BEHAVIOUR OF A PRESTRESSED CONCRETE INTERIOR BEAM-COLUMN ASSEMBLY UNDER CYCLIC LOADING: UNIT 1 TEST RESULTS

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1. INTRODUCTION

The University of Canterbury is at present conducting an experimental investigation into the seismic resistance of concrete building frames. This research project is sponsored by the N.Z. Prestressed Concrete Institute, the Building Research Association (N.Z.) and the University. The test frames will include a range of proportions of prestressing steel and ordinary reinforcing steel to allow a comparison of prestressed and reinforced concrete frames and to establish the possible advantages of combining both systems. The tests are aimed at determining the deformation capacity and degree of damage of such frames when responding to severe seismic load reversals and will establish further basic information for the evaluation and design of framed structures for earthquake resistance.

Testing of the first test specimen has been completed. The specimen (Unit 1) consisted of a beam-column assembly with a prestressed concrete beam. After testing under very severe seismic type loading the damaged concrete was repaired and the assembly retested. This report summarizes the results obtained from the tests on that specimen.

2. DETAILS OF UNIT 1

2.1 Overall Dimensions and Loading

The beam-column test unit represented the part of the building frame shown encircled in Fig. 1. The unit can be regarded as being the part of the frame between the points of contraflexure at a typical interior joint. The unit was loaded as shown in Fig. 2 by an axial load P on the column representing load due to the weight of the building, floor loads, and overturning moment, and by vertical loads on the ends of the beams representing shear induced by earthquake loading. The applied beam loads induce reactive shears at the ends of the column. By reversing the direction of the vertical loads the effects of earthquake shaking was simulated. The vertical loading was such as to enforce deformations into the inelastic range several times following a loading cycle representing the action of a major earthquake. The cyclic loading was applied slowly over a time period of days rather than rapidly over a time period of seconds as would occur in an actual earthquake. However it is considered that the results obtained from the slow load reversals of the tests give a conservative indication of the response of a concrete frame to actual earthquake shaking.

Fig. 3 shows the overall dimensions of Unit 1 and of the other beam-column units of the initial series of tests. The cross-sections can be considered to be representative of full-size members of frames with small numbers of storeys, or to be near full-size. The second moment of area of the beam section is approximately equal that of the column section. Both the beam and the column are continuous through the joint. The steel content is such that the column section is stronger than the beam so that under very severe seismic loading plastic hinging is enforced in the beam rather than in the column. Thus the critical sections are in the beam at the column faces.

2.2 Steel Details and Material Properties

The beam and column sections of Unit 1 are shown in Fig. 4. The beam was prestressed by three 12/0.20 in dia. high tensile steel tendons; the columns were reinforced longitudinally by HY60 steel bars. Both members contained mild steel transverse reinforcement for shear and confinement of concrete. Fig. 5 shows the steel in the joint region before the concrete was placed.

The prestressing steel had a 0.2% proof steel stress of 210,000 psi, an ultimate tensile strength of 236,000 psi and an ultimate strain of 5.4%. The yield stresses of the reinforcing steel were 58,800 psi for the HY60 column steel (based on the nominal bar area), 43,500 psi for the ⅝ in dia. joint hoops, 41,900 psi for the ⅜ in dia. ties and stirrups, and 40,900 psi for the ⅜ in dia. beam steel.

The concrete had an aggregate: cement:water ratio of 6.8 : 1 : 0.55 by weight, a maximum aggregate size of ¾ in , and a slump of 4¼ in. When the testing of the beam-column unit was completed the concrete was 76 days old and at that stage the compressive strength (12 in x 6 in dia. cylinders, air dried after one week of curing with the unit) was 4630 psi, and the modulus of rupture was 837 psi. The concrete strength was lower than expected; 5500 psi at the time of testing had been aimed at.

After the unit has been tested the damaged concrete in the beams was chipped away and replaced. The new concrete had an aggregate: cement:water ratio of 4.6 : 1 : 0.48 by weight, a maximum aggregate size of ⅜ in, and a slump of 3 in. When the repaired unit was tested the new concrete was 66 days old and had a compressive strength (12 in x 6 in dia. cylinders, air dried after one week of curing with the replaced concrete) of 7400 psi.

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2.3 Prestress and Theoretical Member Strengths

The prestress in the beam and the theoretical strengths of the members calculated using the actual strength of the materials (original concrete and steel) are shown in Table 1. The flexural strength calculations were based on the assumptions for concrete compressive stress distribution and strain given in the ACI code(1); the actual stress-strain curves of the steel were used and the flexural strength of the sections found by satisfying force equilibrium and strain compatibility. The shear strengths were calculated using equations from the ACI code(1). The capacity reduction factor ϕ was assumed to be 1.0 in the calculations.

3. TEST RESULTS FROM UNIT 1

3.1 General

Unit 1 was subjected to a series of very severe earthquake type load cycles as previously described which caused deformations well into the inelastic range. The damage in the members after very severe loading occurred mainly in the beams. After testing the beams were repaired by removing the damaged concrete and replacing it by new concrete to give the original cross sectional dimensions. The repaired unit was then retested. The vertical deflections applied to the ends of the beams during the initial test and the test after repairs are shown in Fig. 6. It is to be noted that a comparable reinforced concrete assembly would undergo a beam end deflection of approximately 1.1 in when yielding in the beam commenced, assuming fully cracked beams and columns. An indication of the expected maximum deflections during a severe earthquake, as a function of the yield deflection for reinforced and prestressed concrete frames, has been given previously by Blakeley and Park(2). The axial load applied to the column during testing was 100 tons.

During the loading cycles, the pin ends of the columns were held against lateral displacement and deflections enforced at the ends of the beams. The deflections at the ends of the beams and elsewhere were measured by dial gauges and rules. Strains were measured on the surface of the concrete and on some of the reinforcement. Rotations were measured at the plastic hinge positions in the beams by dial gauges attached to frameworks around the beams at 4 in and 12 in out from the column faces. Fig. 7 shows Unit 1 at the start of testing.

3.2 Load-Deformation Behaviour

Fig. 8 shows the vertical displacement at the end of the left beam plotted against the beam moment at the column face. The numbers on the curves correspond to the peak of the loading cycles illustrated in Fig. 6. In loading cycles 1 to 4 the deformations were within the elastic or cracked elastic range of the beams. In these cycles the area within the load-deflection loops in Fig. 8 was small, indicating little energy dissipation during these initial cycles, and the recovery of deflection on releasing the load was almost complete. In load cycle 5 the beam reached its maximum moment capacity and crushing of compressed concrete commenced at the critical beam sections. On unloading some residual deflection was shown by the beam. The beam reached its maximum moment capacity in the other direction in load cycle 6 and crushing of compressed concrete also commenced in that cycle. With further loading cycles stiffness degradation occurred accompanied by a reduction in the moment capacity of the beam. Fig. 9 shows a plot of the moment-curvature characteristics measured over the 12 in gauge length adjacent to the column face in the left beam. Comparison of Figs. 8 and 9 show that the moment-curvature and moment-end deflection characteristics are very similar indicating the large contribution to the end deflection made by deformation at the plastic hinge in the beam close to the column face.

