REINFORCED CONCRETE BEAM-COLUMN JOINTS FOR SEISMIC LOADING

Part II - Experimental Results

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ABSTRACT

The results of tests on four beam-column joint specimens are reported. It is shown that a joint which contained bond plates to prevent a bond failure of the flexural reinforcement in the joint, and was proportioned to limit yielding of the steel in this zone, had a markedly superior performance to specimens designed to comply with the ACI-318-71 code, or the proposals of May 1976 for the revision of the N.Z. Ministry of Works and Development code of practice for the design of public buildings (PW 81/10/1).

1. BEAM-COLUMN JOINT TEST PROCEDURE

The theory relating to the earthquake resistant design of beam column joints was reviewed and extended in an earlier paper(1) and need not be repeated here.

The four tests reported herein were designed to compare the performance of joints proportioned in accordance with existing practice with a special unit in which yield in the joint zone was controlled and bond slip was prevented by the use of bond plates welded to the flexural steel.

In designing the tests a careful watch had to be kept on the costs involved. For this reason the units' dimensions were kept small. This economic constraint, together with the size of the existing testing frame, resulted in a unit with a rather long column which gave a relatively flexible structure requiring a high section to displacement ductility ratio.

The column axial load was taken as zero to simplify the tests and to create a severe loading condition for the joint. With no axial load, small joint dimensions (thereby accentuating the bond problem) and the high ratio of section to displacement ductility, the test conditions were severe compared with previous tests reported in the literature (see References 3-8 in Reference 2 and References 13-37, 39-44 in Reference 3).

The test set-up is illustrated in Figure 1. The units were tested in a horizontal position in a frame that was bolted down on to the test floor. The joint zone reinforcement for the four units is shown in Figure 2.

Unit 1 was designed to comply with the ACI 318-71 code. Unit 2 was the first of two units in which bond plates were detailed to prevent slip. Unit 3 was designed by Mr. I. C. Armstrong (District Structural Engineer, M.W.D., Auckland) to comply with the draft revision PW 81/10/1 - May 1976. Finally, Unit 4 was an improved version of Unit 2. The main features of these are listed in Table 1. In all cases the columns had an appreciably greater theoretical ultimate flexural strength than the beams.

The concrete for the tests was supplied by a local ready mix company. The specified cylinder strength was 35 MPa with a 75 mm slump and a maximum aggregate size of 10 mm. The concrete strengths listed in Table 1 are the averages of three 300 mm x 150 mm cylinder crushing tests for each mix.

1.1 Bond Plate Design Details (Units 2 and 4)

The units with bond plates were designed to transmit all the joint shear by diagonal strut action (see Figures 3 and 2 (b) and (d)). No bond transfer was assumed for the main flexural steel in the joint zone and, consequently, the complete change in force in the bar, from yield in compression on one side to yield in tension on the other, was taken out on the bond plate which supported the diagonal compression force on the concrete.

The plates were proportioned so that the bearing stress on the concrete was approximately at the cylinder crushing strength. There was no evidence of local crushing adjacent to the plate so that it is likely that the procedure is conservative, possibly overly so bearing in mind the confinement of the diagonal strut.

To reduce the possibility of a local brittle failure in the bar due to high welding shrinkage stresses, the bond plates were cut in two and each half was then butt welded to the bar. The junctions between the two halves of the plate were then lightly welded together.

If the bar between the bond plates yields (in this region of the bar only yield in tension is possible), then, on load reversal, the bar would slip back through the concrete to allow the second bond plate to bear against the concrete. Clearly, to prevent this slippage and the consequent degradation of stiffness, it is necessary to prevent, or at any rate limit, the inelastic extension of the flexural reinforcement in the joint zone.

In Unit 2 an attempt was made to limit the yield extension between the bond plates

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by the addition of extra flexural reinforcement in the beam. This took the form of four 12 mm bars which were placed on the inside of the main flexural reinforcement cage. These bars extended between the bond plates, but away from this location they were bent into the centre of the beam (see Figure 2(b)). The test results showed that this was only partially successful. At the higher ductility levels these bars appear to have become debonded allowing extensive yielding of the main steel in the joint and, therefore, slip of the bars through the joint.

Yielding of the main beam steel in the joint of Unit 4 was prevented by the addition of two 10 mm bars fillet welded onto each 24 mm bar between the bond plates (see Figure 2(d)). This arrangement worked extremely well and no slippage was apparent.

With diagonal strut action as the principal method of load transfer in the joints of Units 2 and 4, joint shear reinforcement was not required. However, some joint steel is needed to control the diagonal splitting which occurs in the joint, as indicated in the previous paper(1). The areas of joint reinforcement actually used to control this cracking (1584mm^2 and 999mm^2 for units 2 and 4 respectively) were markedly less than those required for joint shear resistance in units 1 and 3 (see Table 1). In the case of Unit 4 only half the steel specified by equation (7) and Reference (1) was used (i.e. $\gamma = \frac{1}{2}$, see equation (8)). It was felt that once diagonal cracks form the tension stiffness across the diagonal decreases and the forces required to be transmitted across the crack are substantially reduced. The experimental results bear out this conclusion.

1.2 Load Sequence and Experimental Measurements

The loading sequence was the same for all units, being similar to that used in recent MWD tests(4). In the first two complete cycles of the test on Unit 1, the load was increased in each direction to three-quarters of the level calculated to give first yield of the reinforcement. The resulting load-deflection relationship was plotted and a straight line was fitted to the points and extended to the calculated first yield level. The displacement corresponding to this was then used as a reference value of "ductility one". From this stage on displacements were imposed on the beams as indicated in Figure 4(a). All subsequent units were subjected to the same displacements as Unit 1.

Readings of the beam deflections were taken at the two points shown in Figure 4(b). In addition, the deflected shapes of the complete units were picked out using a theodolite to measure the movement of targets attached to the unit. These measurements were taken at only a few representative load increments.

Electrical resistance strain gauges were attached to the main flexural steel in Unit 1. In all, eighteen gauges were used, their positions being as shown in Figure 7.

Demec gauge readings were taken on all units. However, extensive yielding of the reinforcement in the joint zones of Units 1, 2 and 3 rapidly caused the strains to exceed the gauge capacities. By comparison, the cracks in the joint zone of Unit 4 were well controlled and this allowed readings to be taken to the end of the ductility six displacements.

In Units 3 and 4 measurements were taken of the expansion of the central 1.6 m of the beam under cyclic loading. At low ductility levels these values were found using demec gauge readings, but at higher levels the expansion was measured directly with a tape.

2. EXPERIMENTAL RESULTS

The main experimental results are recorded in Figures 5-14.

2.1 Units 1 and 3

Unit 1 was designed to comply with the current ACI code (318-71), and Unit 3 was proportioned to meet the draft provisions of PW 81/10/1 of May 1976, with the exception that as the gap between the flexural steel in the column was small no intermediate column bars were used. The effect this may have had on the joint is unknown. (See Table 1 in Reference 1.) The flexural tension steel percentage in the beam in Unit 3 was considerably less than in the other units (1.3% as against 1.8%), as it was limited by the calculated shear strength of the joint zone corresponding to the maximum quantity of stirrup reinforcement that could be fitted into this region.

The behaviour of both joints was similar in that both suffered severe load and stiffness degradation. The considerable increase in the ratio of shear reinforcement to flexural steel increased the toughness of Unit 3, but the gain appears to be small (Figures 5, 6, 9, and 10).

Both joints cracked diagonally at a load of 18 kN in the jacks. These cracks opened up when the units were taken into the inelastic region. The inelastic extension of the steel was concentrated in the joint and in the beam close to the joint.

