement (ρ_t = 0.75% to 1.15%) and the magnitude of joint shear stress. The beam bar anchorage situation in the joint area was also reported. The total height of the specimen between supports is 88.5 in., so 3% of the floor height represents lateral deflection of 2.65 in., 4% represents 3.54 in. The details of the six test assemblages are listed in Table 2. The hysteretic curves for these six assemblages are shown in Figs. 21 through 26.

Specimen XI (Fig. 21) failed in joint shear and bond damage. Bond deterioration and beam bar pull through were observed during the test. However, when the specimen experienced column end displacement equal to 0.03 of the floor height no substantial loss of strength nor bond damage was observed. When the specimen experienced displacement equal to 0.04 of the floor height, 10% loss of strength was observed. At floor drift equal to 0.05 of the floor height, the loss of strength was about 25%. Even though the joint finally failed in shear and bond, the joint was performing well at drifts of 0.03 without substantial loss of strength and bond damage.

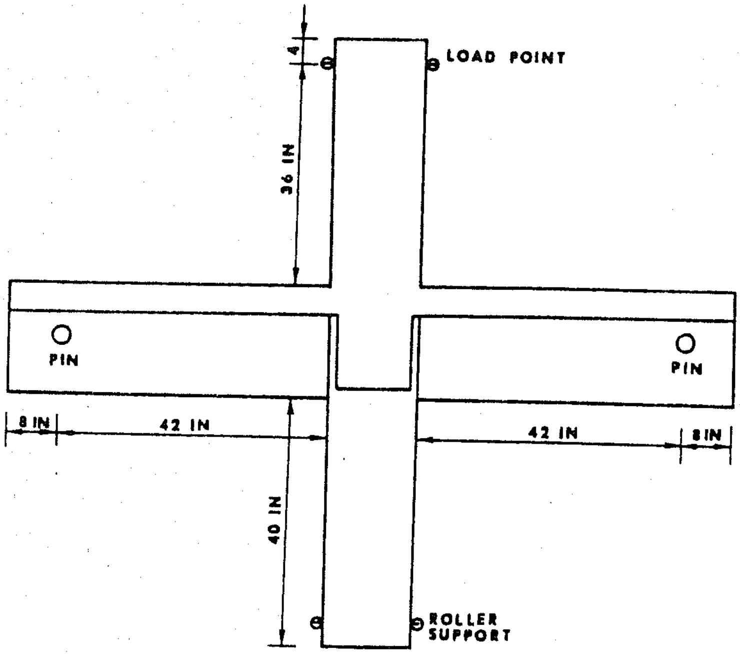


Fig. 19 Overall dimensions of a specimen in the University of Michigan [17].

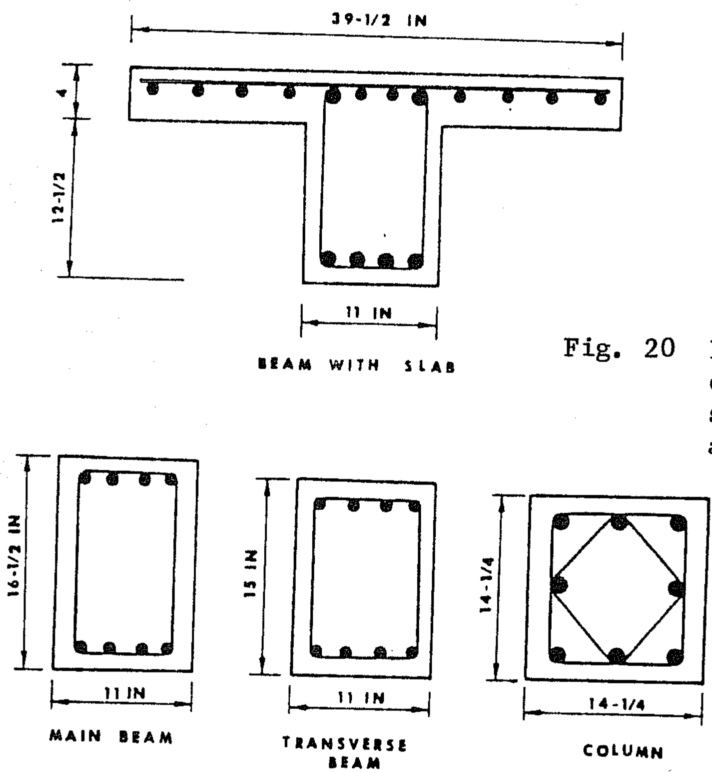


Fig. 20 Beam, slab and column sections in specimen X1 - X3 and S1 - S3 [17].

TABLE 2. DETAILS OF THE UNIVERSITY OF MICHIGAN TESTS [17]

Specimen	Moment Ratio	Joint Reinf. %	Joint Shear Stress Coeff. α	Slab	f'_c, psi
X1	1.25	0.76	13.2	no	4980
X2	1.37	1.15	13.5	no	4880
X3	1.22	0.76	10.4	no	4500
S1	1.22	0.76	13.2	yes	6030
S2	1.21	1.15	15.3	yes	4460
S3	1.32	0.76	12.5	yes	4100

$$\text{Moment Ratio} = \frac{\Sigma M_{cols}}{\Sigma M_{beams}}$$

$$\alpha = \frac{V_{joint}}{(bh)_{col} \times \sqrt{f'_c}}$$

TABLE 3. MEASURED BEAM BAR STRAINS--U. OF MICHIGAN TESTS [17]

Specimen	Position of Beam Bar	Column End Deflection in.	Equivalent Interfloor Drift %	Measured Strains in Beam Bar %
X1	bottom	3.3	3.7	1.8
X2	bottom	2.7	3.0	1.8
X3	bottom	2.5	2.8	2.1
S1	bottom	3.2	3.6	2.2
S2	top	2.7	3.0	1.6
S3	top	2.7	3.0	1.7

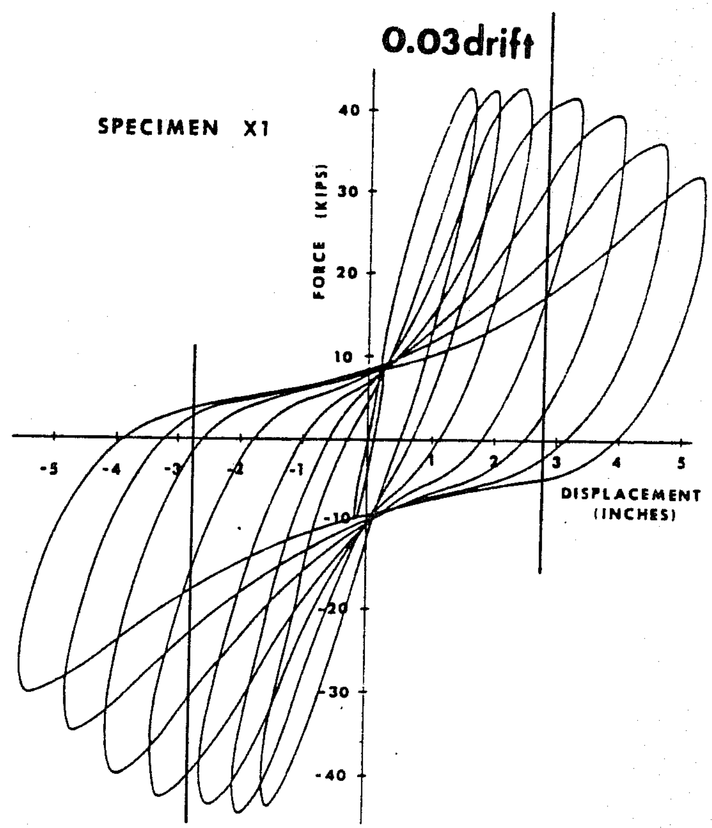


Fig. 21 Column load vs. column load point displacement hysteresis curves for specimen X1 [17].

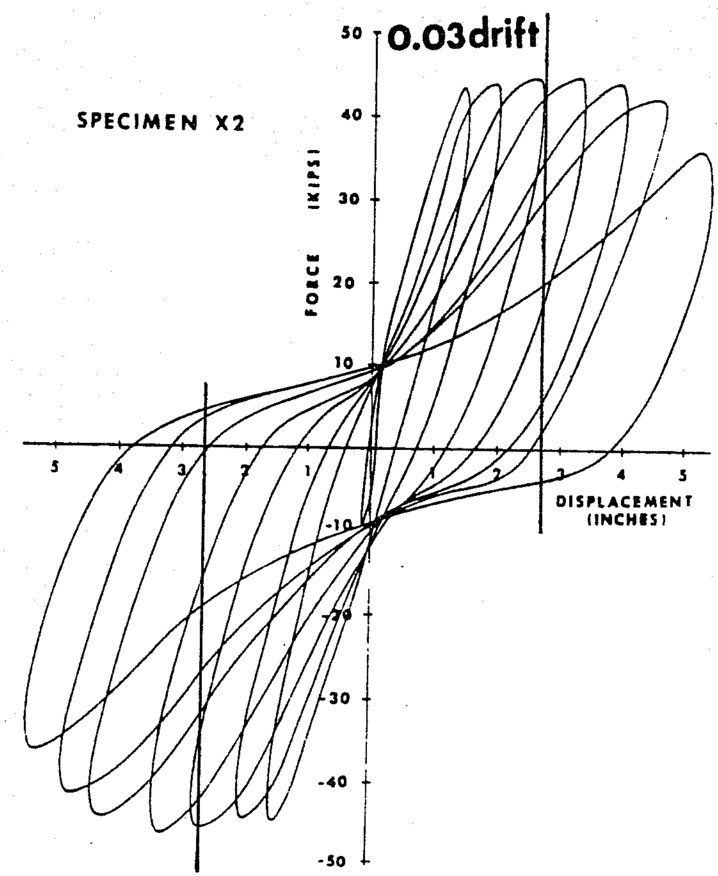


Fig. 22 Column load vs. column load point displacement hysteresis curves for specimen X2 [17].

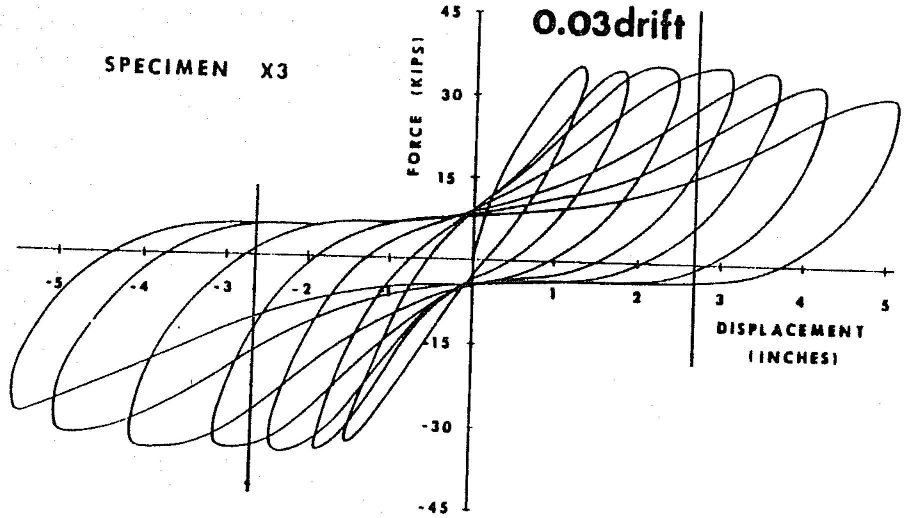


Fig. 23 Column load vs. column load point displacement hysteresis curves for specimen X3 [17].

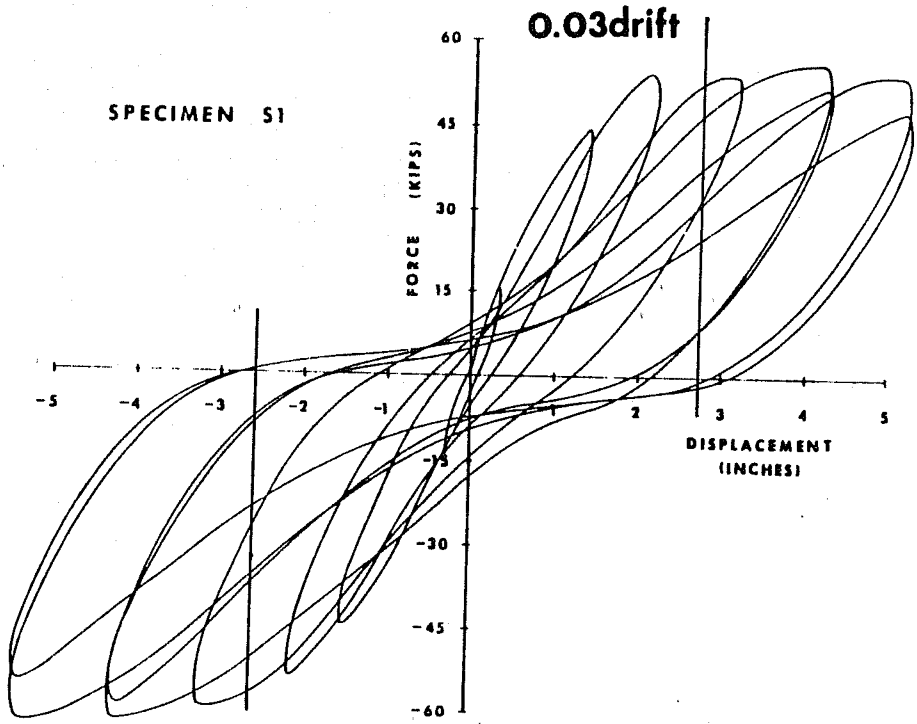


Fig. 24 Column load vs. column load point displacement hysteresis curves for specimen S1 [17].