The degradation in stiffness and reduction in moment carrying capacity indicated in Figs. 8 and 9 occurred after crushing of concrete commenced and was due to the concrete cover surrounding the beam stirrups breaking away. The region of damaged concrete also penetrated some way into the core concrete between the stirrups. Figs. 10 and 11 show the test unit after crushing of the beam concrete had commenced and the reduced cross section due to crushing is evident.

The maximum bending moments reached in the left hand beam (at the column face) were 1626 kip in at a downward beam end deflection of 2.0 in and 1538 kip in at an upward beam end deflection of 1.9 in. These two experimental ultimate moment capacities were 9% and 3% greater than the theoretical value of 1495 kip in (see Table 1). It should be noted that a moment higher than the theoretical value was sustained at the column face because of the extra confinement given to the beam concrete there by the adjacent column and its reinforcement. The beam section where crushing of concrete occurred was actually a short distance (a few inches) away from the column face (see Figs. 10 and 11) and the experimental moment at that section agreed more closely with the theoretical ultimate moment. At the maximum imposed downward beam end deflection of 4.9 in in cycle 11 the moment capacity had decreased to 1038 kip in (64% of the maximum value); at the maximum imposed upward beam end deflection of 4.8 in in cycle 12 the moment capacity had decreased to 1067 kip in (69% of the maximum value).

3.3 General Observations

Fig. 10 shows the unit when the maximum moment capacities had been reached in each direction. The cracks have been marked with a felt tipped pen to make then more visible in the photographs. Features at this stage are:

1. Diagonal tension cracks existed in the joint region. These were caused by the horizontal shear forces resulting from the tensile forces in the prestressing tendons, the compressive forces in the concrete and the shear force in the column. However these cracks were of small width and were well controlled by the shear reinforcement in the joint.

2. There was an absence of diagonal tension shear cracks in the beam. This was to be expected as the shear force in the beam at the maximum moment was less than the theoretical shear capacity of the concrete.

3. Longitudinal cracks existed in the compression zones of the beam near the joint due to crushing

of concrete. These resulted in a subsequent reduction in moment capacity.

Fig. 11 shows the unit when the maximum deflections had been imposed in each direction. Features at this stage are:

1. The extensive reduction in concrete section area in the beams near the joint due to the loss of cover concrete outside and between the stirrups in the compression regions.

2. The intact nature of the joint region which, despite the formation of plastic hinges in the beam each side of the joint, only had slight diagonal tension cracks.

3. The buckling of the ⅞ in dia. longitudinal beam steel between the stirrups in the plastic hinge region.

4. REPAIR AND RETESTING OF UNIT 1

4.1 General

After the testing of Unit 1 it was decided to repair the damaged concrete and to retest the unit to see if satisfactory structural behaviour could be achieved. The damaged beams were first jacked back to the horizontal position. (in practice this "straightening" of the structure may be the most difficult part of the repair operation). The damaged concrete in the plastic hinge regions was then chipped away (see Fig. 12) and two ⅜ in dia. stirrups were placed around the remaining core of the beams in each damaged region. A mould was placed around the beam and new concrete was cast to give the original cross-sectional dimensions. After a time interval for curing and gaining strength the repaired test unit was loaded through the sequence shown in Fig. 6.

4.2 Load-Deformation Behaviour

Fig. 13 shows the vertical displacement at the end of the left beam plotted against the beam moment at the column face. Comparison with Fig. 7 shows that the initial stiffness of the repaired unit was less than that of the original unit. This would have been due to the early cracking of the new concrete and the presence of cracks in regions of the original concrete. It is to be noted that the new beam concrete was not prestressed. The tendons merely acted as ordinary reinforcement in that concrete. Cracking was not excessive in the new concrete at service load levels. As with the original test, the maximum moment capacity in each direction was reached in load cycles 5 and 6 and crushing of compressed concrete commenced during those load cycles. With further load cycles a degradation of strength and stiffness occurred but the percentage reduction in strength was less than for the original test. Fig. 14 shows a plot of the moment-curvature characteristics measured for the 12 in plastic hinge length adjacent to the column face in the left beam.

The maximum bending moments reached in the left hand beam were 1288 kip in at a downward beam end deflection of 2.5 in and 1447 kip in at an upward beam end deflection of 2.4 in. These experimental moment capacities were 86% and 97% of the original theoretical value of 1495 kip in. However, in this test the top and bottom tendons were not prestressed, plastic strains in the initial test having reduced the prestress to near zero, and therefore the forces in the tendons at the ultimate moment would have been different from the previous test. It is difficult to calculate the theoretical ultimate moment for the repaired unit without knowing the initial steel strains and the modified stress-strain curves for the steel after cyclic loading. However the theoretical ultimate moment of the repaired beam would not be significantly different from the value for the original beam.

At the maximum imposed downward beam end deflection of 5.6 in in cycle 9 the moment had decreased to 1090 kip in (85% of the maximum experimental value); at the maximum imposed upward beam end deflection of 4.4 in in cycle 8 the moment was 1246 kip in (86% of the maximum experimental value). These percentage reductions in moment show that the repaired unit with the inclusion of two extra stirrups in the plastic hinge region was able to maintain a higher percentage of its maximum moment capacity than the original unit, thus demonstrating the benefit of the extra confining steel in the beam.

4.3 General Observations

Fig. 15 shows the repaired unit when the maximum moment capacities had been reached in both directions. Fig. 16 shows the repaired unit when the maximum deflections had been imposed in each direction. Features of interest are:

1. The joint region is undamaged apart from small diagonal tension cracks.

2. The better confinement of the core concrete of the beams near the joint due to the presence of more stirrups than in the original unit.

5. DISCUSSION AND CONCLUSIONS

The original unit showed an ability to be loaded to near its maximum moment capacity and then to be unloaded with negligible residual damage. The maximum moment capacities reached in the beams at the column faces exceeded the theoretical flexural strength by up to 9%. Subsequent to the commencement of crushing of the compressed concrete the energy dissipation capacity was large. Degradation of stiffness and strength occurred after crushing of the concrete had commenced because of the reduction in the cross-sectional area of the beams. A similar result has been obtained from previous tests by Blakeley and Park[3].

It is evident that the reduction in stiffness and strength after crushing of the concrete had commenced could have been reduced had more transverse steel (stirrups) been present in the beam plastic hinge region. Sufficient shear reinforcement had been provided for all the beam shear to be carried by the stirrups alone, and this resulted in ⅜ in dia. stirrups at 7 in centres (0.39 of the overall section depth). Inspection of the member showed that had the stirrup spacing been smaller the loss of concrete section between stirrups would not have been so great thus allowing the plastic hinge to maintain a higher moment capacity at large plastic deformations. More tests are needed on this aspect but it is evident that a stirrup spacing of approximately 4 in should not be exceeded in beams of these dimensions for good ductile behaviour. It is also evident that the presence of compression steel would have improved the beam ductility.