For load cycles involving ductilities greater than two slippage of the beam reinforcement through the joint zone was apparent. Inspection of the joint of Unit 1 showed clear evidence of bar slip, with the deformations on the steel moving relative to the indentations in the concrete by as much as 3 - 5 mm. Such slippage gives rise to beam displacements as is apparent from the flat portions of the load deflection curves at higher ductilities (see Figure 5). In the joint of Unit 3 the concrete contained around and between the six bars in each face of the beam was extensively broken up. It was apparent in this case that the bond was destroyed not so much at the steel-concrete interface as by disruption of the concrete surrounding the bars in the joint region. For this type of bond failure the reduction in bar diameter from 20 mm in Unit 1 to 12 mm in Unit 3 did not appear to help because it was the concrete interface right round the group that failed.

In the load cycles involving ductilities of four and more the stiffnesses of the units were severely reduced due to the slip of the bars. The shear resistance was the result of diagonal strut action. Extensive yield of the beam reinforcement (see Figure 7), coupled with the high cyclic bond stresses, destroyed the bond forces in the joints. With each load cycle the flexural tension steel was anchored further back into the compression zone of the beam on the far side of the joint. The extensive bar slip at low loads, which was a characteristic of these two joints, gave poor energy absorption characteristics (see Figures 5, 6, and 10).

Some of the strain gauge readings which are tabulated in the report noted below* are shown in Figures 7 and 8. The readings indicated that the beam reinforcement in Unit 1 yielded, but the column steel remained in the elastic range. It is interesting to note that the bond did not fail on the column bars even though these stresses were very high and of a cyclic nature. Figure 8 shows that the flexural tension forces in the column steel were effectively anchored by bond on the far side of the joint. This confirms the diagonal strut mechanism (see Figure 3 (c) of Reference (1)). The experimental measurements show that sustained cyclic bond stresses of at least 6.0 MPa, and probably 9 MPa, were sustained. It is believed that this magnitude of bond resistance could be maintained only because:

(i) the column bars had stirrups anchored round them, thereby increasing bond resistance(5), and

(ii) the bars did not yield.

Extreme spalling of the cover concrete in the joints of Units 1 and 3 occurred during the first cycle of ductility six.

2.2 Units 2 and 4 - Bond Plate Units

Both these units contained bond plates, and they were designed to resist shear by diagonal strut action. Extra flexural steel was added to the joints to limit the yield extension of the bars between the bond plates on each side of the joint. The main difference in these units lay in the way in which the extra steel was added.

In Unit 2 two extra 12 mm bars were placed below the bond plates in each face of the beam, as shown in Figure 2 (b). This gave rise to a theoretical increase in moment capacity of the joint of 16% at first yield of the steel. In fact, the actual increase in bending moment after first yield of the steel was 21% and, consequently, inelastic extension of the steel would have occurred in the first cycle of ductility four. It is believed that the deterioration of joint performance after the completion of the cycles of ductility four was due to:

(i) the inelastic extension of the bars mentioned above, and

(ii) the inadequate anchorage of the 12 mm bars in the beams. Under cyclic loading these additional bars became debonded making them ineffective in limiting yield of the reinforcement between the bond plates.

In spite of these weaknesses, Unit 2 performed marginally better than Units 1 and 3, even though it had a much smaller area of joint reinforcement (see Tables 1 and compare Figures 6, 9, 10 and 11).

In Unit 4 the area of beam reinforcement in the joint zone was increased by welding two 10 mm bars to each of the 24 mm bars in each face. This gave a local increase in beam steel area through the joint of 35%. As the peak moment capacity was 33% above the calculated first inelastic extension of the steel, yield in this zone should have been avoided. By tack welding the 10 mm bars to the main steel, and butt welding them to the bond plates, the possibility of anchorage failure was prevented.

The performance of Unit 4 stands out from the others (see Figures 14, 12, 9 and 10). The cracks in the joint were fine and well controlled up to the second cycle of ductility six. At the first cycle of ductility eight the concrete in the joint zone started to spall, and it was not until the second cycle of ductility eight that the unit started to degrade. However, right through the whole loading sequence the performance of the joint was very much better than the others. It should be noted that this marked improvement in performance occurred in spite of the fact that the amount of shear reinforcement in the joint was very substantially less than in all the other tests (see Table 1).

It is believed that the degradation of the joint arose from yield in the flexural steel of the column. The column was designed with an ultimate strength 40% greater than that of the beam, but to first yield of steel the corresponding value was only 24%. This difference was insufficient to prevent yielding of the column bars when displacement ductilities of six were imposed. Measurements made on the unit (at ductilities of eight) indicated that the column had expanded by 3 mm over the central 600 mm of the joint zone. This is a clear indication of yield of the column steel.

2.3 Beam Expansion and Sliding Shear in Beams

Under cyclic loading involving inelastic extensions the reinforcement extends in tension, thereby opening up the cracks in the concrete, but under load reversal these cracks tend not to close completely in plastic hinge zones. Furthermore, in joint zones which act by the formation of diagonal struts, inelastic tensile strains may be expected to exist simultaneously in both the top and bottom steel through the joint (see Figure 3 (c) of Reference (1)). This behaviour was confirmed in Unit 1 (see Figure 7). The inelastic tensile extension of the reinforcing steel, which is not removed by subsequent compression, results in a growth in the length of the beams. Of course, in actual structures the growth in length will be restrained by slabs, shear walls or fixings (the slab effect).

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* Fenwick, R.C., and Irvine, H.M., "Beam-Column Joints for Seismic Loading", UASE Report No. 142, March 1977.

The increase in length of the beams was recorded for Units 3 and 4 and the results are shown in Figure 13. It will be noticed that these increments are far from negligible, particularly in Unit 4, where extensive plastic hinging occurred over an appreciable length of the beams. The total extension at the last cycle on Unit 4 was 30 mm, which is approximately 1% of the total beam length! The reason for the extension being greater in Unit 4 than in Unit 3 is because bar slip occurred in Unit 3, thereby allowing the large ductilities to be achieved without such high plastic strains as were necessary in Unit 4 (where the joint behaved itself).

The continuous opening of the flexural cracks in the plastic hinge zones of the beams gave rise to the problem of sliding shear. Units 1 and 3, which failed in the joint zone and did not develop major plastic hinges in the beams, showed no distress in shear. The beams of Units 2 and 4, in which the shear reinforcement was proportioned to carry 70 kN ($\phi = 1$), did show distress in shear. In Unit 2 this arose in the form of wide steep cracks (steeper than 45°). Additional external stirrups were fitted to the beams to prevent a premature shear failure.

Unit 4, in which very wide flexural cracks developed in the plastic hinging zone of the beams, showed distress in sliding shear. To prevent this external stirrups were clamped around the critical cracks. Without these it is almost certain a sliding shear failure would have occurred. The shear reinforcement in Unit 4 consisted of 6.5 mm diameter two-leg stirrups spaced at 60 mm centres. The shear stress, based on the peak load (occurring at the first cycle of ductility six), was 1.19 MPa, or 0.19 $\sqrt{f'_c}$ MPa. This is well below the limit at which sliding shear is normally expected.

Sliding shear in beams is the subject of a current thesis project under the supervision of the senior author. Results from this study will be available at a later date.

2.4 Crack Widths in the Joint Zone

The opening of cracks in the joint zone was indirectly measured from displacement readings made across this region by mechanical strain gauges. A few of the results are shown plotted in Figure 14. It can be seen that the crack widths in the joint zones of Units 1, 2 and 3 increased rapidly once they were taken into the inelastic region. By contrast, the crack widths in the joint of Unit 4 remained small and well controlled until the second cycle of ductility six.