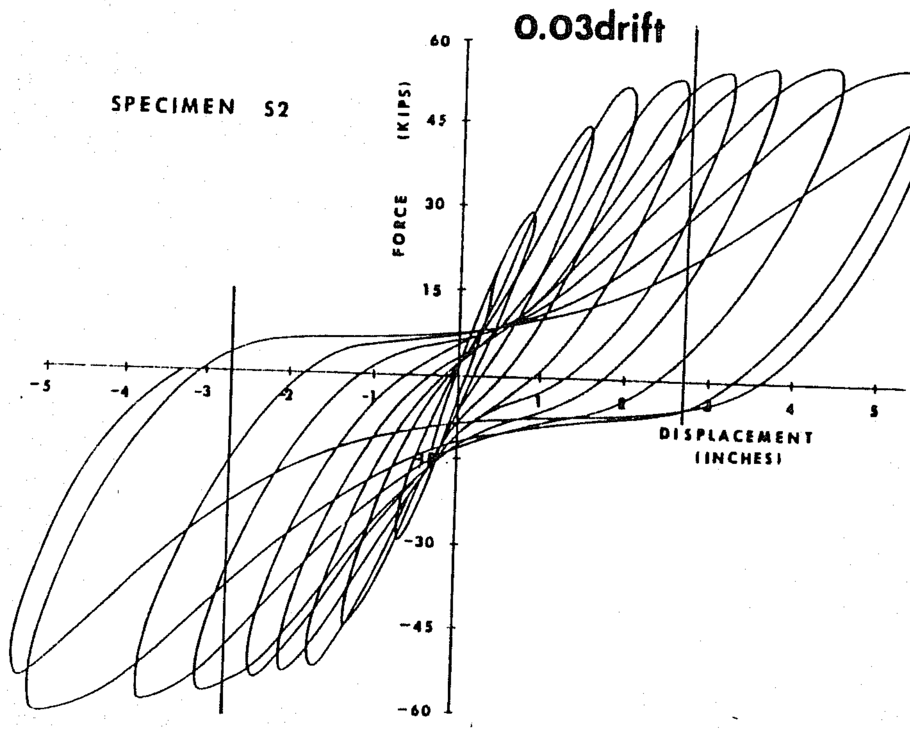


Fig. 25 Column load vs. column load point displacement hysteresis curves for specimen S2 [17].

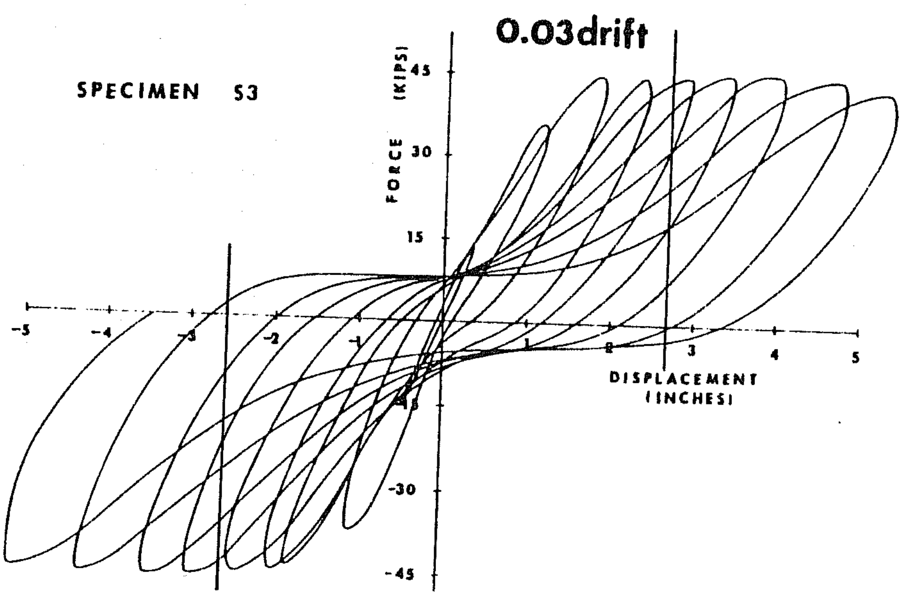


Fig. 26 Column load vs. column load point displacement hysteresis curves for specimen S3 [17].

Because of the higher percentage of joint transverse loop reinforcement in specimen X2 (Fig. 22), relatively less damage occurred in the joint than in Specimen X1. The specimen failed in beam hinging. Only minor slip of beam bars was observed and beam bar pull through did not occur. When Specimen X2 experienced displacement equal to 0.03 or 0.04 of the floor height no substantial loss of strength and bond damage were observed. When the specimen experienced drift of 0.05, substantial loss of strength, about 20%, had already occurred.

In Specimen X2, 4-#7 top beam bars and 4-#6 bottom beam bars were used. Test results demonstrate that at a drift of 0.03 or 0.04 bond damage did not occur.

Comparison of specimens X1 and X2 demonstrates that the beam bar anchorage situation is related to joint distress to some extent. The lower the amount of transverse confining reinforcement the more likely the occurrence of bond damage in the joint.

Specimen X3 (Fig. 23) had a lower shear stress and less joint transverse reinforcement. This specimen failed in shear and bond deterioration. However, at a drift of 0.03 or 0.04 no substantial loss of strength and bond damage was observed. Only at a drift of 0.05, bond damage was observed and a 10% loss of strength was recorded.

With slab and lateral beam the S-series specimens did not have a well defined yield point. The slab bars became increasingly effective with every additional cycle and the load carrying capacity increased accordingly. From the hysteretic curves shown in Figs. 24, 25, and 26, it is clear that the specimen with slab and lateral beams did not show any loss of strength up to lateral deformations equal to 0.05 of the floor height. Gradual bond deterioration occurred during the cyclic loading.

Comparing X series specimens with S series specimens, it is clear that slab and lateral beams improved the joint behavior. By restricting the crack width, beam bar anchorage improved. The measured beam bar strains closest to the column face are listed in Table 3 for specimens X1 to S3.

The test results demonstrate that frame structures need to be divided into two categories, with slab or without slab. In the case of the frame with slab, the bond problem is not as serious as some researchers have indicated. However, attention should be paid to the anchorage of slab steel. In the case of a frame without a slab, the bond problem is not a serious one if the drift is less than 0.03.

3.4 New Zealand

Many beam column joints have been tested in New Zealand. In this section only three beam-column joint test units representing interior plane frame joints are discussed [12]. In all the test assemblages Grade 40 steel was used. The geometry of the test units is shown in Figs. 27 and 28. The test units were all loaded at the beam ends. According to the geometric relationship shown in Fig. 29, the equivalent interstory drift Δc is

$$\Delta c = (\Delta B_1 + \Delta B_2)\, \frac{\ell_c}{\ell_B} \qquad (2)$$

The beam end deflections were equal, and the spans c = 3354 mm, B = 2438 mm. Therefore,

$$\Delta c = \frac{2\ell_c}{\ell_B} \quad B = 1.375\Delta_B$$

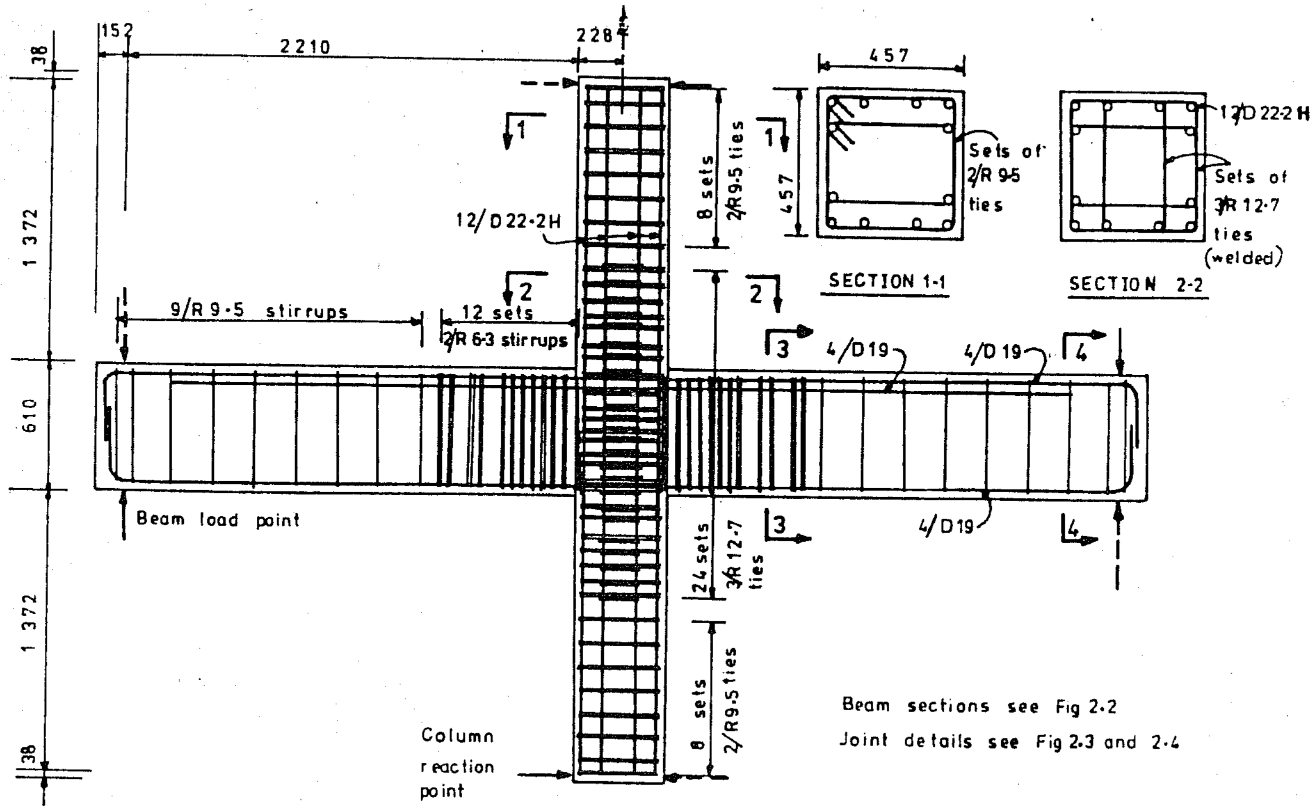


Fig. 27 Test unit, University of Canterbury [12].

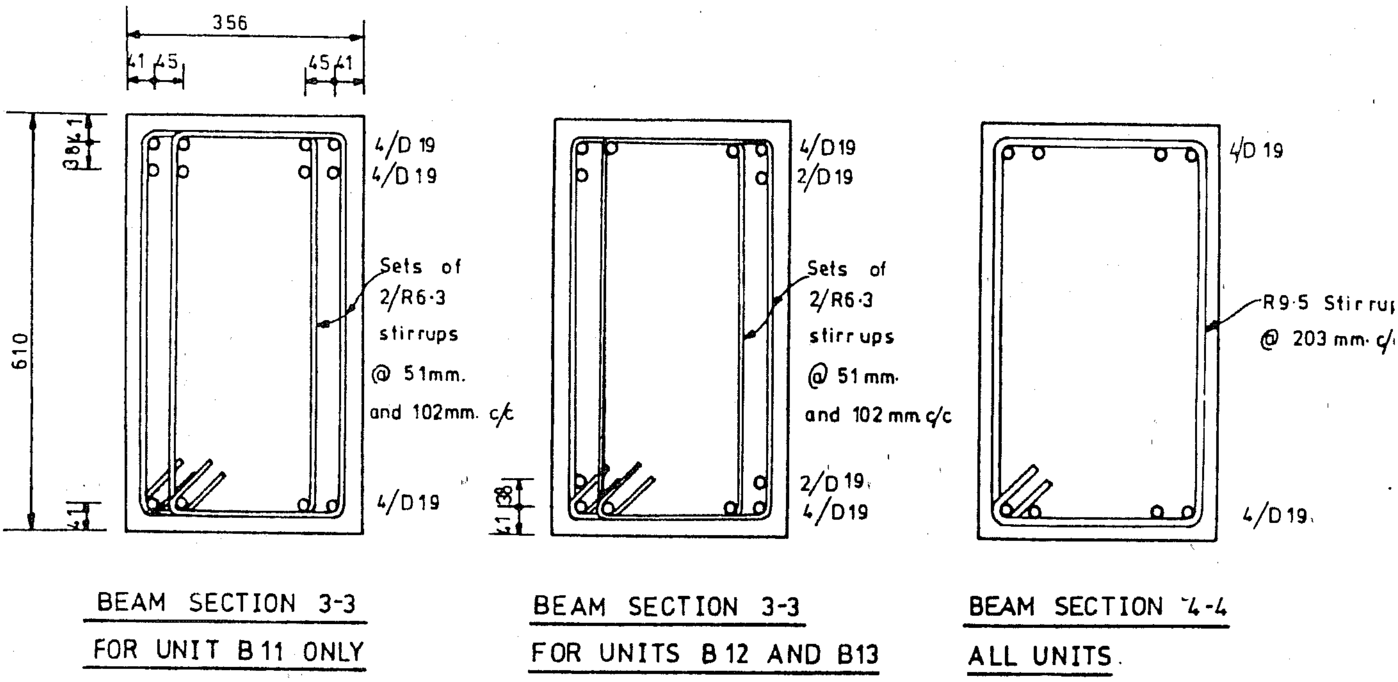


Fig. 28 Beam details, specimen B11, B12, B13 [12].