The shear reinforcement provided in the members and the joint resulted in satisfactory shear resistance. In the joint the maximum horizontal shear force (beam internal forces minus column shear force acting above or below a horizontal plane in the critical region of the joint, i.e. $T_1 + T_4 - T_3 + C + C_5 - V$ for plane 1 of Fig. 17) was 212 kips at the theoretical flexural strength of the beam, whereas the theoretical shear strength of the joint was 217 kips according to the ACI code (see Table 1). It is evident that the ACI design approach resulted in satisfactory joint strength in the case of this structure. However this should not be taken as general approval of the ACI procedure which only crudely approximates the mechanism of joint shear transfer. It should be pointed out that the central prestressing tendon was probably of great assistance in controlling the width of the diagonal tension cracks and in maintaining the shear resistance of the joint. This would be so because of the residual prestress in the central tendon, even after large deformations of the beam, and because the tendon crossed the cracks. This effect of the central tendon will be checked in later tests by comparison with a similar test unit without a central tendon.

The repair of the unit by straightening the damaged members, removing the damaged concrete in the plastic hinge region and replacing the concrete there, demonstrated that it is possible to repair prestressed concrete members. The two extra stirrups placed within the new concrete at each repaired plastic hinge region helped confine the concrete there and the performance of the repaired unit in the subsequent load testing was satisfactory. It is emphasized that the new concrete was not prestressed. The prestressing tendon there functioned merely as ordinary reinforcement. However, the general performance of the repaired unit was satisfactory although as expected the crack control in the new concrete was not as good as in the original unit. For instance, at 63% of the experimental ultimate moment the maximum crack width in the new concrete was approximately 0.015 in, whereas in the original prestressed beam that crack width was not reached until 98% of the experimental ultimate moment had been applied.

6. ACKNOWLEDGEMENTS

The tests reported were carried out in the laboratories of the Department of Civil Engineering, University of Canterbury. The sponsorship of the N.Z. Prestressed Concrete Institute and the Building Research Association (N.Z.) is gratefully acknowledged. The authors are particularly indebted to technicians A. G. Foot and J. M. Adams for assistance with the experimental work.

7. REFERENCES

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71)", American Concrete Institute, Detroit, Michigan, 1971, pp. 78.
2. Blakeley, R.W.G. and Park, R., "Response of Prestressed Concrete Structures to Earthquake Motions", New Zealand Engineering, Vol. 28, No. 2, February 1973, pp. 42-54.
3. Blakeley, R.W.G. and Park, R., "Seismic Resistance of Prestressed Concrete Beam-Column Assemblies", Journal of the American Concrete Institute, Proc. Vol. 68, No. 9, September 1971, pp. 677-692.

TABLE 1.

PRESTRESS AND THEORETICAL STRENGTH OF MEMBERS

Stress in concrete of beam due to prestress alone at time of transfer	1160 psi
Flexural Strength of beam	1495 kip in
Flexural strength of column (axial load was 224 kips)	2045 kip in
Shear strength of beam ($2\sqrt{f'_c}bd + A_v f_y d/s$)	36.6 kips
Shear strength of column ($3.17\sqrt{f'_c}bd + A_v f_y d/s$)	53.4 kips
Shear strength of joint ($3.17\sqrt{f'_c}bd + A_v f_y d/s$)	217 kips

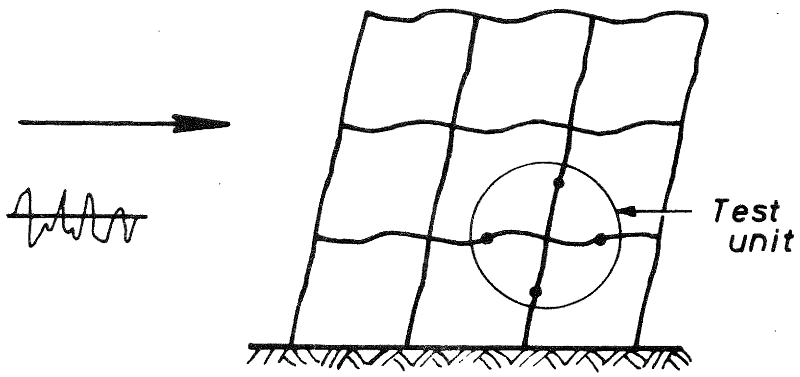


Fig. 1 Building Frame Subjected to Earthquake Motions.

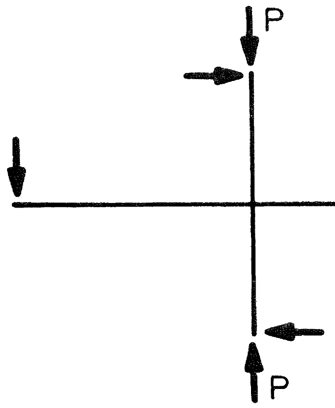


Fig. 2 Loading Simulating the Action of an Earthquake on the Beam-Column Test Unit.

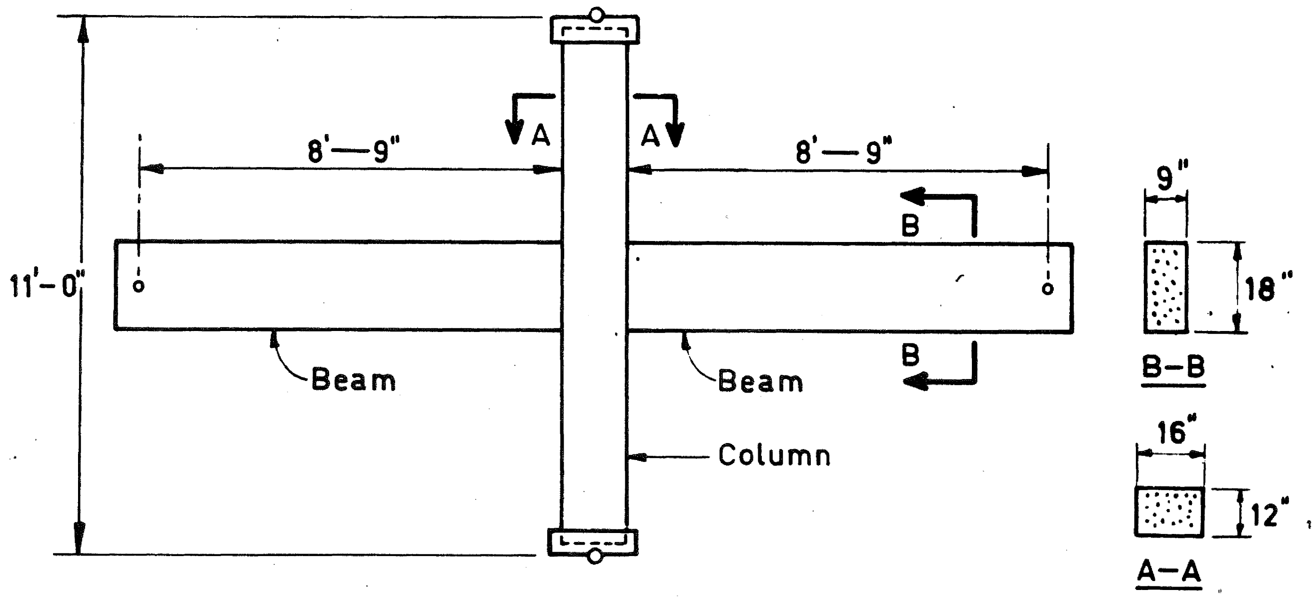


Fig. 3 Dimensions of Beam-Column Unit 1 and Other Test Units of the Initial Series of Tests.