3. CONCLUSIONS

The test results from Unit 4 clearly indicate that beam-column joints may be designed to resist joint shear by diagonal strut action. Such joints require that:

(i) Extra beam flexural steel be placed through the joint to control the amount of yielding in the joint zone and to keep the plastic behaviour mainly in the beams. This additional reinforcement must be adequately anchored in the beams to prevent debonding.

(ii) Bond plates be attached to the flexural steel in the column and beams to transmit the bar forces into the diagonal struts which form in the joint (alternatively, the flexural steel could be cranked through the joint but this appears to be a difficult detail to effect in practice and is, therefore, mentioned here only in passing).

(iii) Stirrups and longitudinal bars be placed in the joint zone to control the width of the main diagonal cracks. The area of steel required to achieve this appears to be of the order of a quarter to one-third of that currently being used.

Beam-column joints designed as described above appear to have a higher sustained strength and a better ductile performance than the more conventional details, in spite of the fact that they have a very much lower shear steel area in the joint. It appears that higher shear stresses may be applied to the joint and, consequently, beam sizes may be reduced. At present, beam depths are large and steel percentages are kept low in order to find room for the joint steel and to reduce flexural forces. This leads to substantially increased costs due to material content, the labour involved in fixing steel in the highly congested joint zones, the difficulty of placing concrete in these zones and the increased height of construction required to accommodate the larger beam sizes.

With the bond plate joint detail the size of the joint zones may be reduced so that they no longer dictate the beam sizes. The next limit on beam size arises from considerations of sliding shear in beam plastic hinge zones. However, the authors believe that this potential failure mode may be prevented, as described below, thus allowing small beams containing up to 2.5% of flexural tension steel to be used with confidence, as compared with current practice that requires approximately 1.3%.

Mr. B. W. Buchanan, M.W.D., Wellington, has suggested that sliding shear failures could possibly be prevented by providing an element with a high dowel shear capacity in the centre of the beam.

A scaffolding tube filled with concrete and placed in a debonded state in the middle of the beam in the plastic hinge zones might satisfy this requirement. Alternatively, the authors believe it may be possible to control sliding shear by the use of some unstressed prestressing strands which are added to the normal flexural mild steel in the beam. This, it is thought, may change the character of the plastic hinge and increase the sliding shear capacity of the hinging region.

With the bond plate detail difficulties would arise with site placing of reinforcement, and it would be important to avoid this problem by having the steel cages prefabricated in a workshop. From discussions with contractors it appears that just this approach has been used to considerable advantage on several recent structures with conventional joints.

To reiterate, the mode of resistance of a bond plated joint is clear and a rational design may be made. The same does not appear to be true of designs made to ACI-318-71, or the MWD's PW 81/10/1. The ACI code does not require any vertical reinforcement in the joint zone, and the draft MWD code PW 81/10/1 requires only nominal vertical steel in that it is noted that column steel should be distributed round the perimeter of the column (see Table 1 in Reference 1).

Consequently it is difficult to determine joint stirrup requirements by a logical analysis of joint forces in columns with low axial loads. The expected mode of action is not clear, and the poor performance of Units 1 and 3 under these conditions is not surprising. Finally, it should be noted that the conditions for these tests were severe in that there was no axial load (a condition which leads to a high vertical stirrup requirement), and the ratio of section to displacement ductility was high.

4. ACKNOWLEDGEMENTS

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5. REFERENCES

(1) Fenwick, R. C. and Irvine, H. M., "Reinforced Concrete Beam-Column Joints for Seismic Loading, Part I - Theory", Bulletin of the N.Z. National Society for Earthquake Engineering, Sept. 1977. Vol. 10, No. 3.
(2) ACI-ASCE Committee 352, "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures", Proc. ACI, Vol. 73, No. 7, July, 1976.
(3) Park, R. and Paulay, T., "Reinforced Concrete Structures", J. Wiley and Sons, 1976.
(4) Priestley, M.J.N., "Testing of Two Reinforced Concrete Beam-Column Assemblies Under Simulated Seismic Loading", Report No. 5-75/1, MWD Central Laboratories, Wellington, 1975. (See, also, Bulletin of the N.Z. National Society for Earthquake Engineering, Vol. 8, No. 1, March, 1975, pp. 38-70.)
(5) ACI Committee 408 - "Bond Stress - The State of the Art", ACI Proceedings, Vol. 63, No. 11, 1966.

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TABLE 1

BASIC DATA FOR BEAM-COLUMN JOINT UNITS

ITEM	UNIT 1 (ACI-318-71)	UNIT 2 1ST WITH BOND PLATES	UNIT 3 PW 81/10/1 - MAY 76	UNIT 4 2ND WITH BOND PLATES.
Cylinder strength f'_c	42.9 MPa	35.6 MPa	39.3 MPa	40.4 MPa
Beam flexural steel (T & B)	3 - D20 mm	3 - D20 mm	6 - D12 mm	2 - D24 mm
f_y	280 MPa	317 MPa	318 MPa	291 MPa
$A_{st} = A_{sb}$*	942 mm^2	942 mm^2	678 mm^2	905 mm^2
width	200	200	200	200
effective depth	260	260	254	252
Beam steel % ρ	1.81	1.81	1.34	1.80
Joint shear steel	5 - R12 - 2 leg	11 - R6.5 - 2 leg	5 - R12 - 2 leg	12 - R6.5 - 2 leg
horizontal (ΣA_h)	5 - R10 - 2 leg	+4 - D12 bars	5 - R10 - 2 leg 5 - R6.5 - 2 leg	
ΣA_h	1916 mm^2	1182 mm^2	2248 mm^2	597 mm^2
Joint vertical steel	2 - D16 mm	2 - D16 mm	Nil	2 - D16 mm
in jt. zone (ΣA_v)	402 mm^2	402 mm^2		402 mm^2
Column - steel (yield f_y as for beams)	4 - D20 mm 4 - D16 mm 2 - D16 mm central	4 - D20 mm 4 - D12 mm 2 - D16 mm central	16 - D12 mm each side	4 - D16 mm 2 - D16 mm 2 - D16 mm central
A_s (total) in columns	2463 mm^2	2463 mm^2	1810 mm^2	2614 mm^2
width	250	250	250	250
effective depth	260	260	254	252
Steel ratios				
$\Sigma A_h/A_{st}$	2.03	1.25	3.32	0.66
$(\Sigma A_h + \Sigma A_v)/A_{st}$	2.46	1.68	3.32	1.10
*L_u-ACI-71 $\phi = 1.0$	50.1 kN	55.5 kN	40.8 kN	52.4 kN
* L_1 load at 1st yield of flexural steel	46.3 kN	52.4 kN	37.0 kN	44.9 kN
* L_2 measured load at ± 2D	49.4 kN	58.6 kN	43.9 kN	52.6 kN
Col. flexural ult. / beam " " (calculated)	1.31	1.31	1.33	1.44

* Notes

L_u - calculated ultimate flexural load on each beam (ACI-318-71 $\phi = 1.0$)