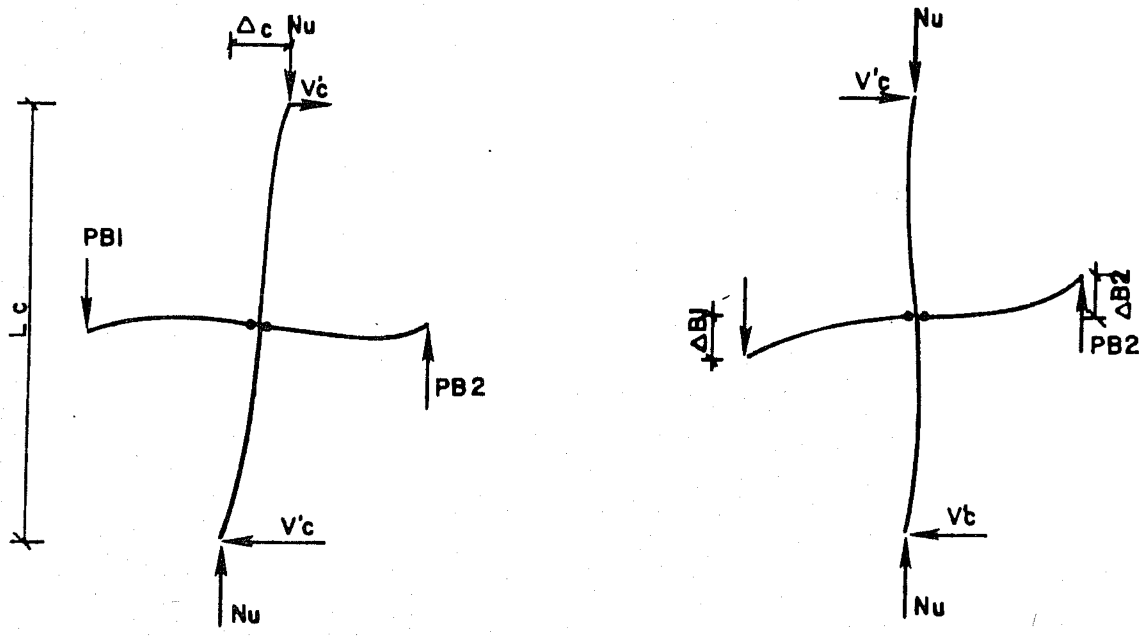


Fig. 29 Relationship between beam end deflection and column end displacement.

TABLE 4. MEASURED BEAM BAR STRAINS IN SPECIMENS B11, B12, B13, U. OF CANTERBURY [12]

Specimen	Position of Beam Bars	Beam End Deflection in.	Equivalent Interstory Drift %	Measured Strains in Beam Bars %
B11	top	2.36	2.5	2.0
B11	bottom	2.36	2.5	2.4
B11	top	3.35	3.5	2.7
B11	bottom	3.35	3.5	3.5
B12	top	2.36	2.5	2.4
B12	top	3.35	3.5	3.0
B13	bottom	2.36	2.5	2.1
B13	top	3.35	3.5	3.2
B13	bottom	3.35	3.5	3.1

Because specimens were reinforced with Grade 40 steel and had low steel ratios in the beam sections, the beam steel yielded at relatively low beam end displacements. Although the beam end reached deflections equal to a ductility ratio $\mu = 4$, the total beam end displacement was equivalent to a story drift of less than 3%. It is clear that the ductility factor alone may not be enough to evaluate the performance of joints in a continuous structure. Ductility is influenced by many factors, including geometry of the joint, steel ratio in the beam, and material properties. Beams may reach a given ductility factor, but it is possible that the displacement may be less than that corresponding to a required interstory drift. It can be seen that interstory drift is a more reasonable criterion for evaluating the performance of beam-column joints.

The hysteretic curves for specimens B11, B12, and B13 are shown in Figs. 30, 31, and 32. For specimen B11 (Fig. 30) at a beam end deflection equivalent to story drift of 0.03, the beam load was about 20% greater than calculated ultimate. No significant pinching was observed prior to cycles at about 3.8% drift. At this point it was reported that slip of bottom bars through the joint commenced. However, the strength of the joint did not drop. A comment in Ref. 12 concerns the loss of anchorage:

> The effect of the slip failure on the response of the test unit was severe insofar as stiffness and energy dissipation capacity was concerned. However, having regard of the fact that it occurred only after the unit had successfully withstood quite severe cyclic loading, and that it did not cause brittle failure nor serious loss of load carrying ability at maximum displacement, it is felt that its significance should not be overrated.

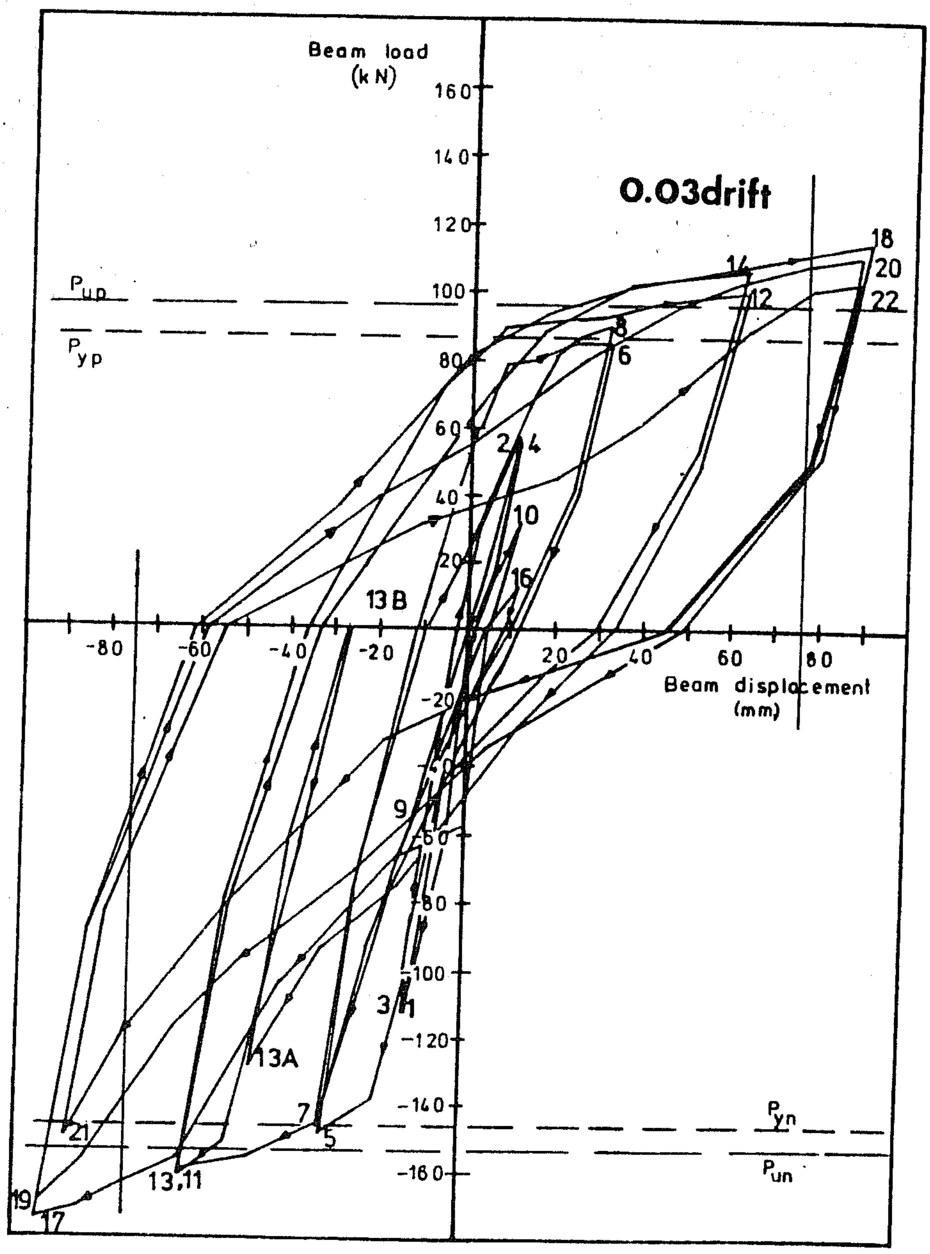


Fig. 30 Load-displacement response, western beam [12].

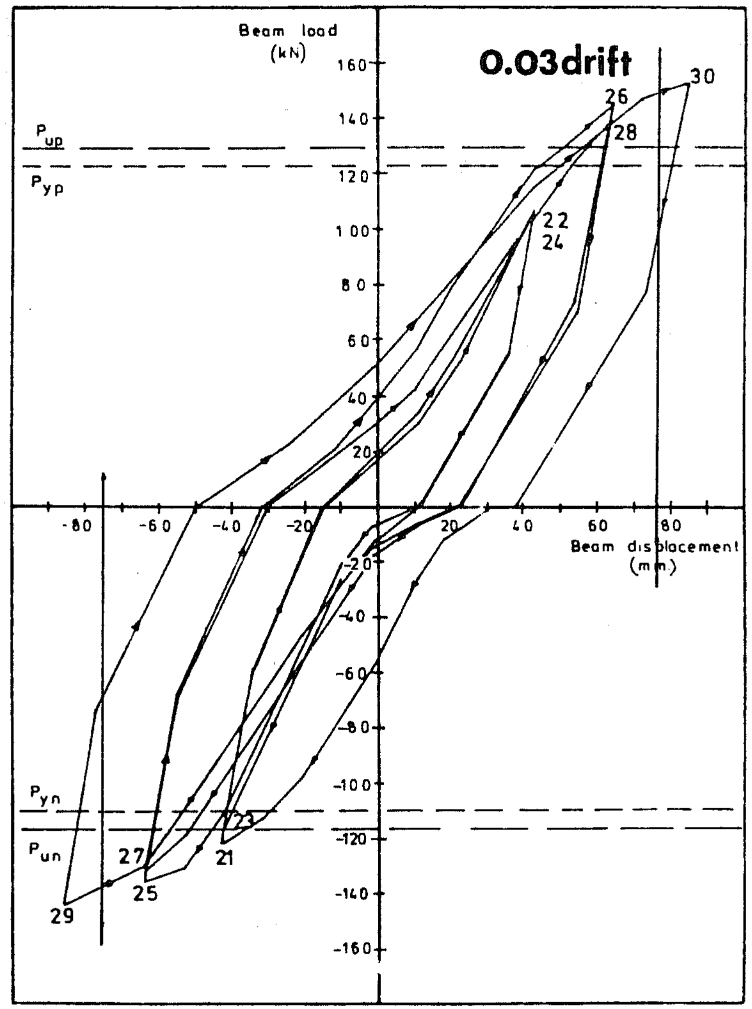


Fig. 31 Beam load-displacement response--
western beam test B13B [12].

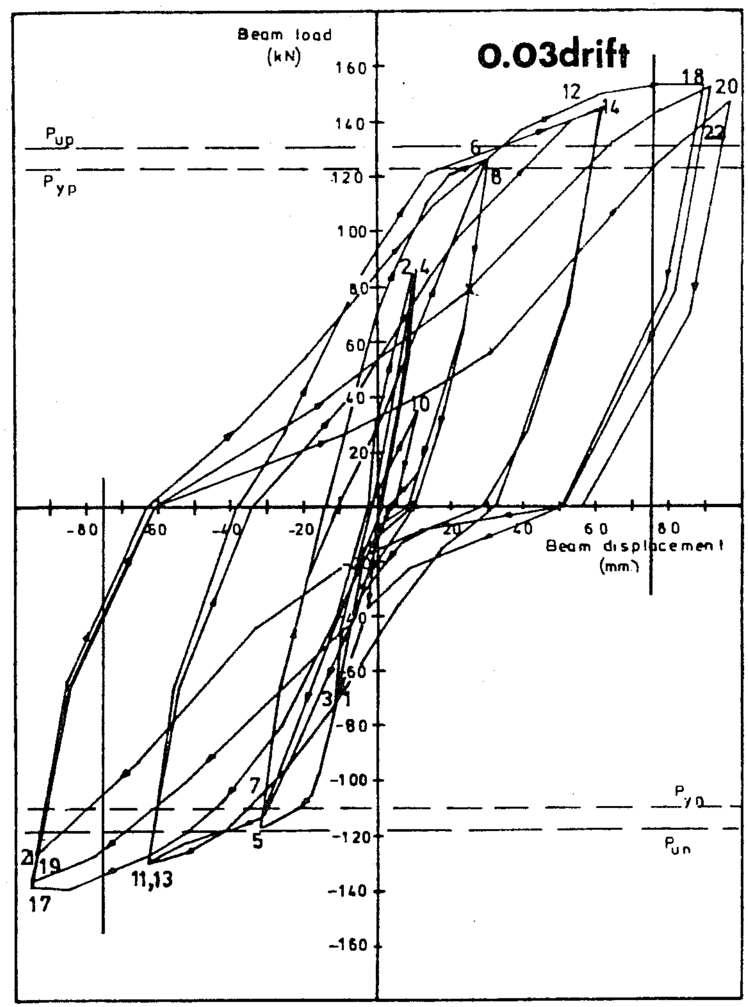


Fig. 32 Beam load-displacement response--
western beam [12].

This means that while bond damage should be avoided in the design of beam column joints, not all situations involving bond damage are unacceptable. As long as brittle failure or serious loss of load carrying capacity is avoided, the performance of the joint may be acceptable even though there is serious bond deterioration. The same phenomenon was observed in specimens B12 and B13. The measured strains in the beam bar 76 mm away from column face are listed in Table 4.