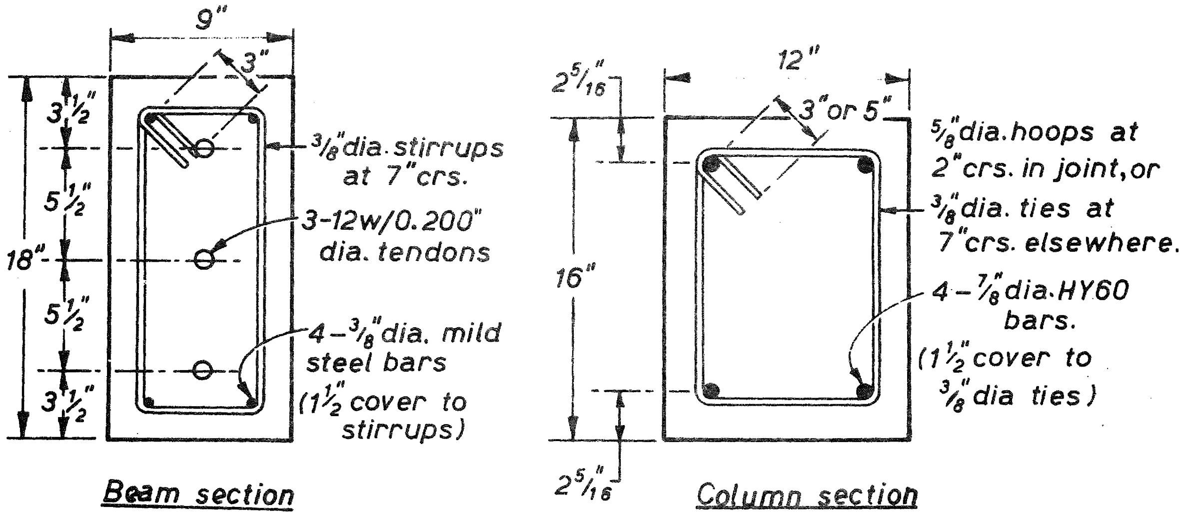


Fig. 4 Unit 1 : Beam and Column Steel Details.

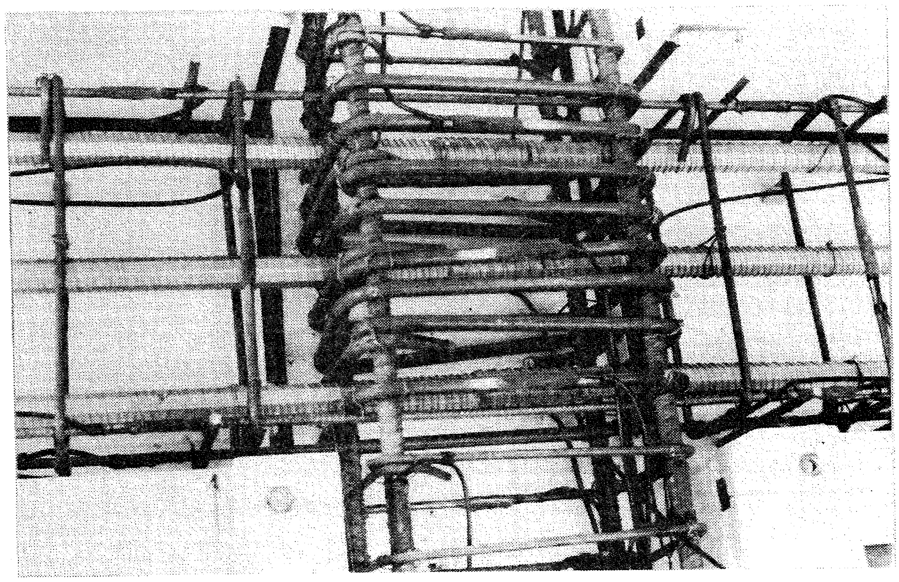


Fig. 5 Steel in Joint Region Before Placing Concrete.

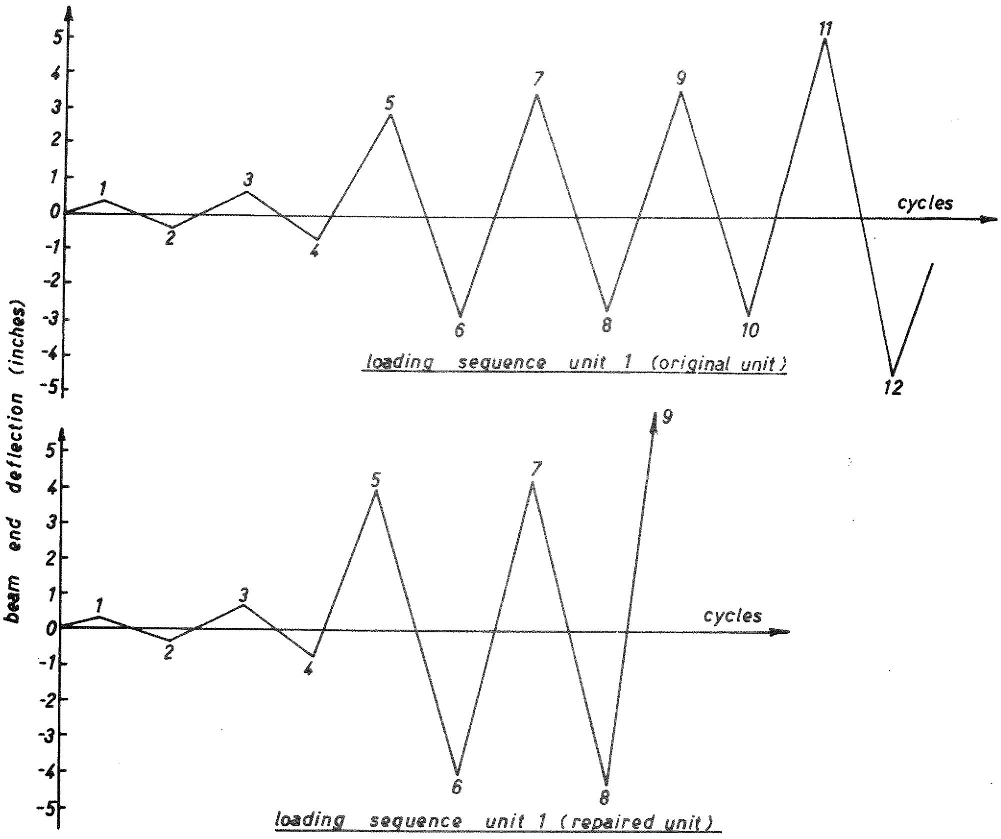


Fig. 6 Deflections of Beam Ends During Loading Cycles.