L_1 - calculated load on each beam at 1st yield of beam steel

L_2 - measured load on each beam at ductility two 1st cycle clockwise direction

A_{sb} - flexural steel bottom of beam, A_{st} - flexural steel top of beam

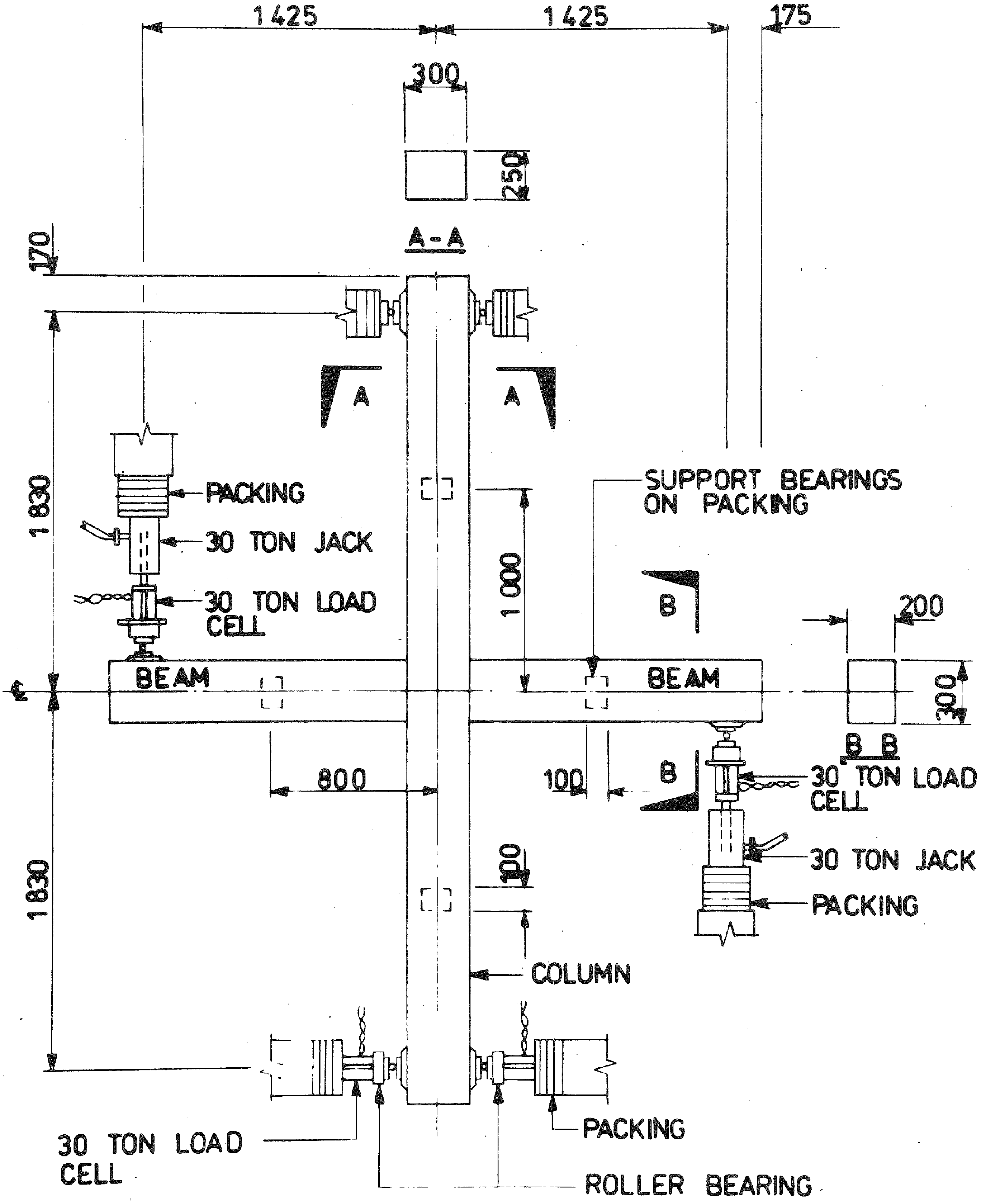


FIGURE 1: PLAN VIEW OF TEST ARRANGEMENT FOR BEAM-COLUMN JUNCTION SPECIMENS.