3.5 The University of Texas at Austin

Researchers at The University of Texas at Austin have reported a number of tests to examine the behavior of beam column joints under earthquake load including the effect of deteriorating bond strength [14, 15, 18, 19, 26, 27, 28]. A recent series of tests conducted as part of a U.S.-Japan cooperative program on large scale testing included a study of exterior and interior beam column joints with slab and lateral beams. Details of one of the interior beam column joint subassemblages (USJ-3) are shown in Figs. 33, 34, and 35. The beam load vs. displacement response is shown in Fig. 36. The equivalent story drift is indicated. From the hysteresis curves it is clear the joint was performing well up to a beam end deflection of 4.8 in. which is equivalent to the relative interstory drift of 0.06. No substantial loss of strength was observed. The ratio of column width to beam bar diameter was 22.5. Most tests have been carried out with smaller ratios. The measured beam bar strains close to the column face are listed in Table 5. Strains in the bottom beam bar were greater than that in the top beam bar at the same beam end deflection. Test data demonstrated that top beam bars and slab steel were mobilized to resist the beam end negative moment. It is likely slab

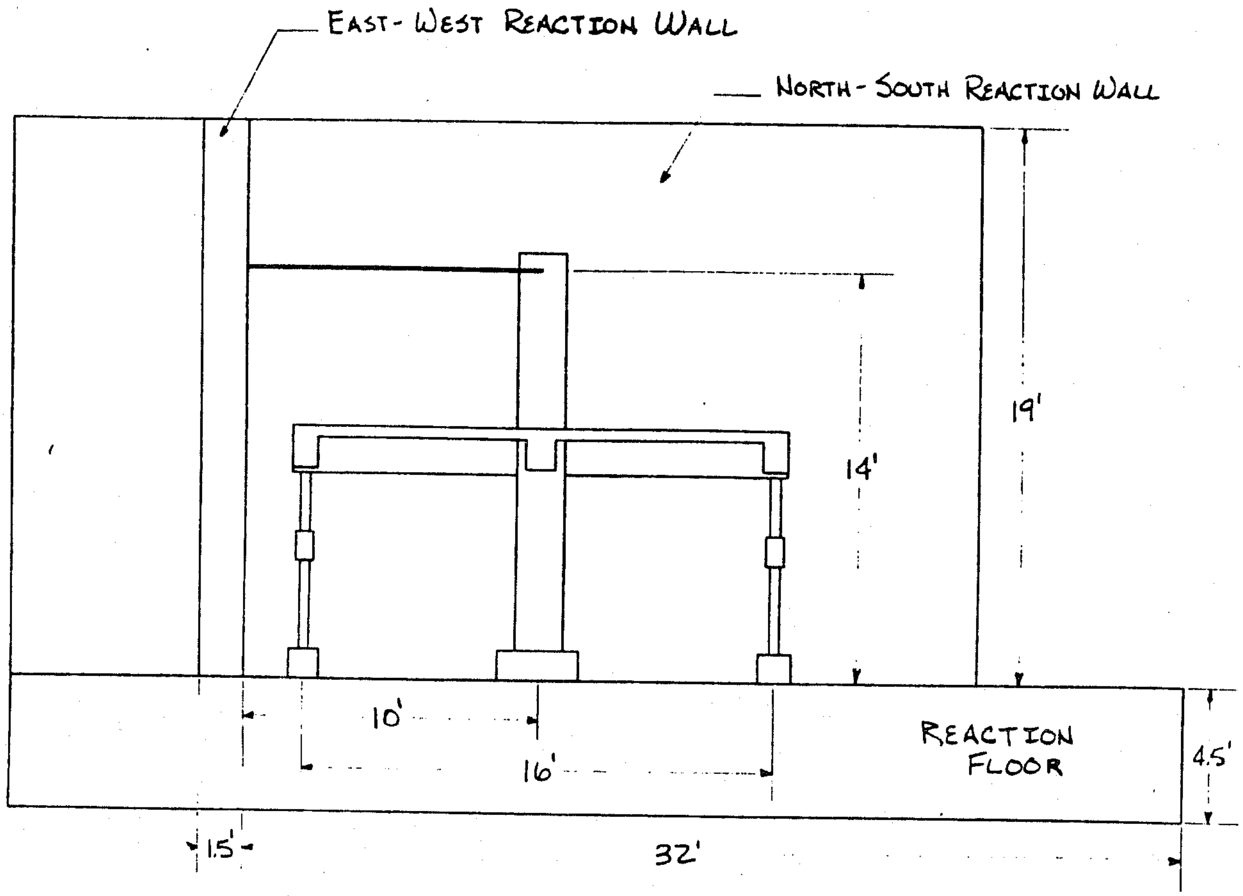


Fig. 33 Beam-column joint test setup of USJ-3, University of Texas at Austin [28].

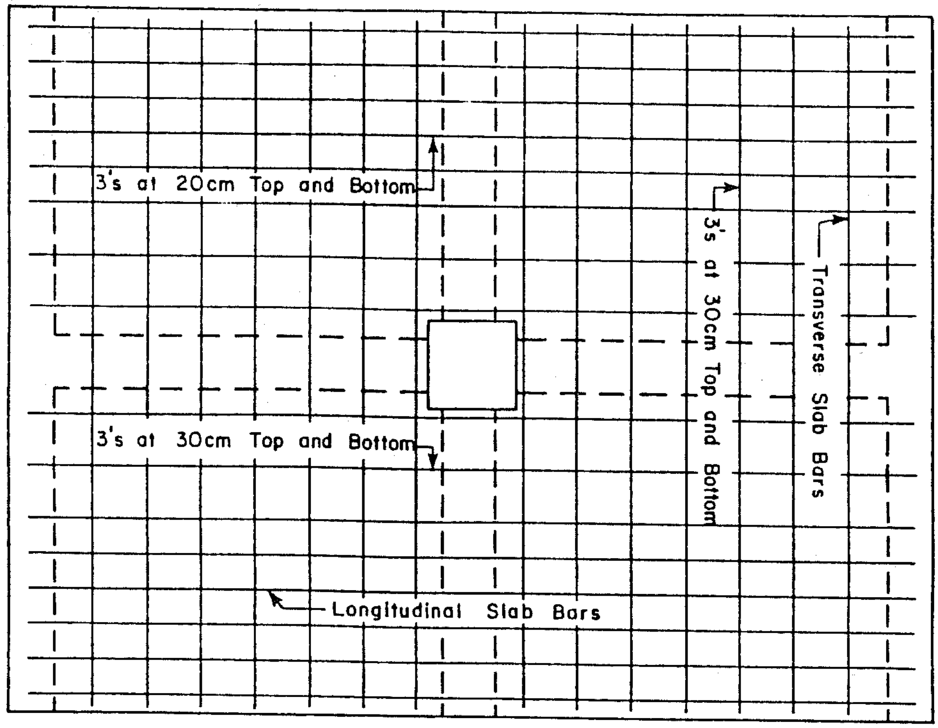


Fig. 34 Slab reinforcement in specimen USJ-3 [28].

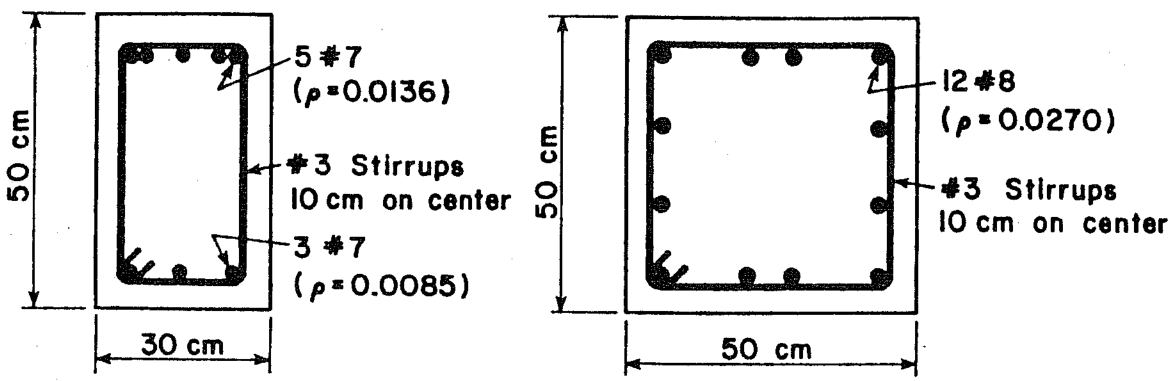


Fig. 35 Beam column section in specimen USJ-3 [28].

TABLE 5. MEASURED BEAM BAR STRAINS IN SPECIMEN USJ-#3, U. OF TEXAS

Position of Beam Bar	Beam End Deflection in.	Equivalent Interstory Drift %	Measured Strain in Beam Bar %
Top	1.2	3.0	0.13
Top	2.4	4.5	0.4
Top	3.6	6.0	0.79
Bottom	1.2	3.0	0.28
Bottom	2.4	4.5	0.83
Bottom	3.6	6.0	1.2

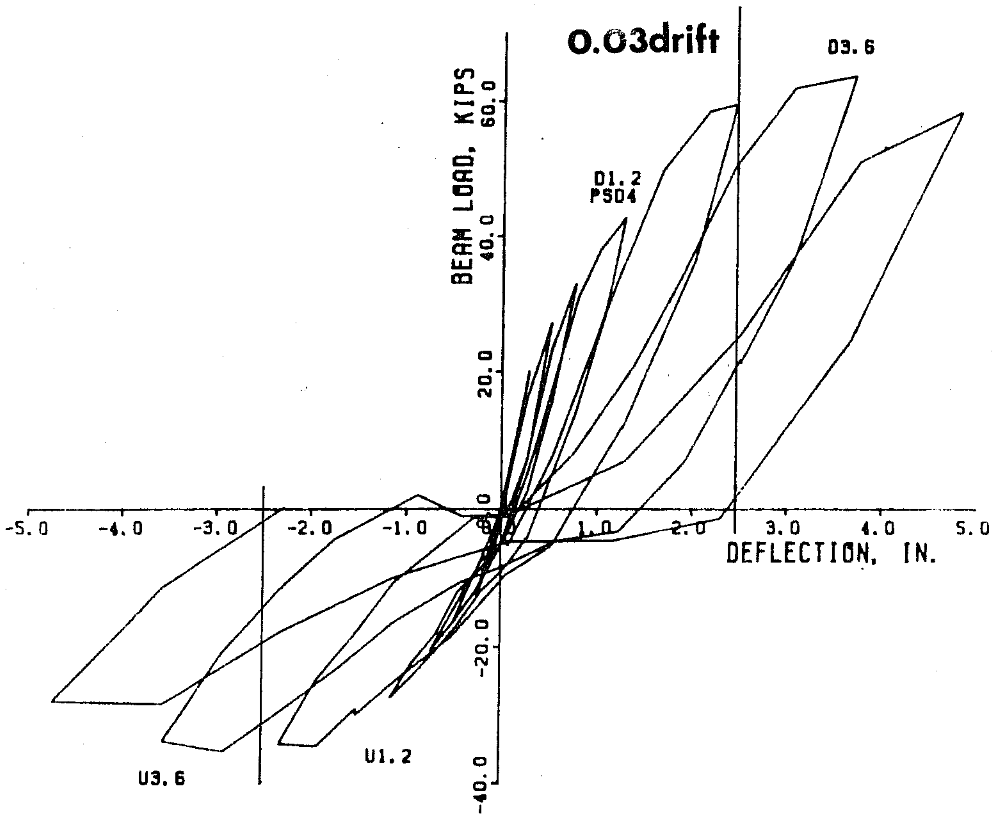


Fig. 36 Load displacement curve for specimen USJ-3 [28].

reinforcement restricted the opening of the crack at the face of the column. Since bond deterioration of the beam bar is sensitive to the deformation history, the existence of a slab improved the anchorage performance of the top beam bars. Beam bar slippage was measured during the test. No serious bond damage was observed until the beam end deflection reached an equivalent interstory drift of 0.045. The ratio of column width to beam bar diameter of 22.5 was sufficient to prevent serious bond damage under a severe earthquake for a joint with slab and lateral beams.

In order to investigate the mechanism by which shear carrying capacity and stiffness of the subassemblage degrade under severe cyclic loading fifteen full scale tests on beam-column joints subjected to severe biaxial cyclic loads were tested at The University of Texas at Austin [18, 26]. Thirteen of the 15 specimens were interior beam column joints which are listed in Table 6. Typical geometry of the specimen is shown in Fig. 37. Typical load displacement hysteresis curves are shown in Fig. 38. The hysteresis curves demonstrate that serious stiffness deterioration and pinching occurred during cyclic loading. A number of factors affected the stiffness deterioration of the specimen. A detailed discussion is found in Ref. 26.

As far as bond deterioration of the beam bars was concerned, the test results demonstrate that the ratio of column width to beam bar diameter and deformation history were the controlling factors.