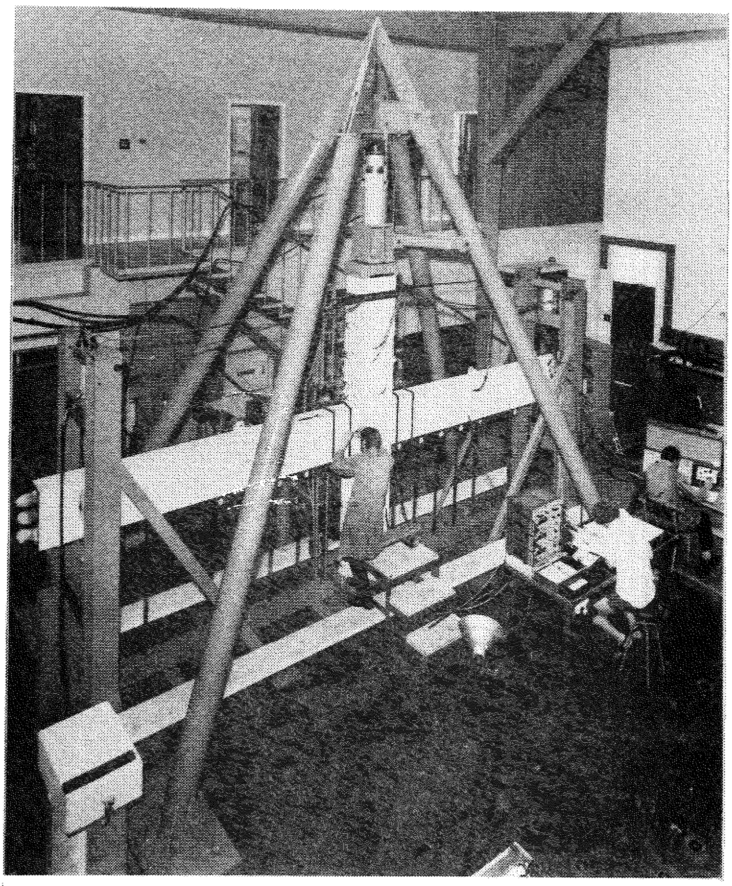


Fig. 7 Unit 1 in Test Frame at Start of Testing.

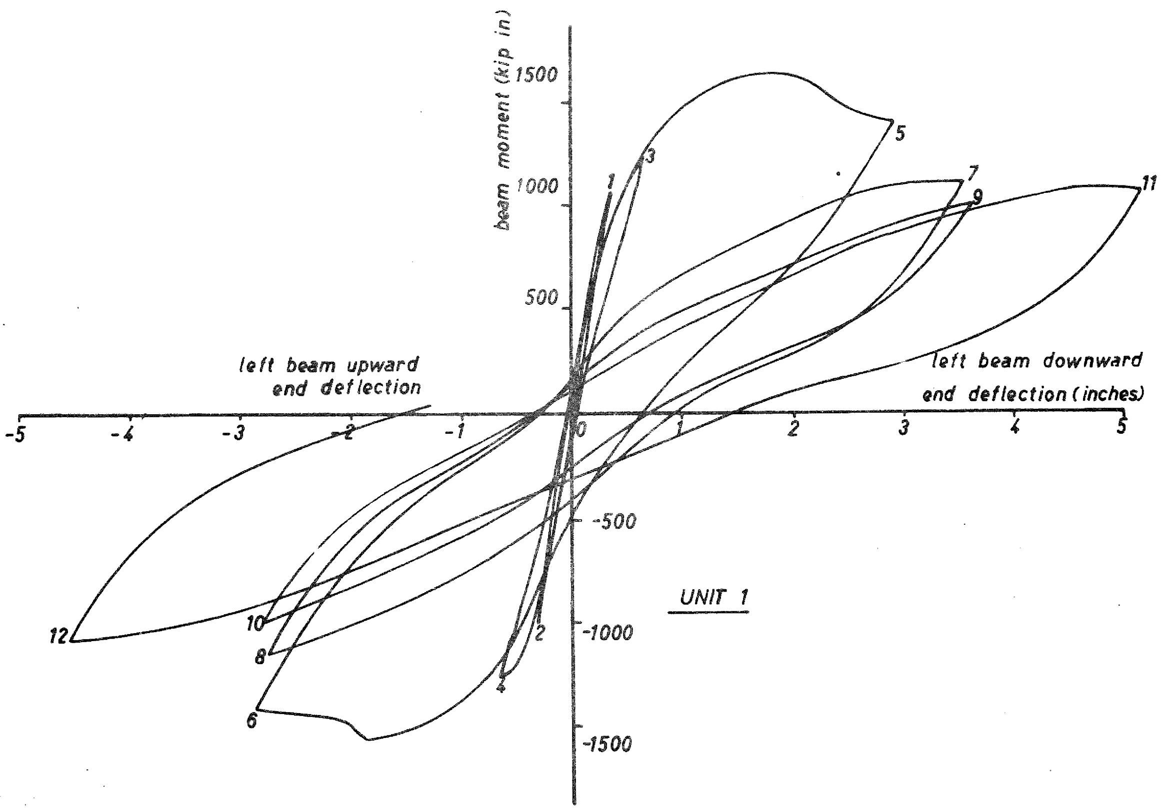


Fig. 8 Unit 1 Beam Moment at Column Face versus Vertical End Deflection of Beam.