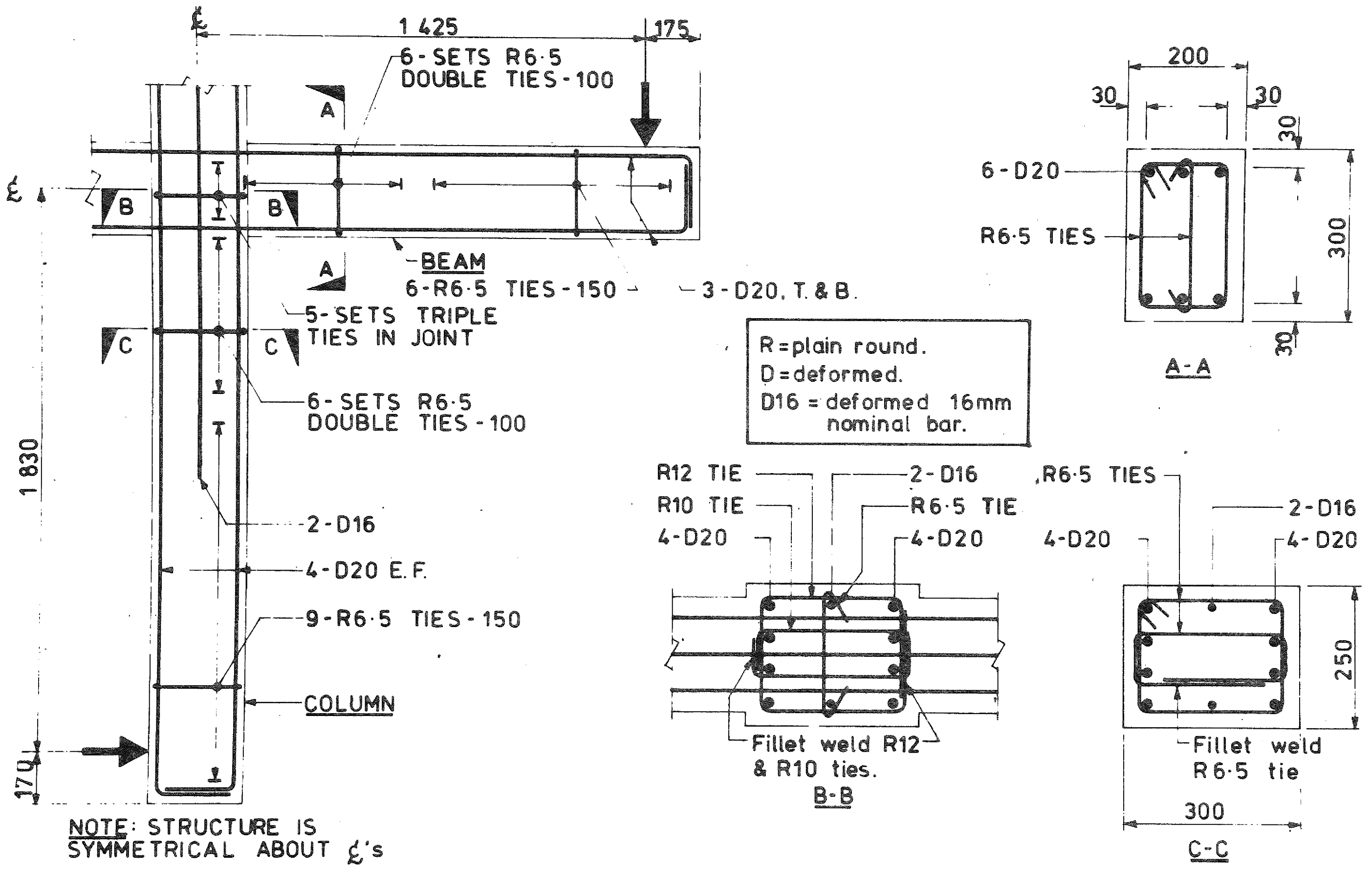


FIGURE 2(a): REINFORCEMENT DETAILS FOR THE FOUR SPECIMENS (a) UNIT 1

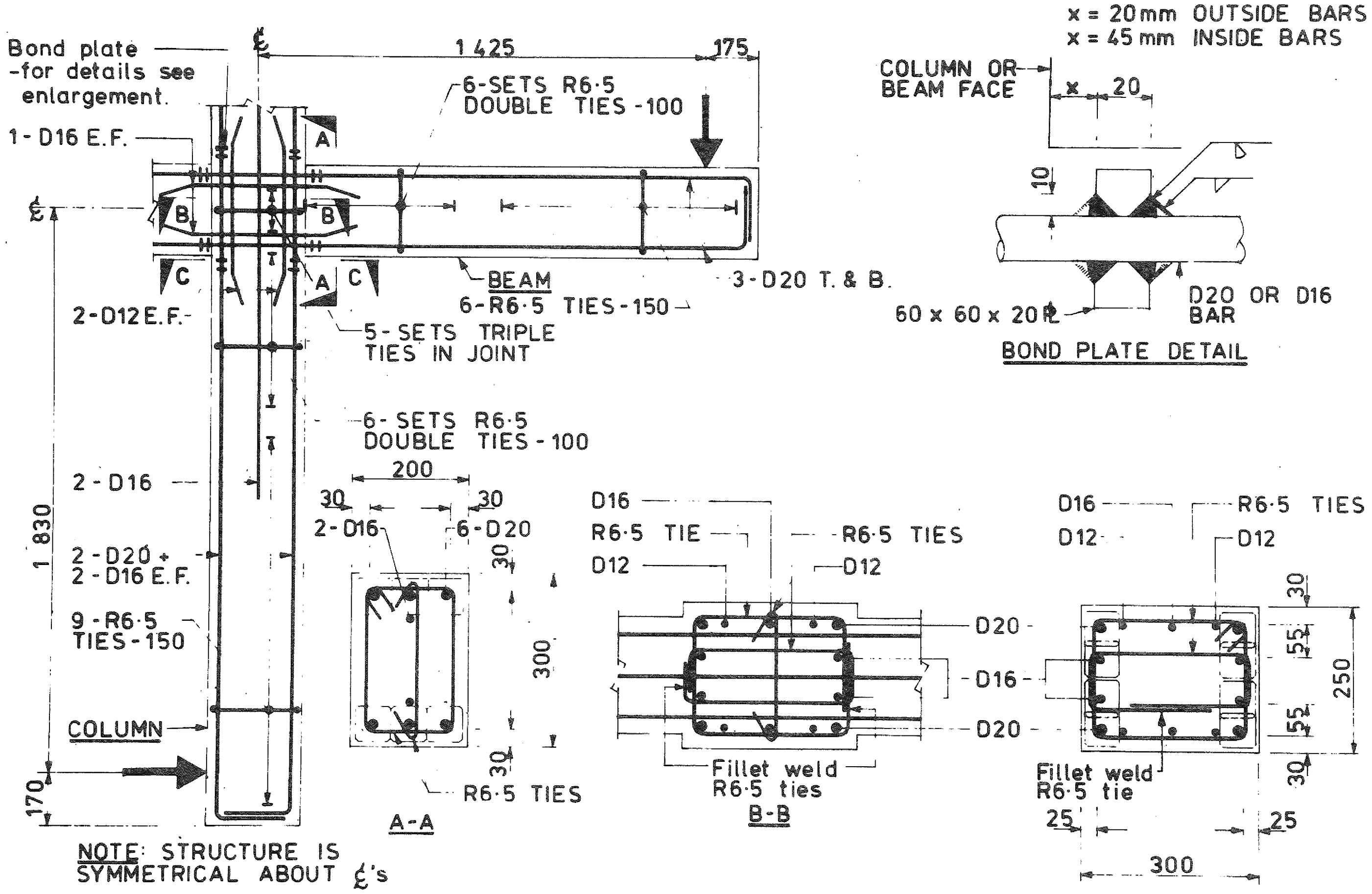
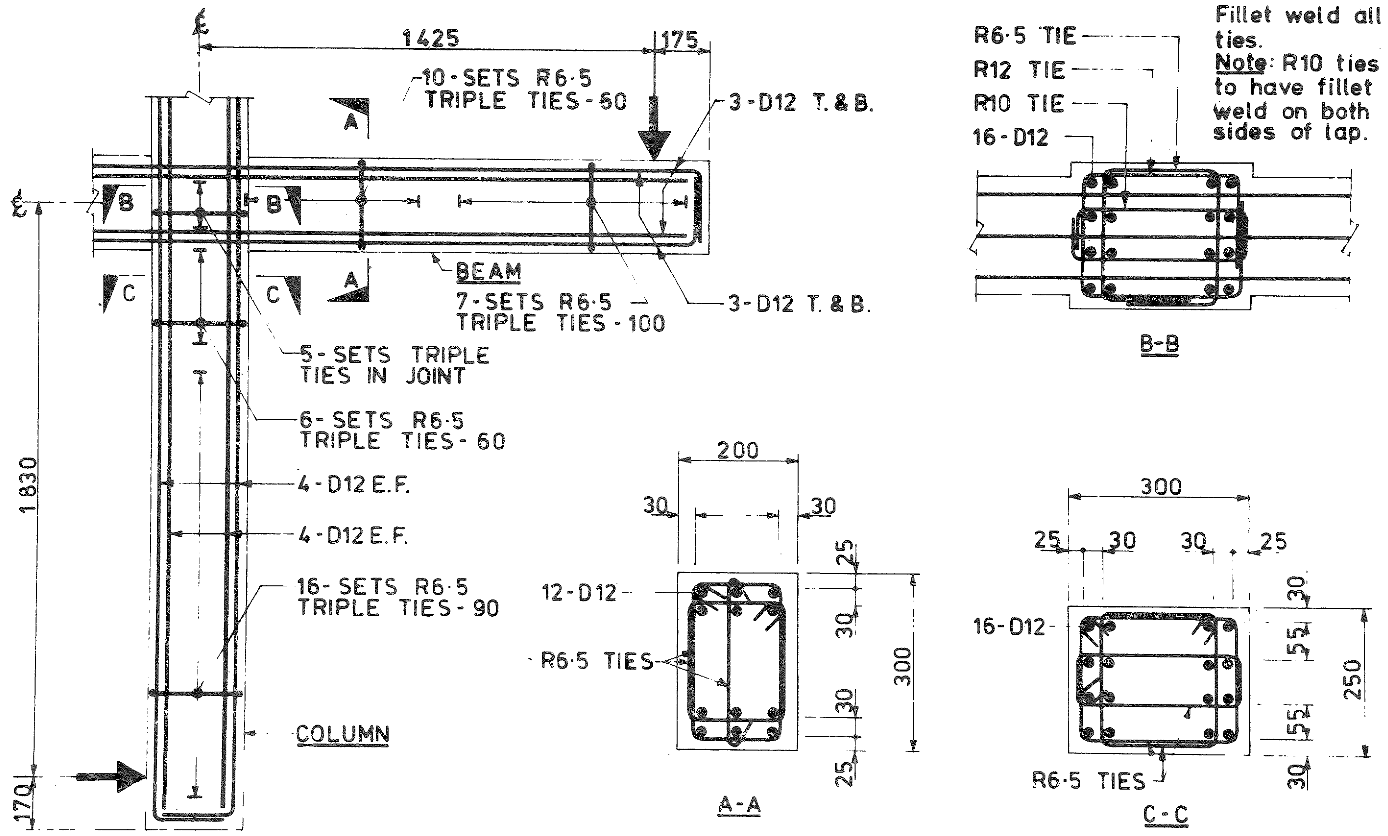
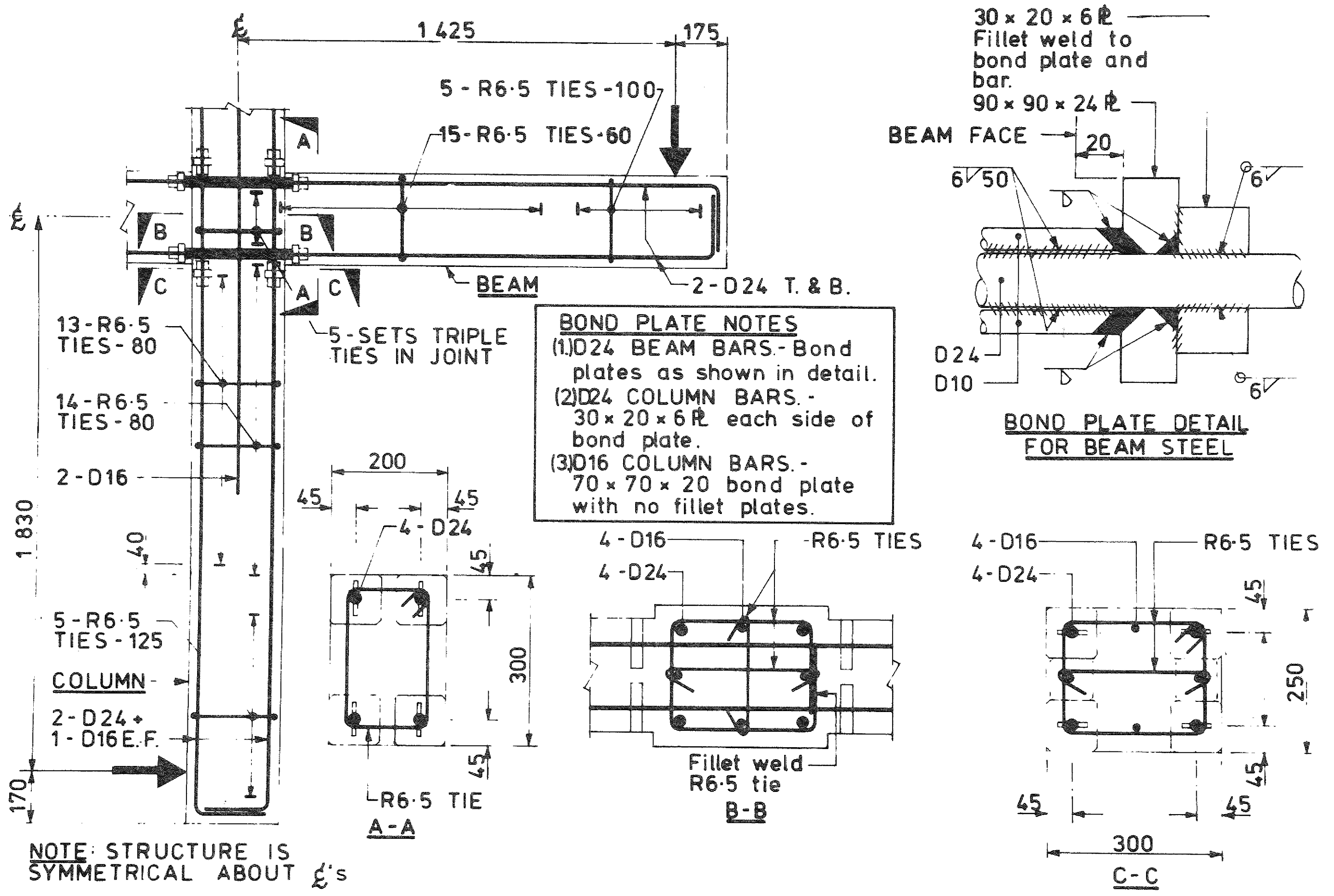