For #8 top bars, when beam end deflection was equivalent to 2% drift, serious bond deterioration occurred but bond was not lost totally until 4% drift was reached in all the tests. For #6 bottom steel the bond damage did not occur up to beam deflection equivalent to 4% of the floor height although

TABLE 6. DETAILS OF SPECIMENS, UNIV. OF TEXAS [26]

Specimen	Column Steel	Beam Negative	Beam Positive	Joint Steel	Loading*	Beam Mask Ratio %	Slab
BCJ1	12 #9	3 #10	3 #8	2 #4	u	86	no
BCJ2	12 #9	3 #10	3 #8	2 #4	BA	86	no
BCJ3	12 #9	3 #10	3 #8	2 #4	BS	86	no
BCJ4	12 #9	3 #8	3 #8	2 #4	BS	86	no
BCJ5	12 #9	3 #8	3 #6	2 #4	BS	86	no
BCJ6	12 #9	3 #8	3 #6	2 #4	BS	86	no
BCJ7	12 #9	3 #8	3 #6	10 #4	BS	86	no
BCJ8	12 #9	3 #8	3 #6	2 #4	BS	86	no
BCJ9	12 #9	3 #8	3 #6	2 #4	BS	86	yes
BCJ10	12 #9	3 #8	3 #6	2 #4	BS	86	no
BCJ11	8 #11	2 #10	2 #8	2 #4	BS	60	no
BCJ12	12 #9	3 #8	3 #6	2 #4	BS	100	no
BCJ9A	12 #9	3 #8	3 #6	2 #4	BS	86	yes

* u = unidirectional; BA = biaxial alternate; BS = biaxial simultaneous

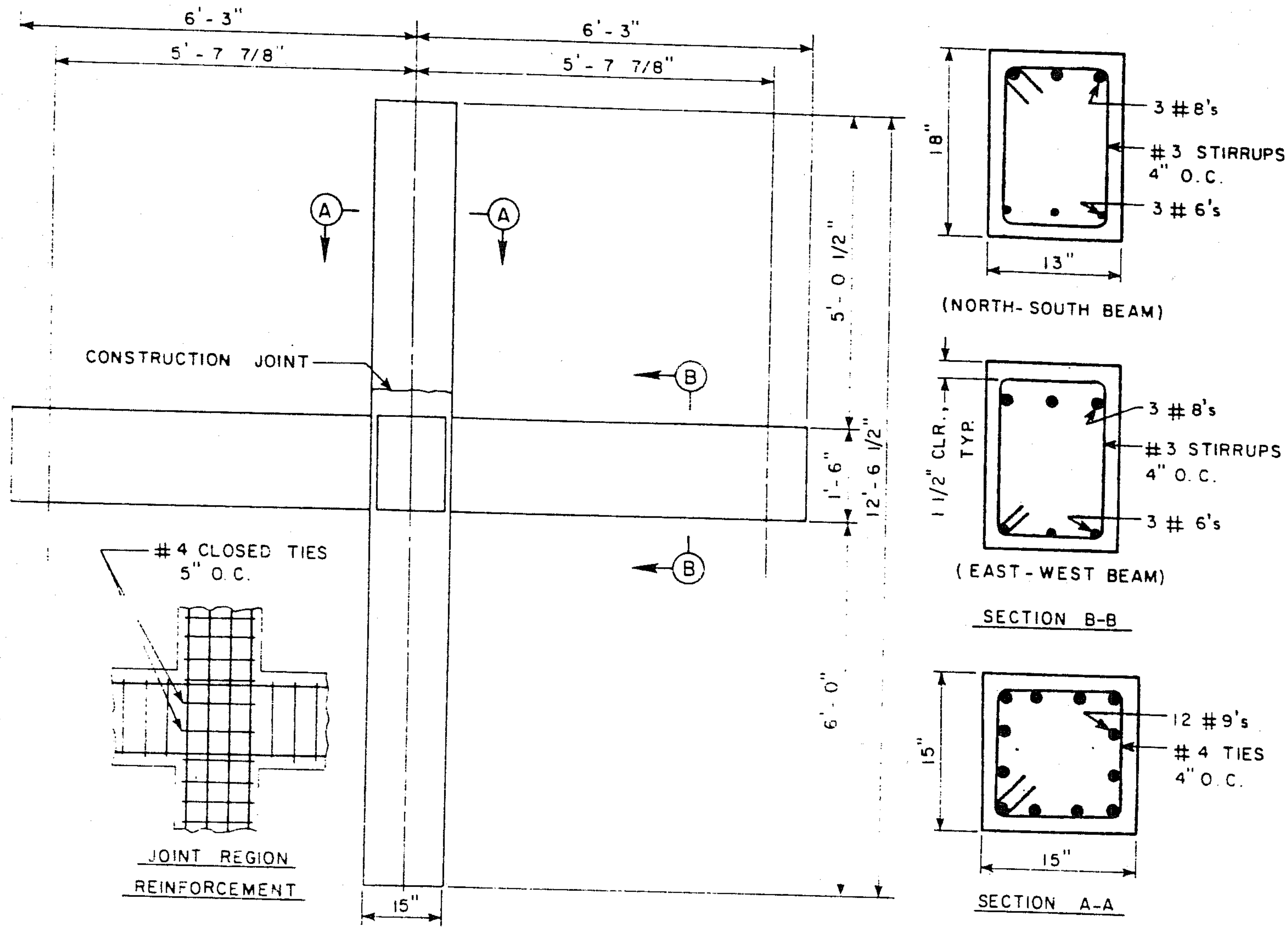
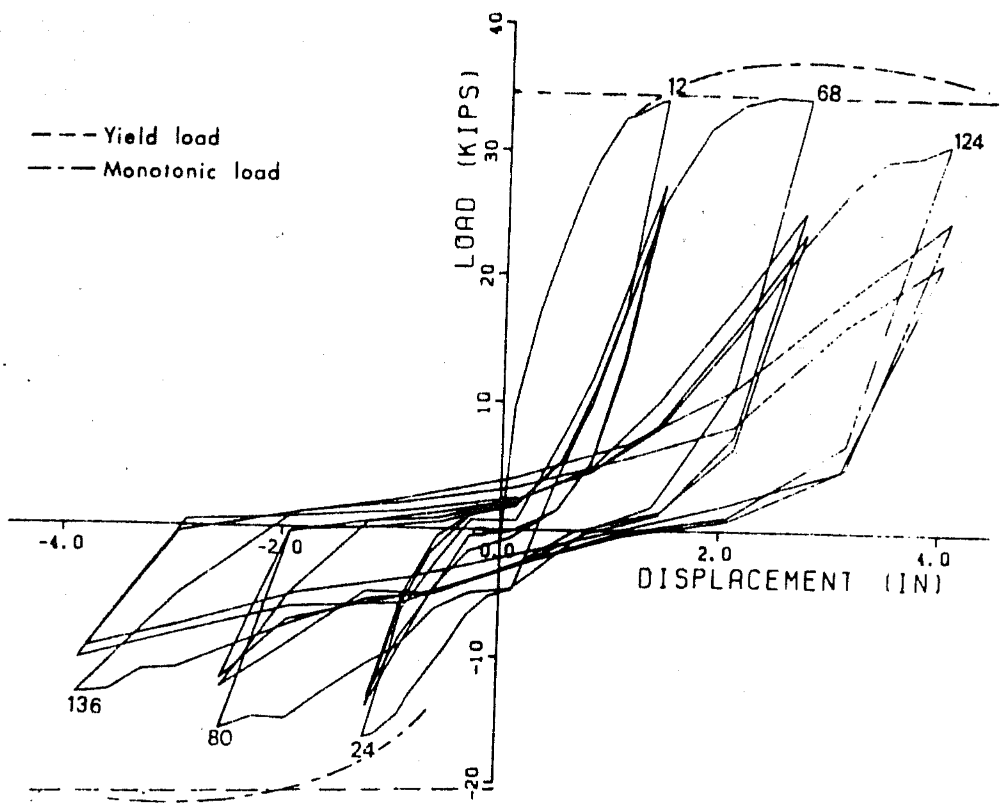
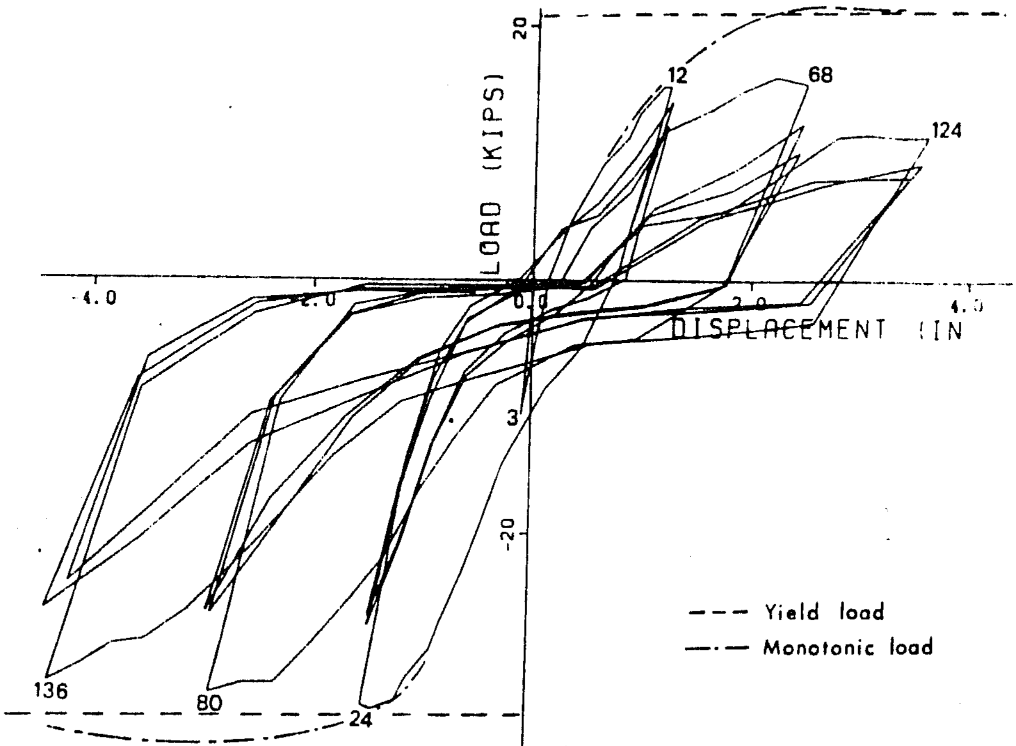


Fig. 37 Typical geometry of specimen series BCJ, University of Texas at Austin [18, 26].



a. Load-deformation for North beam [BCJ8]



b. Load-deformation for South beam [BCJ8]

Fig. 38 Typical load displacement hysteresis curves in test series BCJ [18, 26].

gradual bond degradation was observed. In #8 top bars tension at both faces of column was observed during the test, which indicated that bond damage had already occurred. However, tension at one end and compression at the other was observed in the #6 bottom bars during the test [see Fig. 8]. The average bond stress calculated from measured steel stress is shown in Fig. 39b. This result demonstrates that for #8 steel bars a value of 15 for the ratio of column width to beam bar diameter was not sufficient to prevent serious bond damage during a cyclic loading while for #6 bars, a ratio of 20 was sufficient. This conclusion can be deduced from the recorded beam deflection-bar slip curves shown in Fig. 39a. When beam end deflection was equivalent to 4% of the floor height, the top bar slip was about 0.13 in. and the bottom bar slip was about 0.1 in.

3.6 University of California, Berkeley

Ciampi, Eligehausen, Bertero and Popov presented an analytical model for the local bond stress-slip relationship under generalized excitations (1981) [11]. It was extended by Eligehausen, Bertero and Popov (1982) [11] and is illustrated in Fig. 40. Two kinds of tests were conducted in the Berkeley program to study bond experimentally. In one series of experiments, the bars were embedded in concrete stubs with a depth comparable to that of columns; in the other, very short bars (embedded lengths) were used to determine a local bond stress-slip relationship. A typical column stub with the test bar simulating one of the main beam bars is shown in Fig. 41. Using this setup, the influence of the anchorage length on the bar response was studied. The loading history used in the test was chosen as three cycles with $\epsilon = \pm 1.5\%$ plus three cycles with $\epsilon = \pm 3.0\%$ at the column face. The embedment lengths used in

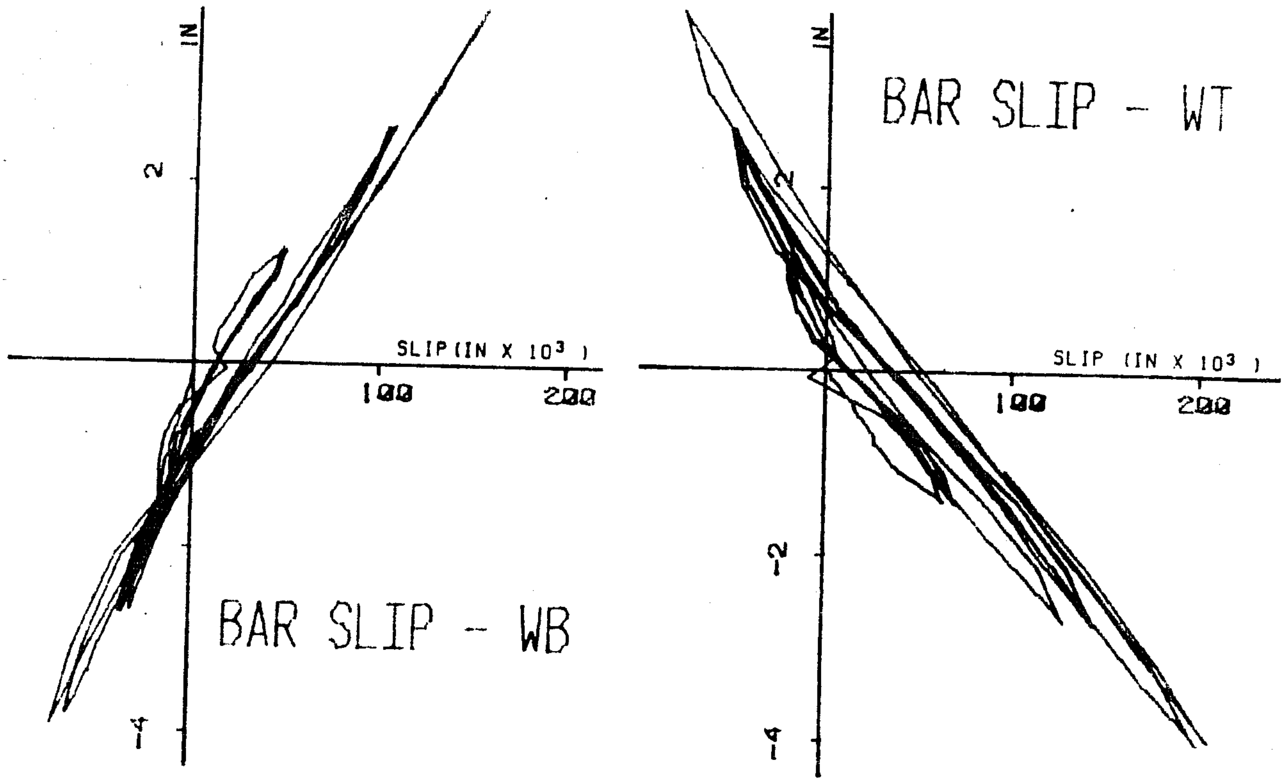


Fig. 39a Beam end deflection-bar slip curves [26].