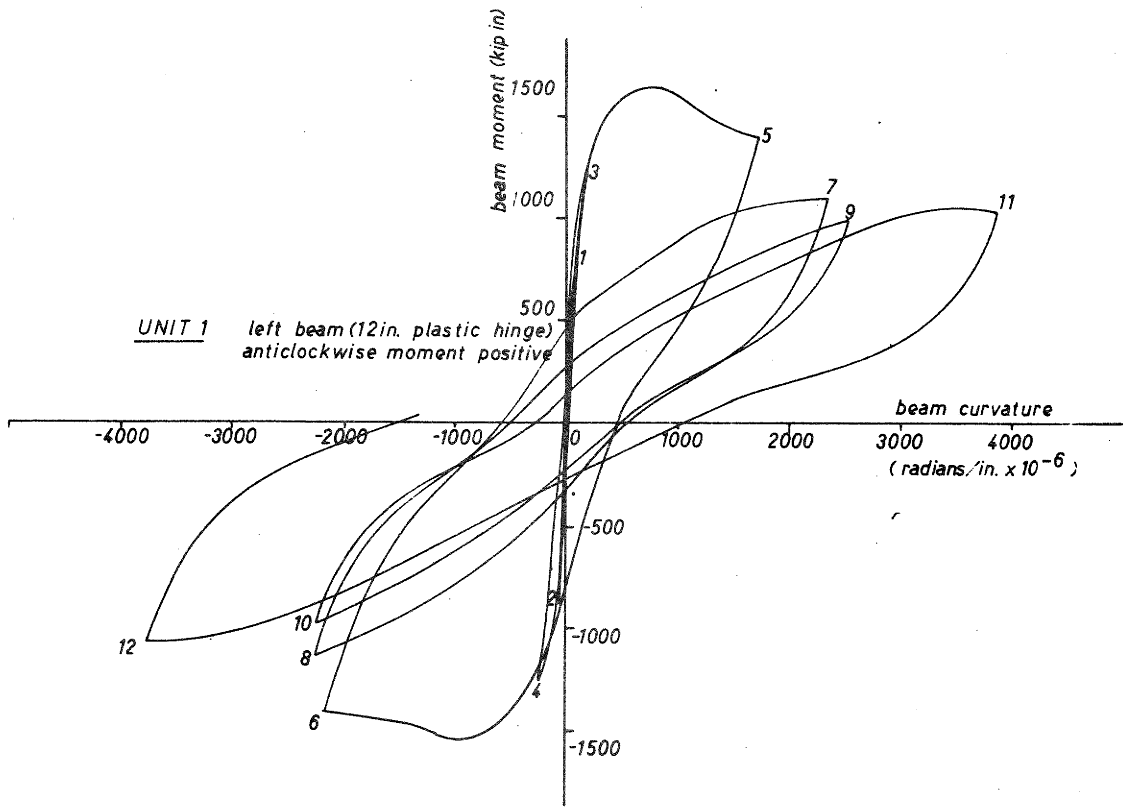


Fig. 9 Unit 1 Beam Moment at Column Face versus Average Curvature in Beam Over 12 in Gauge Length Adjacent to Column Face.

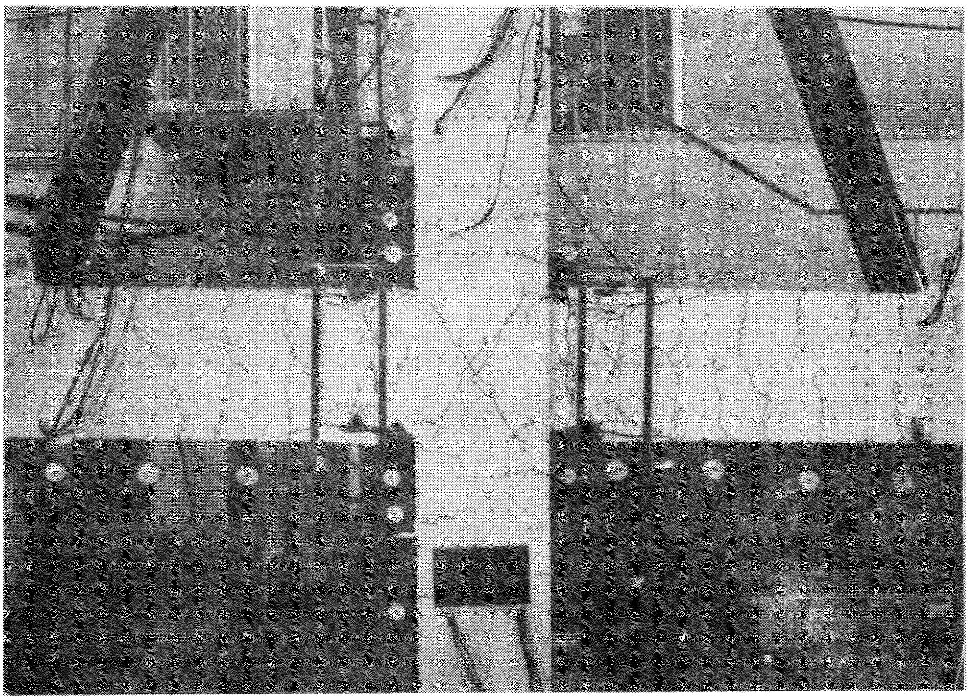


Fig. 10 Unit 1 At Maximum Moment (Cycle 6)

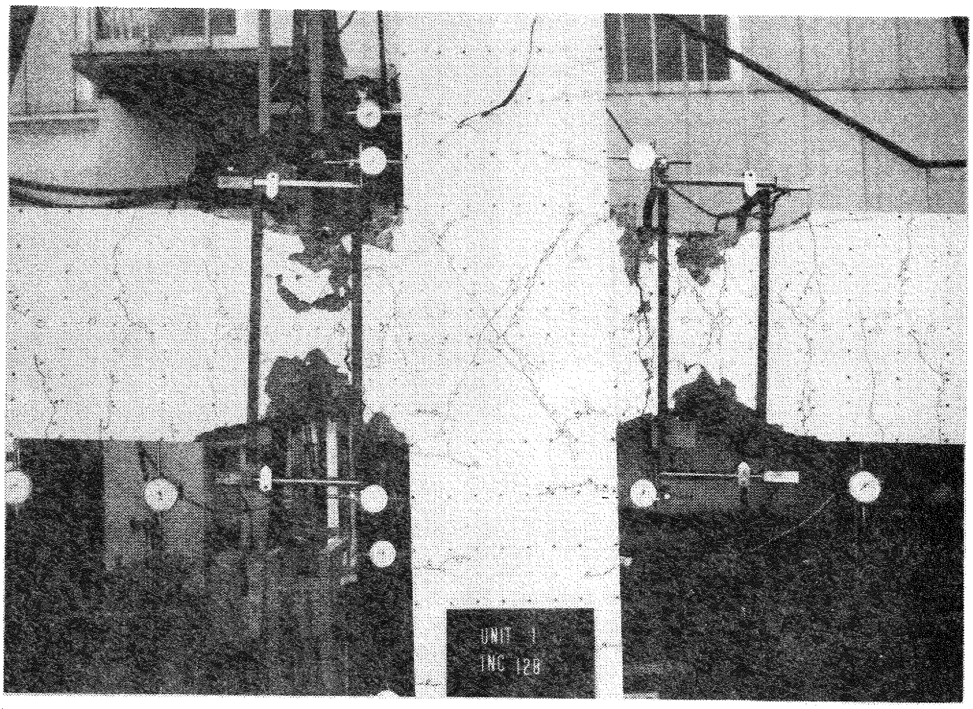
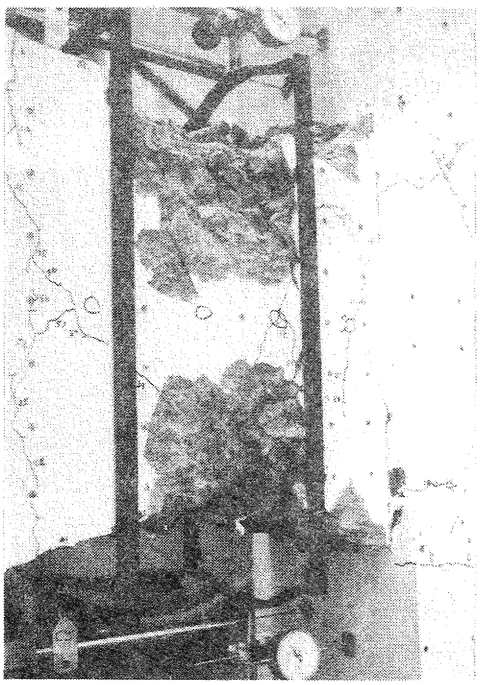
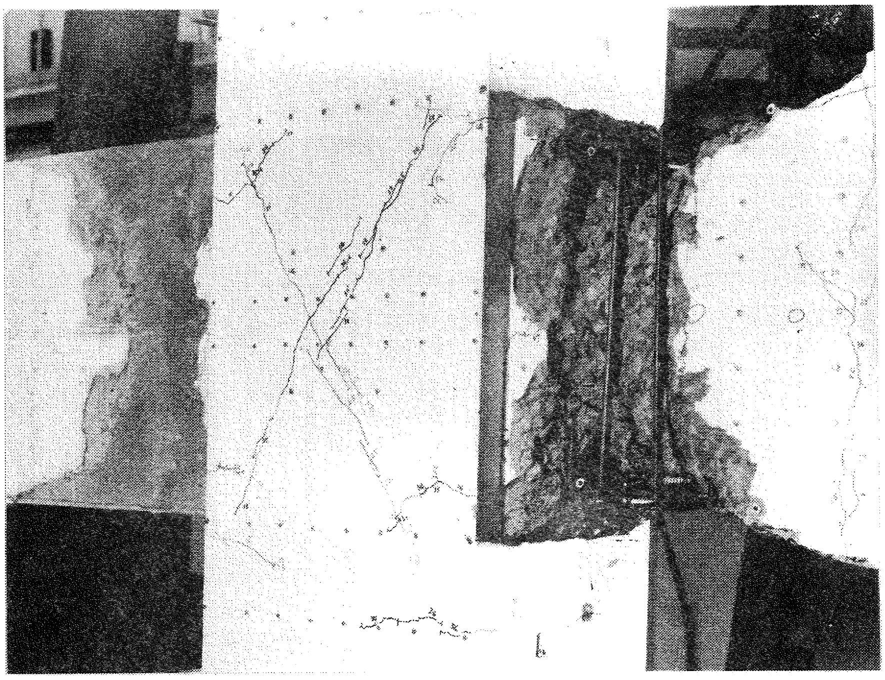