FIGURE 2(b): REINFORCEMENT DETAILS FOR THE FOUR SPECIMENS (b) Unit 2



NOTE: STRUCTURE IS SYMMETRICAL ABOUT ℄'s

FIGURE 2(c): REINFORCEMENT DETAILS FOR THE FOUR SPECIMENS (c) Unit 3



BOND PLATE NOTES
(1) D24 BEAM BARS.- Bond plates as shown in detail.
(2) D24 COLUMN BARS.- 30×20×6 ℞ each side of bond plate.
(3) D16 COLUMN BARS.- 70×70×20 bond plate with no fillet plates.

NOTE: STRUCTURE IS SYMMETRICAL ABOUT ℄'s

FIGURE 2(d): REINFORCEMENT DETAILS FOR THE FOUR SPECIMENS (d) Unit 4

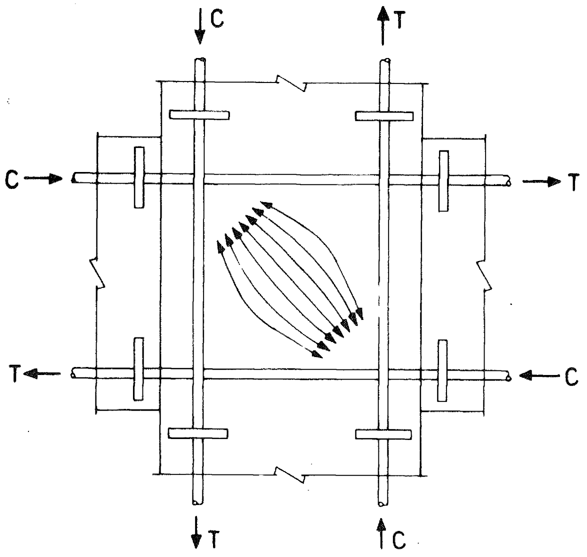
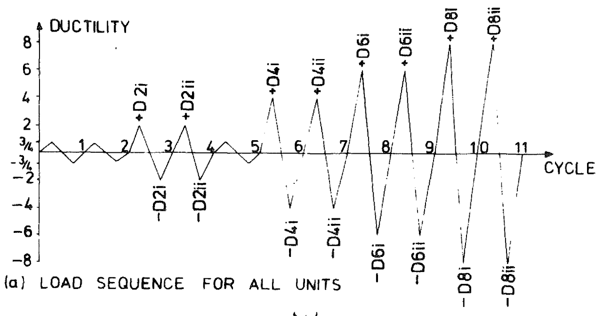
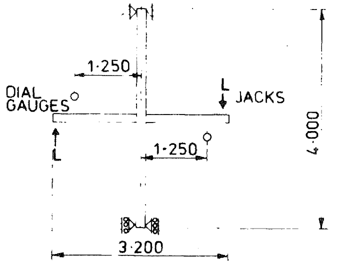