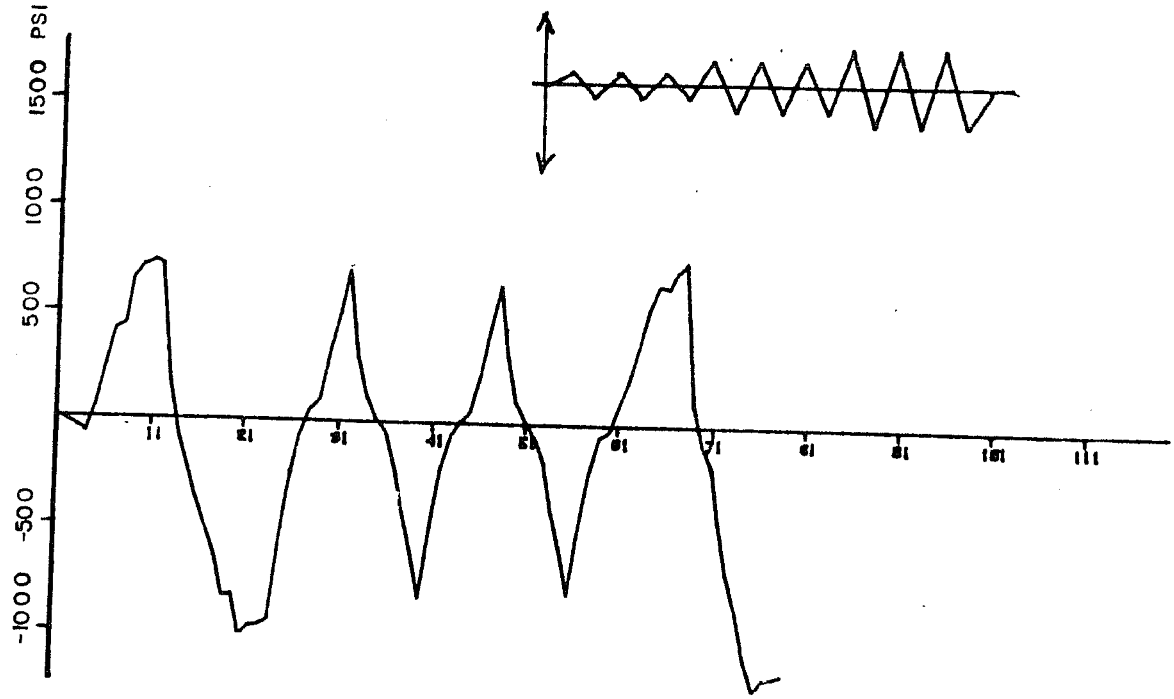


Fig. 39b Bottom bar bond stress in specimen BCJ8 [26].

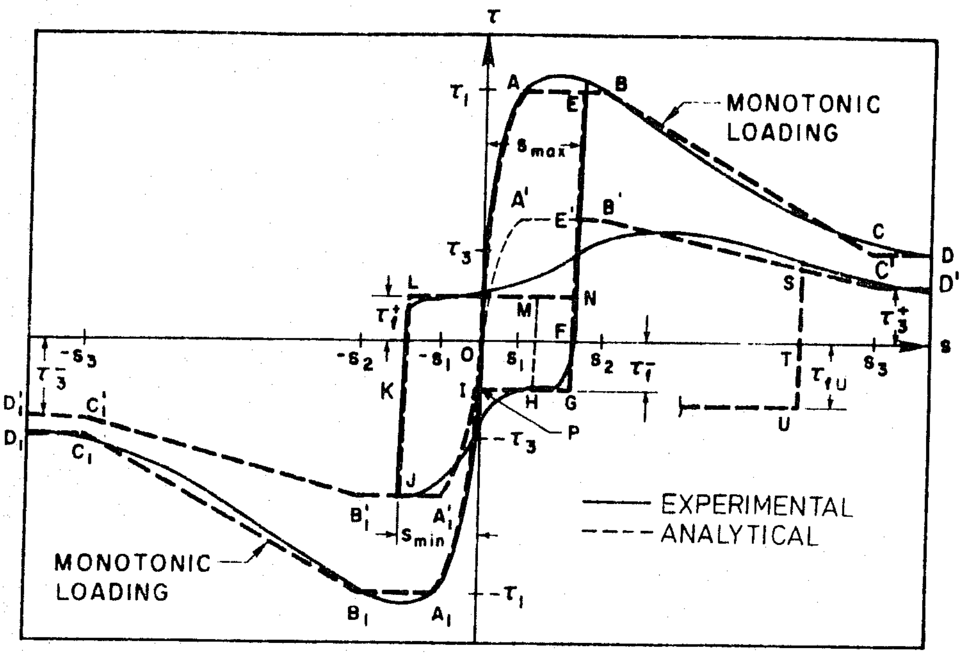


Fig. 40a- PROPOSED ANALYTICAL MODEL FOR LOCAL BOND STRESS-SLIP RELATIONSHIP [11].

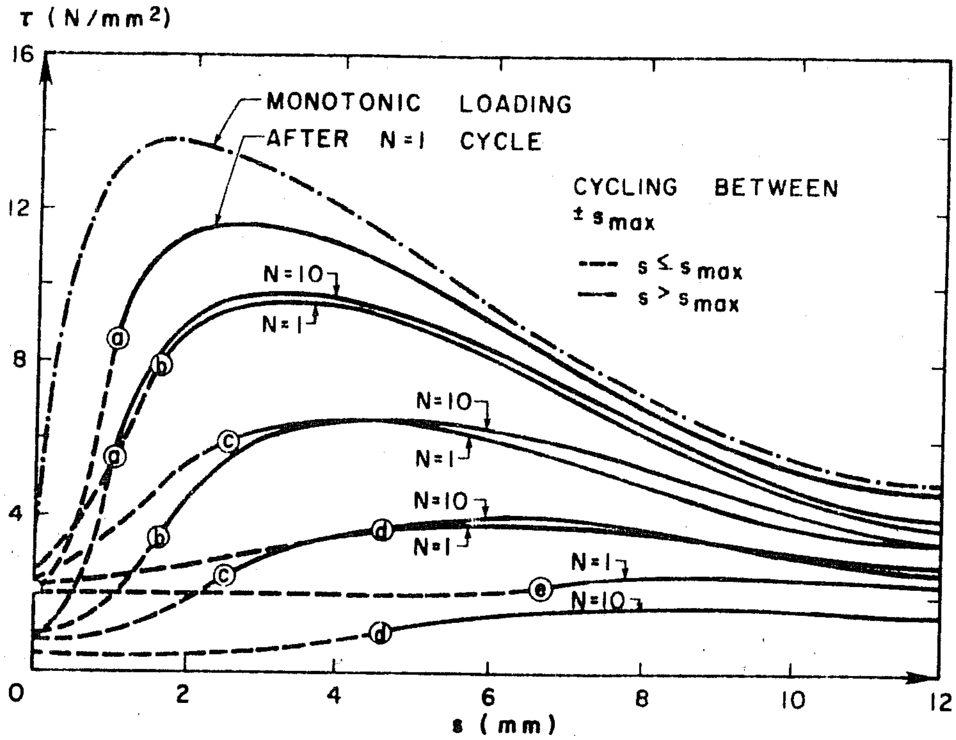
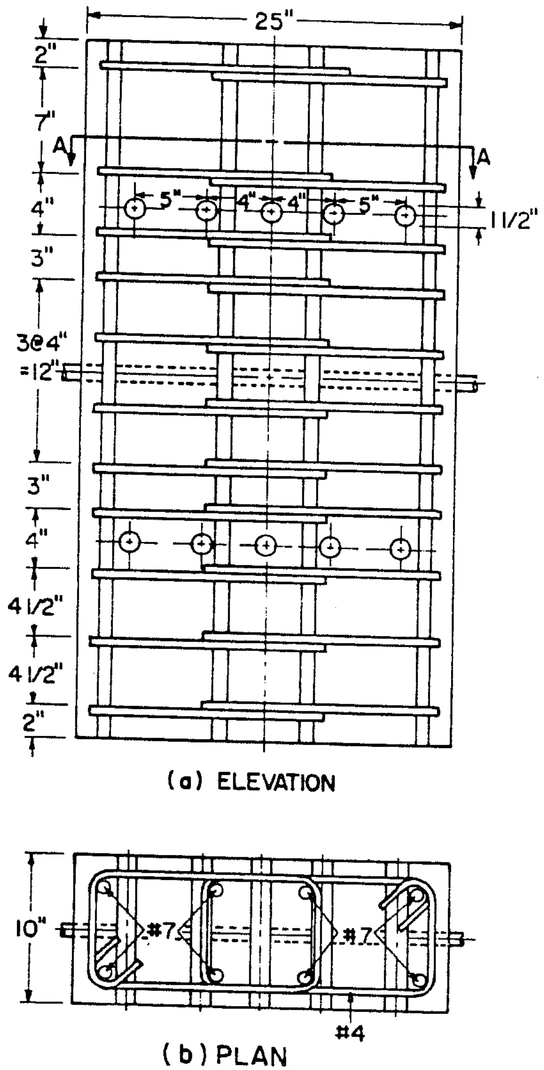
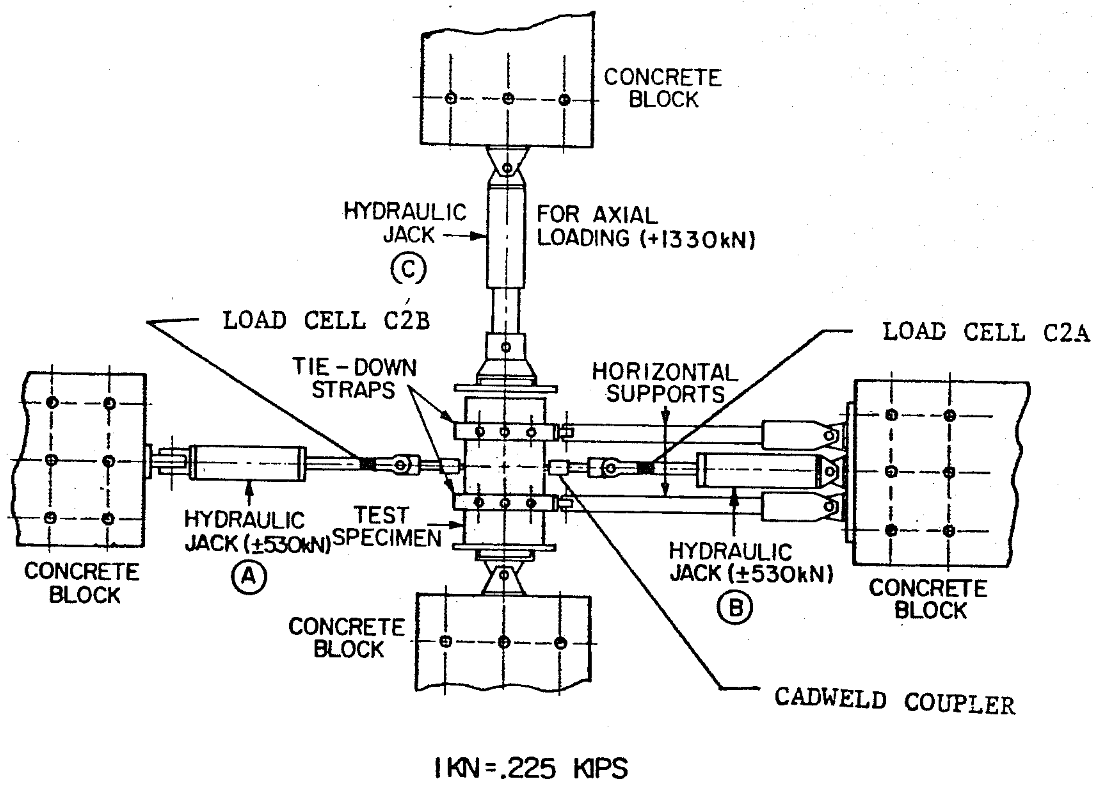


Fig. 40b - EFFECTS OF NUMBER OF CYCLES AND OF THE PEAK VALUES OF SLIP s_{max} AT WHICH THE CYCLING IS PERFORMED ON THE ENSUING BOND STRESS-SLIP RELATIONSHIP FOR $s > s_{max}$ [11].



a. Test specimen.



b. Test equipment.

Fig. 41 Bond deterioration test specimen and equipment, University of California, Berkeley [20].

the test were 15 d_b, 25 d_b, and 35 d_b. The criteria defining satisfactory response were (1) inevitable bond damage during cyclic loading is limited to the end region of the embedment length, (2) the hysteretic loops of the anchored bar remain essentially stable, (3) the strength of the anchorage continues to increase for slip values larger than the peak values during previous cycles. From the tests it was concluded that

(1) to minimize bond damage along the anchorage length and influence of slip on the dynamic response of the structure during major earthquakes, the anchorage length should be

25 d_b to 30 d_b (Grade 40 steel)

35 d_b to 40 d_b (Grade 60 steel)

(2) if the width of column at interior joints is smaller than the proposed anchorage length the formation of plastic hinges in the girders near the column faces should be avoided by detailing the beam reinforcement in an appropriate manner.