Fig. 11 Unit 1 At Maximum End Displacement (Cycle 12)



(a) A Beam Section after Testing Prior to Removing Remaining Damaged Concrete.



(b) New Concrete Placed at One Beam Section, Other Beam Section About to be Boxed Ready for Placing Concrete.

Fig. 12 Repair of Unit 1.

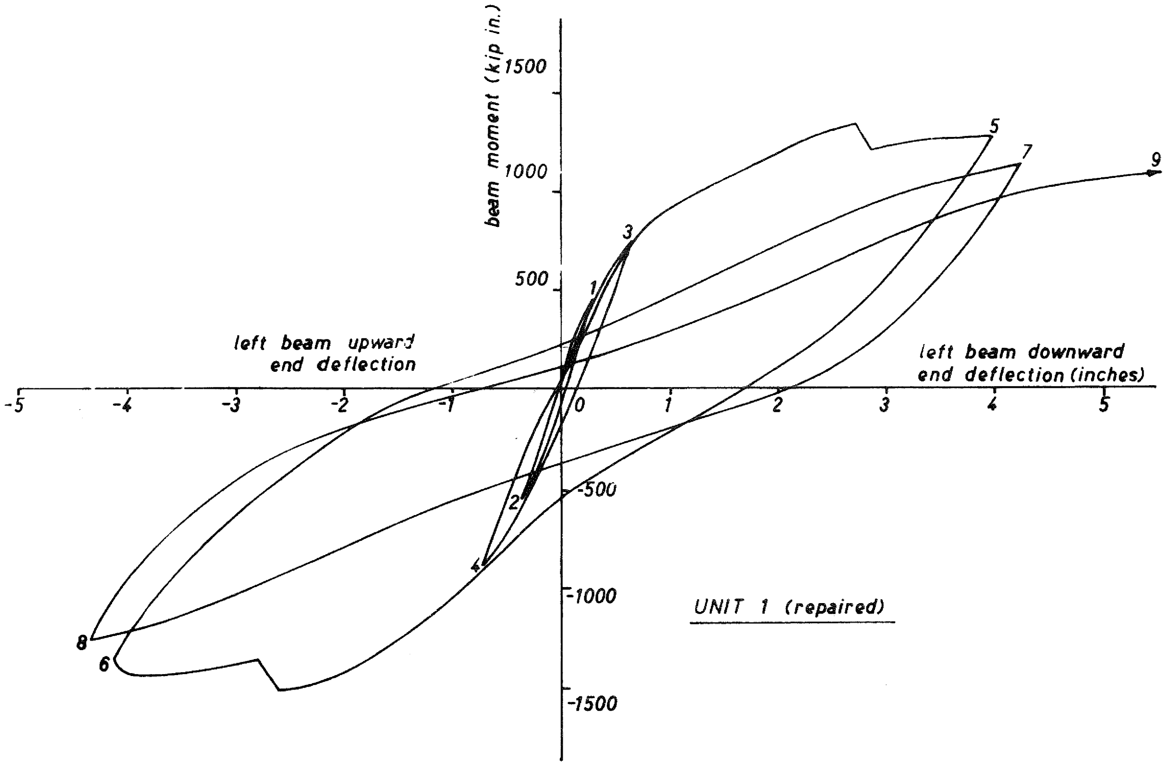


Fig. 13 Unit 1 (Repaired) Beam Moment at Column Face versus Vertical End Deflection of Beam.