FIGURE 3: DIAGONAL STRUT ACTION FOR BOND PLATE SPECIMENS



(a) LOAD SEQUENCE FOR ALL UNITS



(b) SKETCH SHOWING DIAL GAUGE POSITIONS ON BEAMS

FIGURE 4

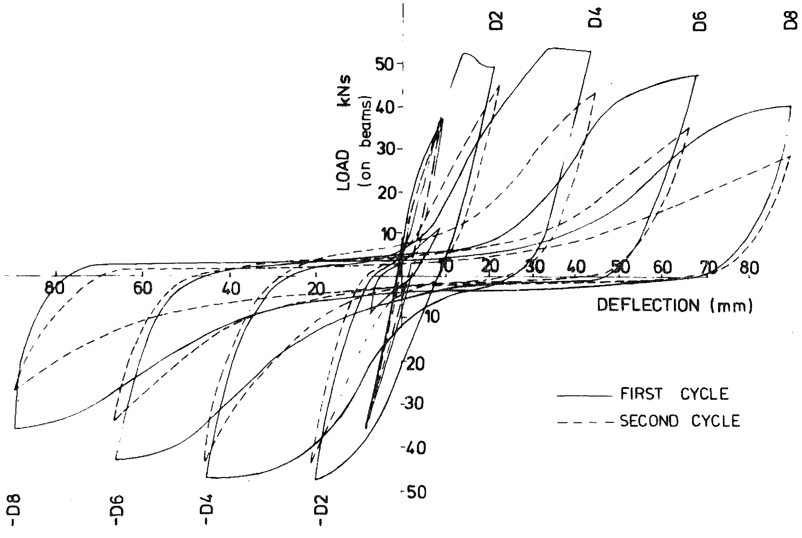


FIGURE 5: LOAD DEFLECTION CURVES FOR UNIT 1

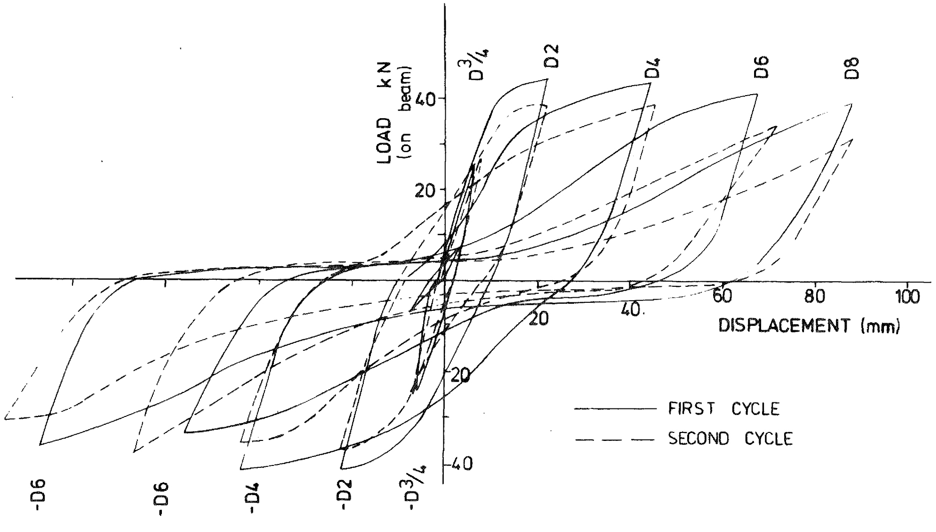


FIGURE 6: LOAD DEFLECTION CURVES FOR UNIT 3

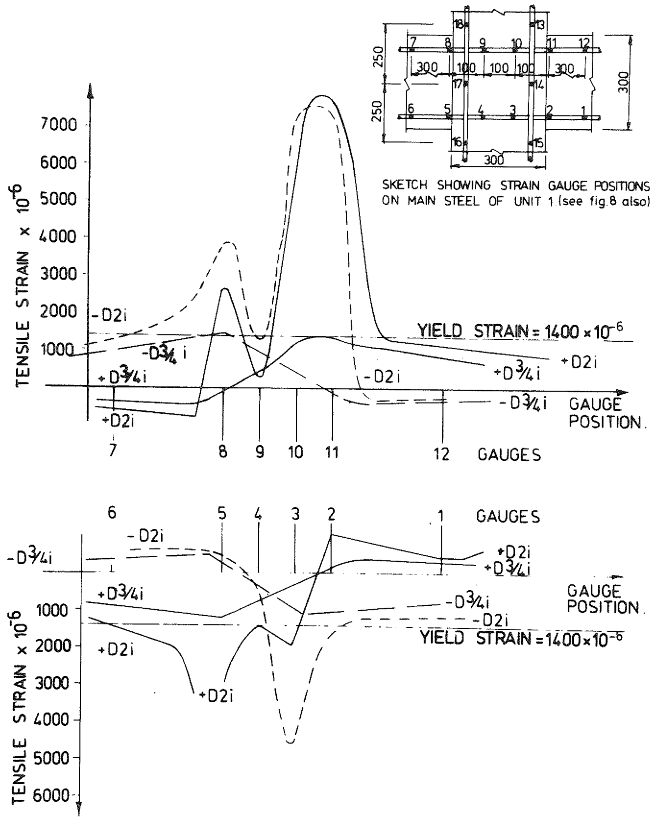


FIGURE 7: STRAIN READINGS ON BEAM BARS UNIT 1 UP TO DUCTILITY 2

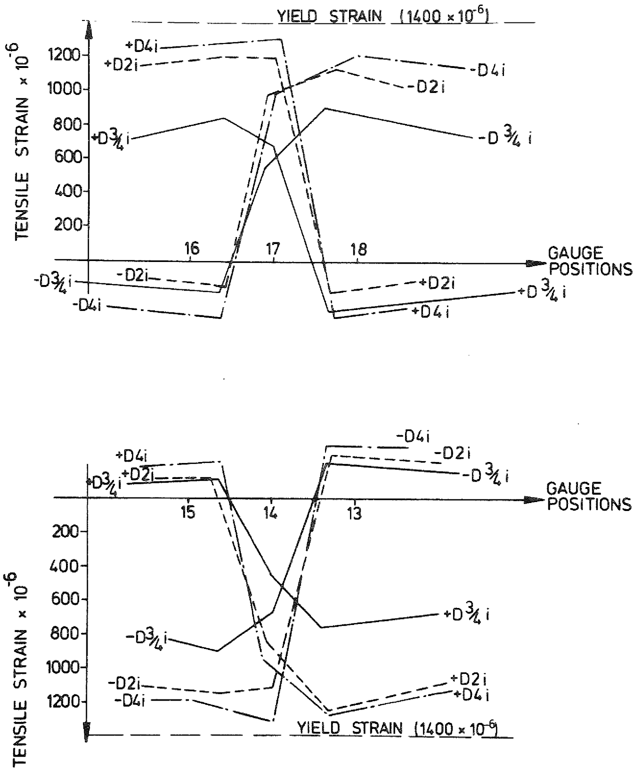


FIGURE 8: STRAIN READINGS ON COLUMN BARS UNIT 1 UP TO DUCTILITY 4 (SEE FIGURE 7 FOR STRAIN GAUGE LOCATIONS)

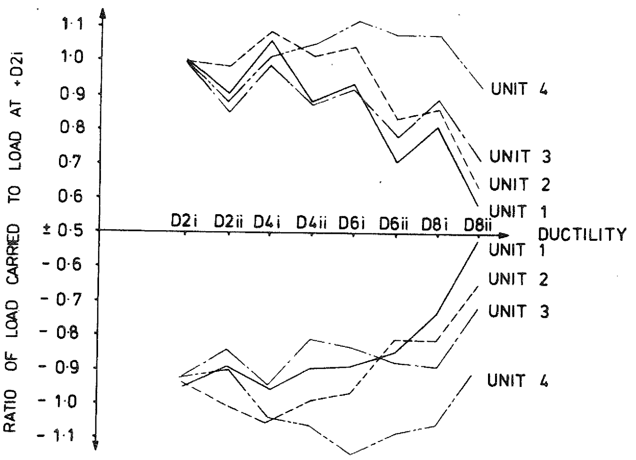


FIGURE 9: LOAD SUSTAINED AT DIFFERENT DUCTILITIES (i= first cycle, ii=second cycle)