Another possibility is to take into account the influence of slip of beam bars on the dynamic response of reinforced concrete structures subjected to strong ground motion [11].

Actually, in most cases it is impossible to proportion a frame structure to satisfy development length requirements of 35 to 40 d_b for Grade 60 steel. Detailing the beam reinforcement in an appropriate way to avoid the formation of plastic hinges in the girders near the column faces is possible, but not practical in many cases. Congestion of steel bars leads to difficulty in fabricating the steel cages or placing concrete in joint area.

The deterioration of bond strength is very sensitive to the deformation history. The strain value in beam bars at the beam column intersection is influenced by a number of factors, such as the geometry of the section, the steel ratio in the section and the interstory drift. If an interstory drift of 0.03 is used, previous discussion of beam-column joint test results indicates that the strain in the beam steel would be less than 3%, the value used in the University of California (UCB) bond tests. The measured beam bar strains in the University of Michigan tests for Grade 60 steel were less than 2% (see Table 3). The measured beam bar strains in tests at The University of Texas at Austin were less than 1.5%. If less severe deformation history, e.g., $\epsilon = \pm 0.02$, were used the conclusions reached from the UCB tests might have been different.

As pointed out in Sec. 2.1, the situation of a beam bar in compression at one end and in tension at the other end only exists at a certain loading stage. After the beam touches the column the deformation and force situation in beam steel is changed. The beam bar could be in tension at one end and also in tension at the other as was observed in some tests [15, 17]. In actual beam column joints the deformation history in one end of the beam steel could lead to tensile strains as large as $\epsilon = +0.02$ but at the other side of the joint the compressive strain is not likely to reach $\epsilon = -0.02$.

In real structures, before the relative interstory drift reaches a critical value, say 0.03, destructive nonductile failures could occur elsewhere in the structure limiting the load applied to the joints. Such nonductile failures could be premature shear failure of beams or columns, compression failures of columns or anchorage failure in various locations. This may be one reason why the occurrence of joint failure in framed structures in recent destructive earthquakes has seldom been reported. This also demonstrates that the deformation history of a beam column joint in a real frame structure might not be as severe as that imposed on isolated beam column test subassemblages.

It is likely that the deformations imposed on test subassemblages are considerably higher than those in real frame structures during earthquakes. The deformations imposed on specimens in the UCB test program [11] are even higher than that in most test subassemblages.

Viwathanatepa, Popov and Bertero [13] reported an interior beam-column joint test, BC3. The geometry of this test subassemblage is shown in Fig. 42. The hysteretic curves are shown in Fig. 43. In this test, before the interstory drift reached 0.04 no serious drop of strength and bond damage was observed, the joint was performing very well. After the column displacement exceeded 0.04 of the floor height, appreciable slippage of bottom bars was noted. This test also demonstrated that a ratio of column width to beam bar diameter of 22.7 was sufficient to prevent bond damage at 3% interstory drift.

Using the specimen and equipment shown in Fig. 42, Viwathanatepa also studied the effects of generalized loadings on bond deterioration for bars embedded in confined concrete blocks. The loading history used in this test is shown in Fig. 44 [20]. In the test the strain distribution, displacement and force along the beam bar embedded in the concrete block were measured. Except for regions near the end the bond stress was surprisingly uniformly distributed (see Fig. 45). The results of five specimens tested under cyclic loading are listed in Table 7. The concrete within the column cage is defined as the confined region. The cover close to the pulled end is defined as

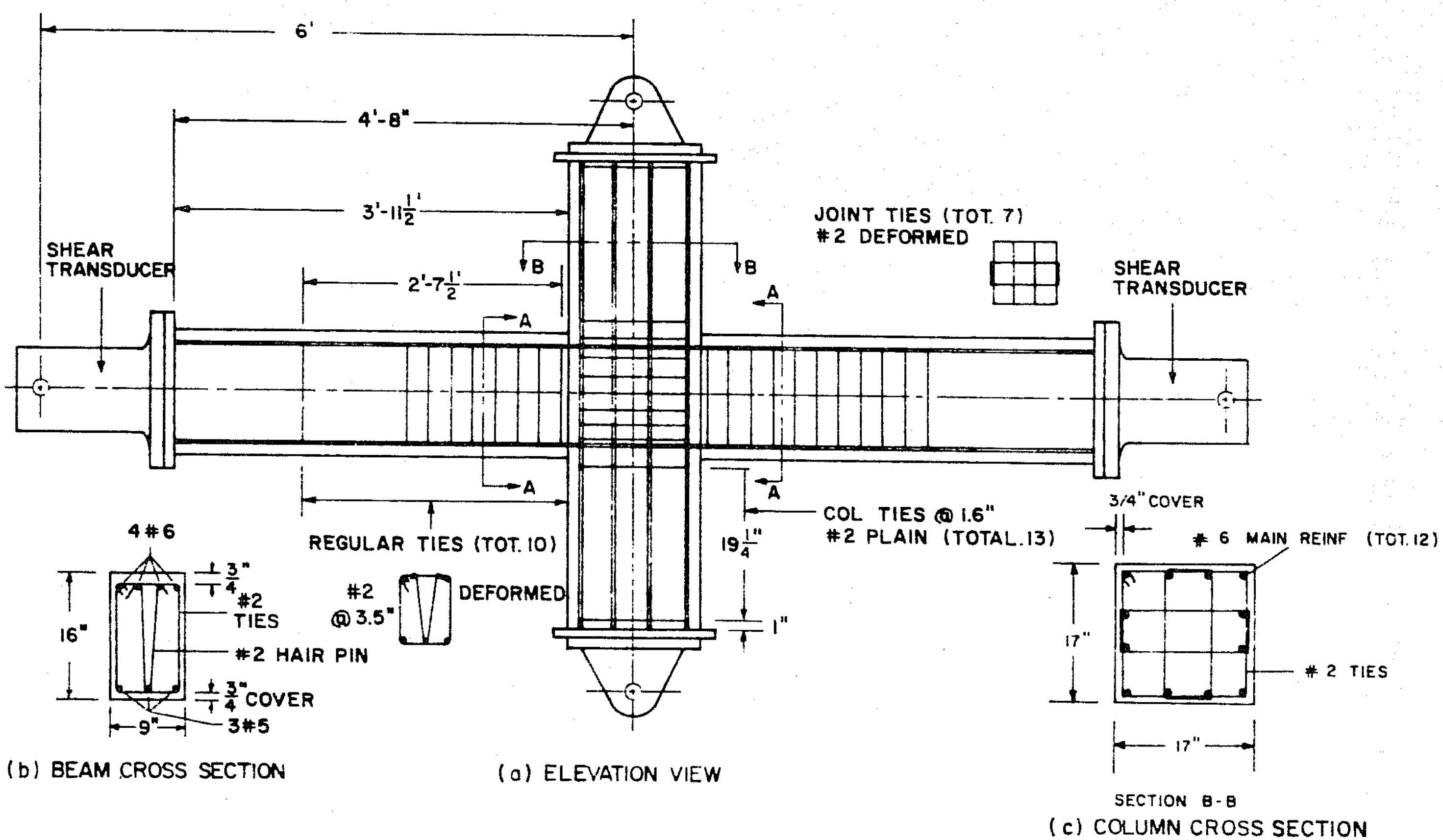


Fig. 42 Typical Steel Reinforcement in Beam-Column Subassemblage [13].

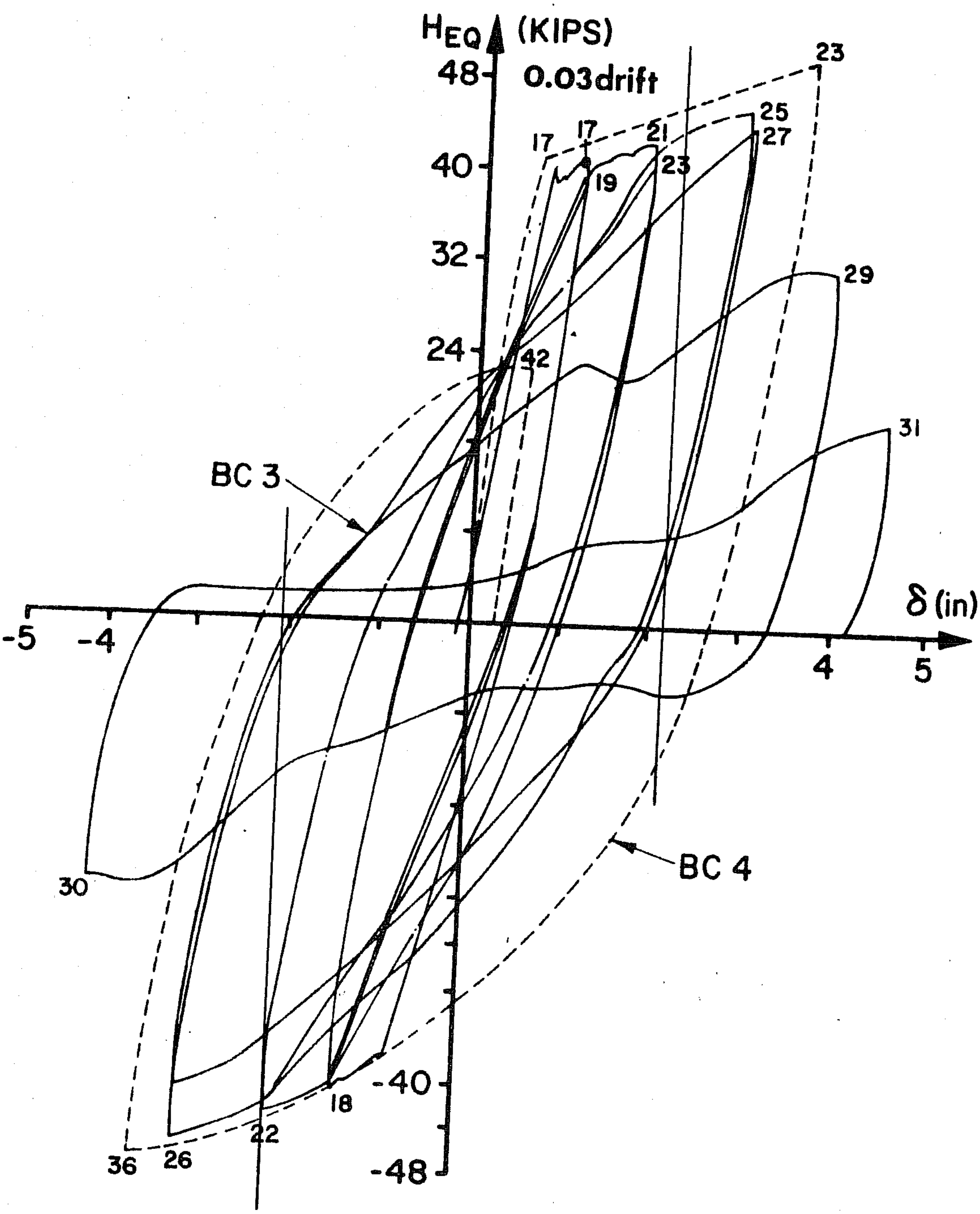
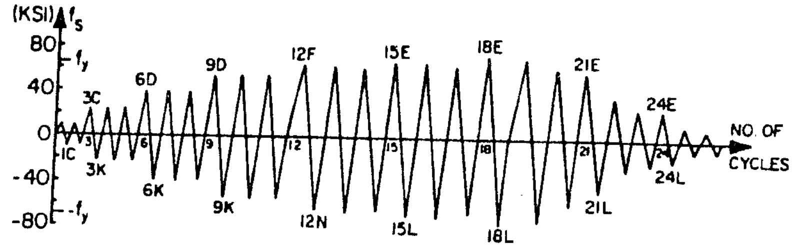


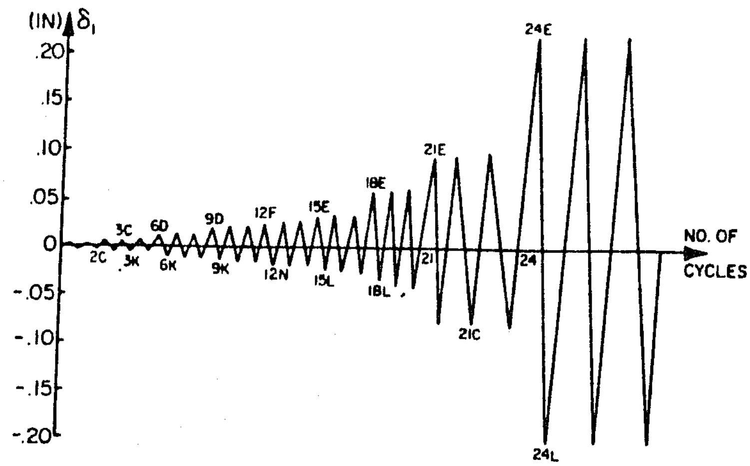
Fig. 43 $H_{EQ} - \delta$ Diagram for Specimens BC3 and BC4 [11].

TABLE 7. BOND STRENGTH FOR DIFFERENT DIAMETER BAR, U. OF CALIFORNIA-BERKELEY [20]

Specimen No.	Bar Size	d_b in.	Block Width in.	Concrete Strength psi	Maximum Bond Stress Unconf. Region ksi	Conf. Region ksi	Pushed End Region ksi
4	# 6	0.75	15	3890	0.9	1.9	4.3
8	# 8	1	15	4640	0.7	1.7	4.5
11	# 6	0.75	20	4230	0.9	2.0	4.9
14	# 8	1	25	4740	0.9	1.9	7.0
17	# 10	1.27	25	4290	0.8	1.9	7.4



(a) Force or Stress History



(b) Displacement History

Fig. 44 Loading history used in U.C. Berkeley for getting the bond distribution across a concrete stub.