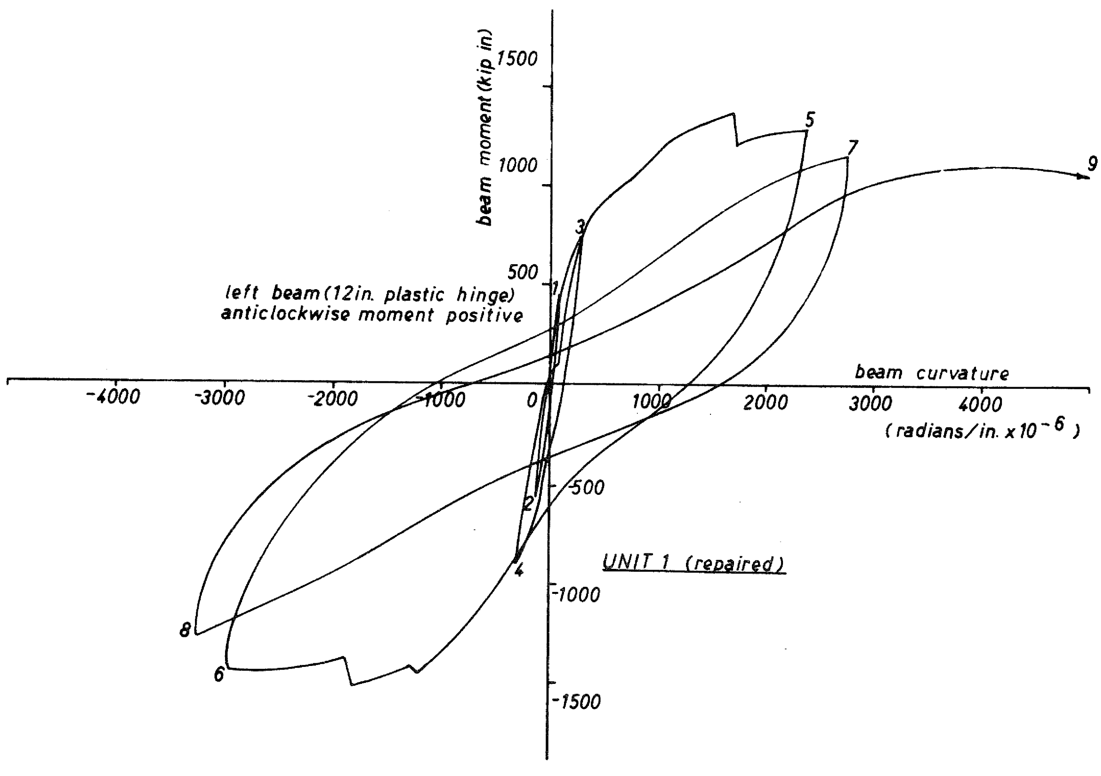


Fig. 14 Unit 1 (Repaired) Beam Moment at Column Face versus Average Curvature in Beam Over 12 in Gauge Length Adjacent to Column Face.

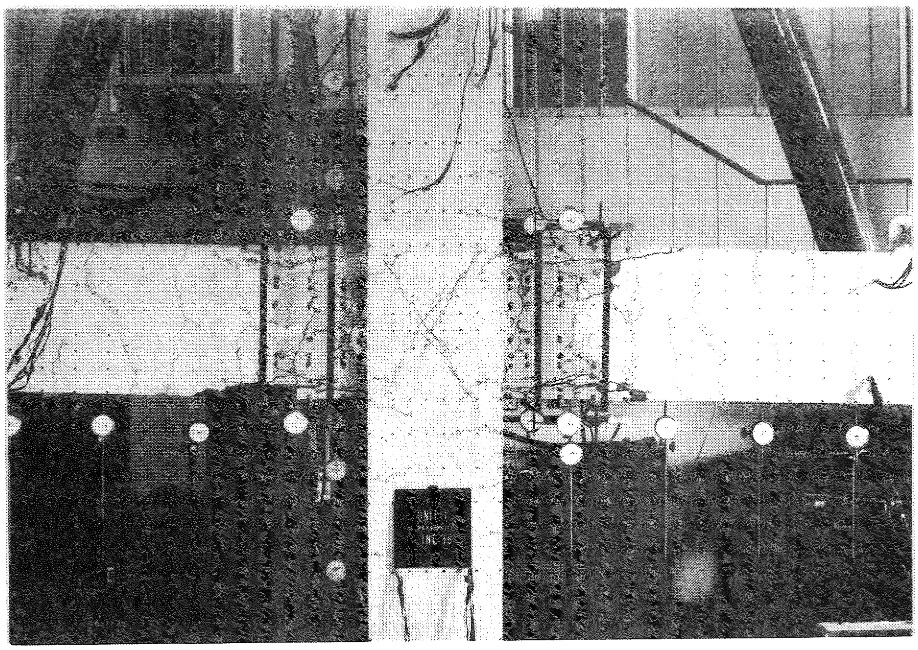


Fig. 15 Unit 1 (Repaired) At Maximum Moment (Cycle 6)

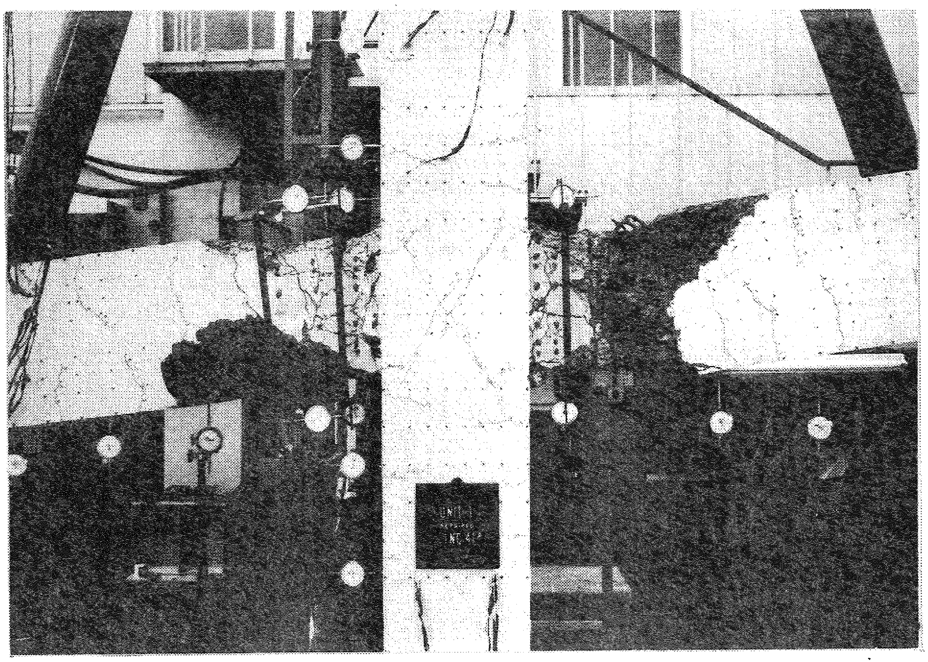


Fig. 16 Unit 1 (Repaired) At Maximum End Displacement (Cycle 9).

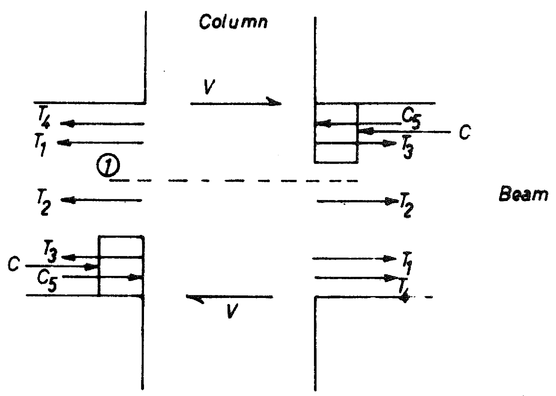


Fig. 17 Beam Forces Acting on Joint.