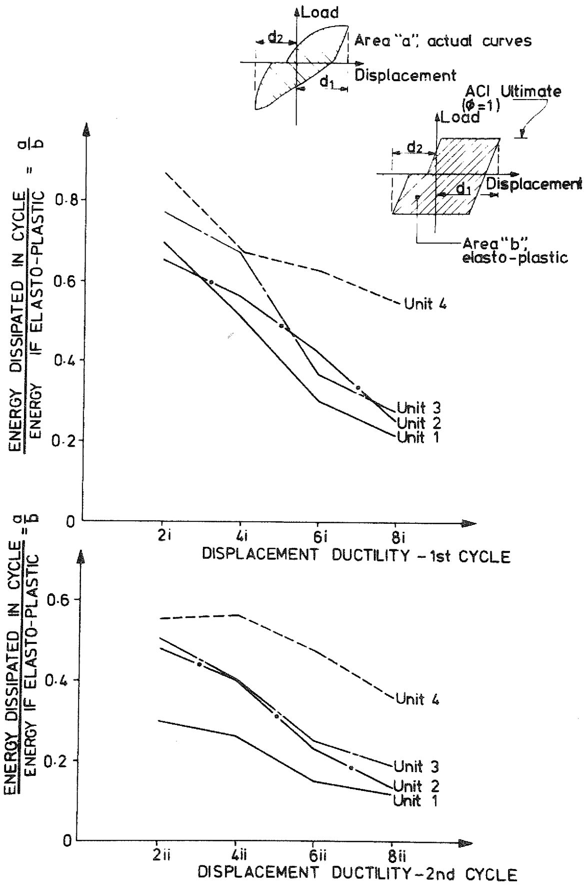


FIGURE 10: ENERGY DISSIPATION VERSUS DUCTILITY

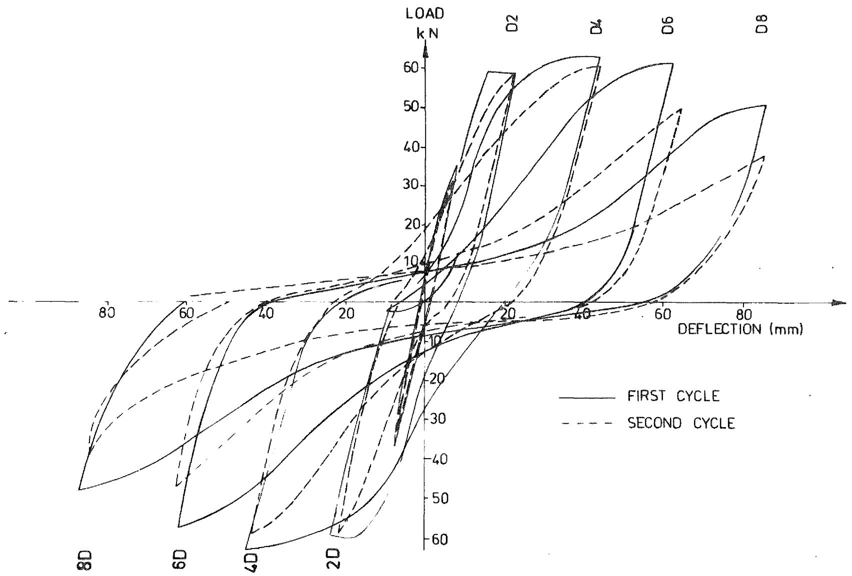


FIGURE 11: LOAD DEFLECTION CURVES FOR UNIT 2

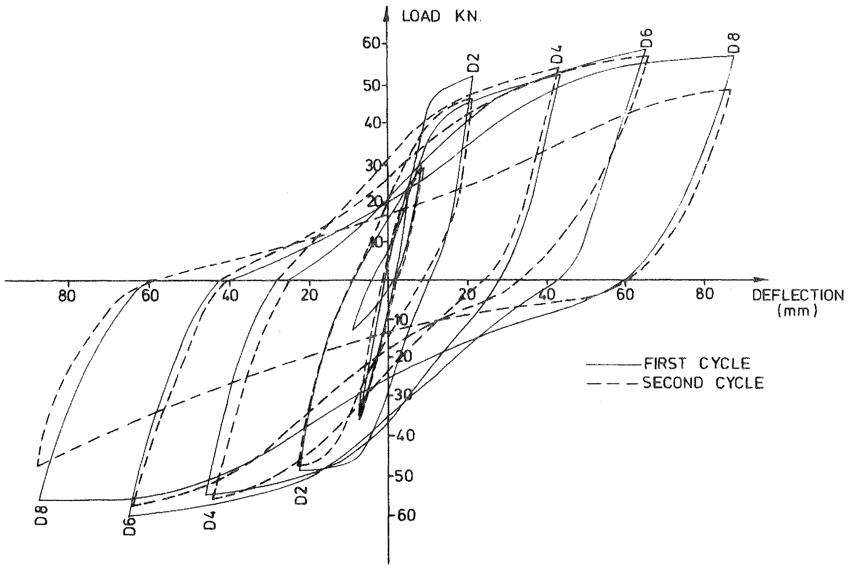


FIGURE 12: LOAD DEFLECTION CURVES FOR UNIT 4
(COMPARE WITH FIGURES 5, 6 AND 11)

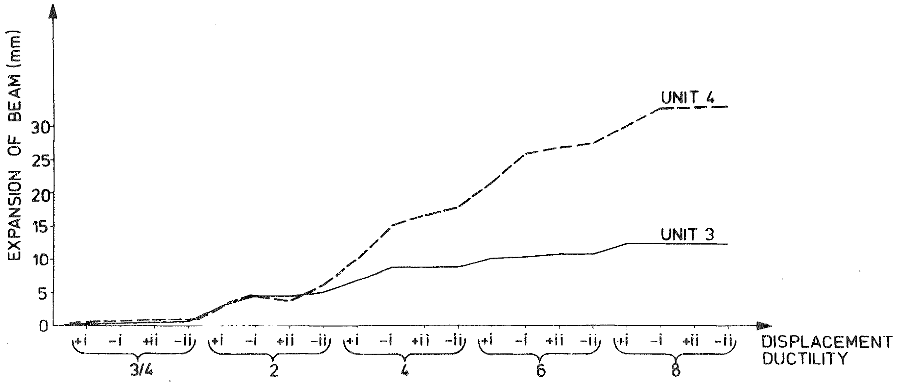


FIGURE 13: EXPANSION OF BEAM IN UNITS 3 AND 4
UNDER CYCLIC LOADING
MEASURED OVER CENTRAL 1.600m OF BEAM

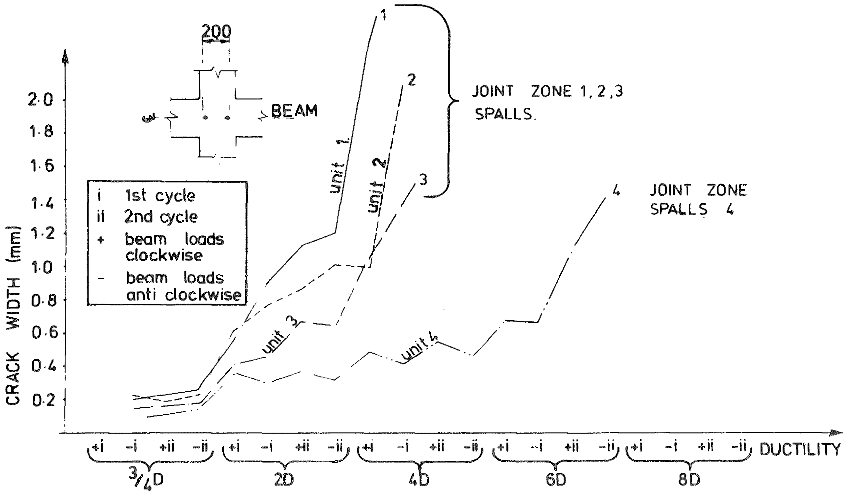


FIGURE 14: CRACK WIDTH IN JOINT MEASURED BY
DEMEC GAUGE ON BEAM