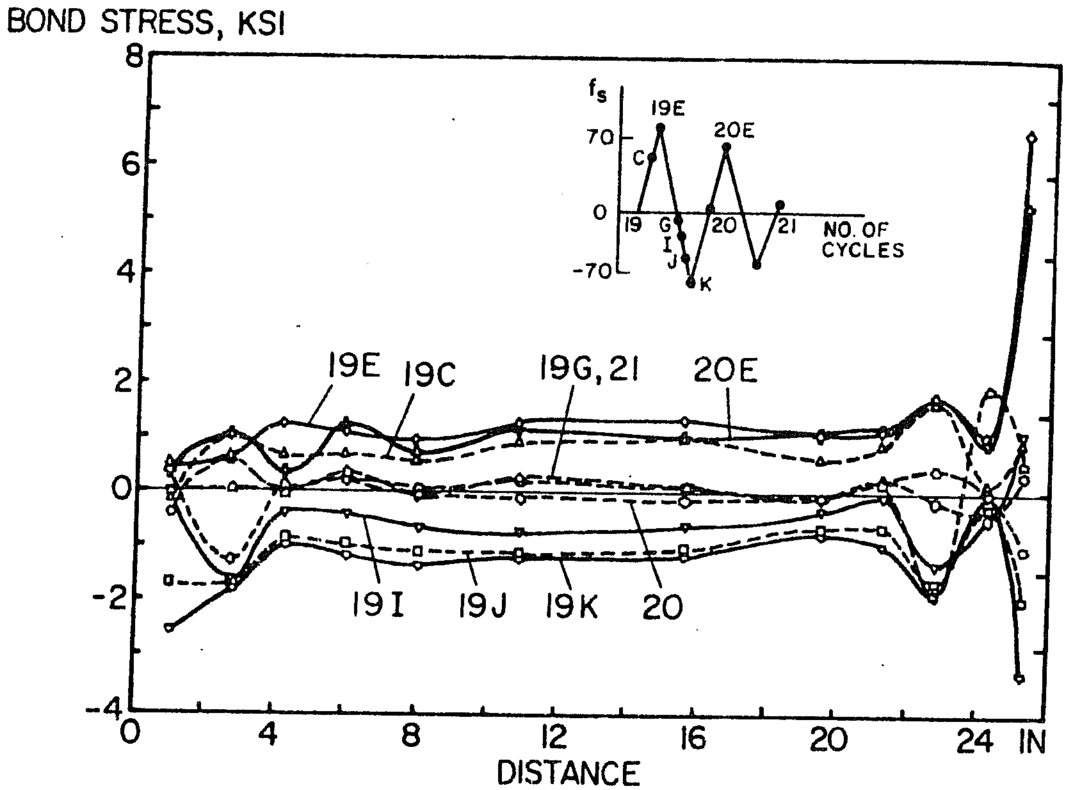


Fig. 45 Bond stress distribution diagrams at pull through range (pull-push) [20].

unconfined region while the cover in the other side as the pushed end region. It is interesting to note that the maximum bond stress in the confined region shows little variation with varying column widths, bar sizes, and concrete strengths (3890 - 4740 psi).

CHAPTER IV

ANCHORAGE LENGTH

4.1 Anchorage Length for Beam Bar

Deterioration of bond strength is dependent on the deformation history. In any beam column joint test, bond damage occurred if the deformation was sufficient. However, if a relative interstory drift of 0.03 were used as a criterion to evaluate the performance of a beam column joint, test data demonstrate that total bond damage could be avoided as long as the ratio of column width to beam bar diameter is large enough. Test data listed in Table 8 show that this ratio ranges from 16.3 to 24. Serious bond damage was observed for ratios of 15 at an interstory drift less than 0.03 in a test series at The University of Texas at Austin. Most test results conducted by different researchers show that for satisfactory bond conditions this ratio should be somewhere between 19 and 24. The excellent performance at drifts of 3 and 4% of specimens, in which the ratio was about 22, indicate that the ratios of column width to beam bar diameter around 20-22 are appropriate. Since Viwathanatepa's tests also demonstrate that the maximum bond stress in the confined region varies little with different column widths, bar sizes and concrete strengths, it is reasonable to conclude that a ratio of column width to beam bar diameter of 20 to 22 for #5 - #10 Grade 60 steel should ensure adequate performance. Note that tests were conducted with concrete strength f'_c = 4000 - 5000 psi and conclusions should not be extrapolated to higher concrete strengths.

TABLE 8. BEAM STEEL BOND SITUATIONS IN TESTS

Specimen	Test Series	f'_c psi	Nominal f_y ksi	Slab	Width of Col. in.	Beam Neg. Steel	Beam Pos. Steel	P/Py* at 0.03 Drift %	P/Py at 0.04 Drift %	P/Py at 0.05 Drift %	Column Width Top Bar Diam.	Bond Performance of Top Bars at 0.03 Drift	Column Width Bottom Bar Diam.	Bond Perf.of Bottom Bars at 0.03 Drift
X1	U. Mich. [17]	4980	60	no	14.25	4 #7	4 #6	1.0	0.90	0.75	16.3	good	19	good
X2	U. Mich. [17]	4880	60	no	14.25	4 #7	4 #6	1.0	0.90	0.75	16.3	good	19	good
X3	U. Mich. [17]	4500	60	no	14.25	3 #7	3 #6	1.0	1.0	1.0	16.3	good	19	good
S1	U. Mich. [17]	6030	60	yes	14.25	2 #7 6 #4	4 #6	1.0	1.0	1.0	16.3	good	19	good
S2	U. Mich. [17]	4460	60	yes	14.25	2 #7 6 #4	4 #6	1.0	1.0	1.0	16.3	good	19	good
S3	U. Mich. [17]	4100	60	yes	14.25	2 #7 6 #4	4 #6	1.0	1.0	1.0	16.3	good	19	good
B11	U. Cant. [12]	5205	40	no	18	4 #6	4 #6	1.19	1.19		24	good	24	good
B12	U. Cant. [12]	5017	40	no	18	4 #6	6 #6	1.17	1.17		24	good	24	good
B13	U. Cant. [12]	4553	40	no	18	4 #6	6 #6	1.17	1.17		24	good	24	good
BCJ4	U. Texas [27]		60	no	15	3 #8	3 #6				15	pull through	20	good
BCJ5	U. Texas [27]	4400	60	no	15	3 #8	3 #6				15	pull through	20	good
BCJ6	U. Texas [27]	4290	60	no	15	3 #8	3 #6				15	pull through	20	good
BCJ7	U. Texas [27]		60	no	15	3 #8	3 #6				15	pull through	20	good
BCJ8	U. Texas [27]	4300	60	no	15	3 #8	3 #6				15	pull through	20	good
BCJ9	U. Texas [27]	4100	60	yes	15	3 #8	3 #6				15	pull through	20	good
BCJ12	U. Texas [27]	4500	60	no	15	3 #8	3 #6				15	pull through	20	good
USJ#3	U. Texas [28]		60	yes	19.7	5 #7	3 #7	1.0	1.0	1.0	22.5	good	22.5	good
BC3	U.C.B. [13]		60	no	17	4 #6	3 #5	1.0	1.0		22.7	good	27.2	good

* load on beam/measured load at first yield.

4.2 Anchorage Length for Column Steel

Under reversed load column bar bond deterioration occurs and has a serious effect on the deterioration of joint stiffness. Under certain conditions column bars pull through the joint, as has been reported in some tests [17, 26]. Because of the strong column-weak beam principle, the stress level in the column steel usually is below the yield point. As a result, column bar bond damage has not been observed as often as in beam steel. However, because of the participation of slab and lateral beam in resisting the beam moment, this principle may sometimes be violated and may cause column bar pull through. Besides, inadequate detailing may also lead to column bond damage. Since column bar pull through totally changes the principle of a strong column-weak beam, attention should be paid to this problem. Basically, the nature of column bar bond deterioration is the same as beam steel. Only a brief analysis is presented hereafter.

Figure 46 shows the crushing and spalling of joint concrete after cyclic loading. In this case column bar pull through will inevitably occur no matter how high the ratio of beam depth to column bar diameter. First, crushing of joint concrete and yielding of column steel should be avoided. To reach this goal proposed Appendix A of ACI 318-83 requires that $\Sigma M_c \geq (5/4)\Sigma M_g$, where ΣM_c is the flexural capacity of the column while ΣM_g is the flexural capacity of the girders framing into that joint [10]. Proposed revisions to the ACI 352 report [8] will suggest that this coefficient be increased. If crushing of the joint concrete and yielding of column steel is avoided, the second important factor influencing column bar slippage is the ratio of beam depth to column bar diameter. Table 9 shows some test results.

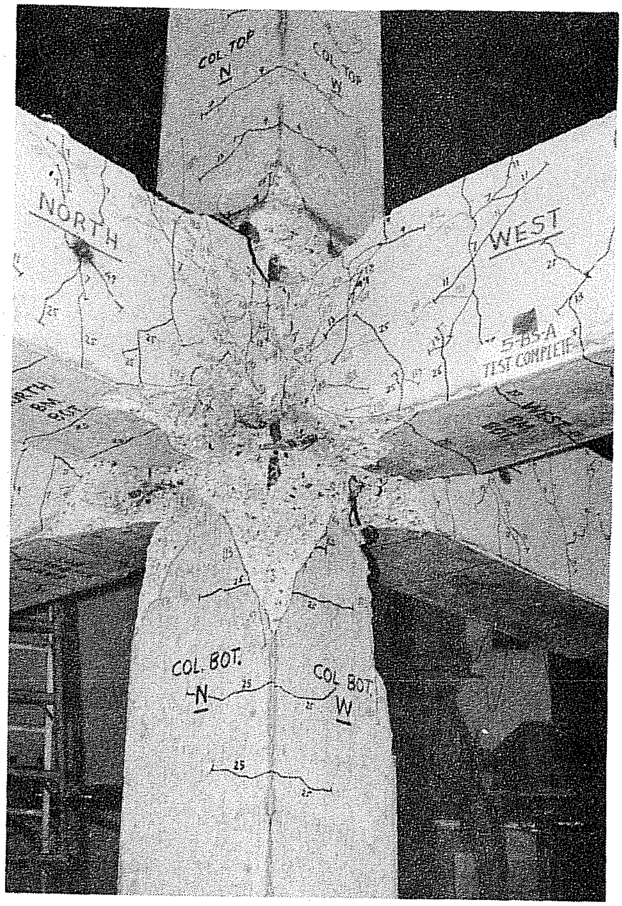


Fig. 46 Specimen 5-BS-A after testing [18].

TABLE 9. COLUMN STEEL BOND

Spec.	Test Series	Depth of Beam h_b in.	Column Bar Steel	$\frac{\text{Beam Depth}}{\text{Column Bar Diameter}}$	Observed Column Bar Bond Situation
X1	Michigan [17]	16.5	8 #8	16.5	Column bar pull through
X2	Michigan [17]	16.5	8 #8	16.5	Column bar pull through
X3	Michigan [17]	16.5	4 #7	18.8	Column bar pull through
BCJ5	U.T. [27]	18	4 #7	16	Crushing of joint concrete; column bar pull through
BCJ6	U.T. [27]	18	12 #9	16	Crushing of joint concrete; column bar pull through
BCJ7	U.T. [27]	18	12 #9	16	Crushing of joint conrete; column bar pull through
BCJ8	U.T. [26]	18	12 #9	16	Crushing of joint concrete; column bar pull through
BCJ9	U.T. [26]	18	12 #9	16	Crushing of joint concrete; column bar pull through
BCJ10	U.T. [26]	18	12 #9	16	Crushing of joint concrete; column bar pull through
B11	U. of Canter-bury [12]	24	12 #7	27	Good
B12	U. of Canter-bury [12]	24	12 #7	27	Good
B13	U. of Canter-bury [12]	24	12 #7	27	Good
USJ3	U. T. [28]	19.6	12 #8	19.6	Good
BC3	U.C. Berkeley [13]	16	12 #6	21.3	Good

Durrani and Wight [17] reported that in specimens X1, X2, X3 the column bar did not experience any yielding, but the slippage of edge bars through joint began from the very first loading cycle. A more serious situation was observed in test series BCJ at The University of Texas at Austin, in which column bar pull through followed the crushing of joint concrete [18, 27]. However, the USJ-3 and BC3 demonstrates that column bar bond damage could be avoided if the ratio of beam depth to column bar diameter were greater than 20.

To prevent column bar bond damage, it is suggested that

(1) column flexural capacity be not less than 1.4 times the beam flexural capacity including the participation of slab and lateral beam;

(2) the ratio of beam width to column bar diameter be not less than 22.

CHAPTER V

SUMMARY

(1) The mechanism of deterioration of bond strength was studied through a review of the test results conducted at the University of Michigan, The University of Texas at Austin, the University of California, Berkeley, and the University of Canterbury in New Zealand. The performance of beam column assemblages was evaluated using a relative interstory drift of 0.03. The use of an interstory drift for evaluating bond performance permits correlation of bond deterioration with overall frame response.

(2) It was found that interior beam column joints performed very well if the ratio of column width to beam bar diameter was greater than 17. No substantial loss of strength and bond damage was observed at interstory drift of 0.03.

(3) From the test data it is suggested that for Grade 60 steel bars the ratio of column width to beam bar diameter be not less than 20 - 22. This value is valid for concrete strength f'_c = 4000-5000 psi.

(4) Column bar bond was also examined in this paper. To prevent loss of column bar anchorage it is suggested that the column flexural capacity be not less than 1.4 times the beam flexural capacity and that the ratio of beam depth to column bar diameter be not less than 22. The beam flexural capacity should include the effect of the slab.

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