THE COUPLING OF REINFORCED CONCRETE SHEAR WALLS

by Thomas Paulay [I]

Synopsis

The behaviour of short and relatively deep reinforced concrete coupling beams, common in shear wall structures, subjected to simulated earthquake loading is reported. The investigation was partly stimulated by the spectacular failures in two multistorey buildings in Alaska. Only the more interesting and unusual features of the experimental evidence, related to the performance of the flexural and shear reinforcement, failure mechanisms, deformation and stiffness characteristics of coupling beams are presented in this brief report.

1. Introduction

The seismic resistance of numerous multistorey buildings is concentrated in reinforced concrete shear walls. Many of these walls contain one or more rows of openings. In such cases it is customary to speak of shear walls which are coupled by beams, situated above and below the openings. A particularly common example of such a structure is the "shear core" of tall framed buildings which accomodates elevator shafts, stair wells and service ducts. Access doors to these shafts separate solid walls from each other which may or may not be interconnected by short and often deep coupling beams at each floor. Frequently the major, if not the whole, lateral load on the building has to be carried by such a core.

Numerous theoretical studies have been published on the elastic analysis of such structures. However little is known about their actual behaviour, particularly under conditions imposed by earthquakes. Certain codes place limitations upon the heigth of shear wall structures and also require the presence of rigid jointed flexible frames, which are expected to be capable of resisting a certain fraction of the prescribed lateral load. An amount of distrust in shear walls, particularly with respect to ductility, is implied in such code requirements. Factual evidence however is lacking. It is for this reason that a project was initiated at the University of Canterbury in which the behaviour of coupled shear walls, particularly in the cracked state, and their failure mechanisms are being studied.

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The major components of such structures are the walls proper and the coupling beams. In the first stage of the project the strength and behaviour of the coupling beams was examined. The more interesting features of this investigation are presented in this paper. The study was partly stimulated by the spectacular failure of such beams in the Mt.McKinley and the 1200 L buildings in Anchorage, both of which were heavily damaged during the March, 1964 Alaska Earthquake [1]. The behaviour of coupled shear walls, which are subjected to flexure, shear and axial tension, is currently being investigated.

2. The Coupling Beams

Two coupled shear walls, the stiffnesses of which are generally considerably greater than those of the connecting beams, impose approximately the same rotations at each end of a coupling beam , where the same frames into the walls. Because of their small span relatively large shearing forces are generated. The depth to span ratio of coupling beams is often in the vicinity of one, Therefore it can be expected that the behaviour of such beams will not obey the laws which are applicable to slender reinforced concrete flexural members. Some of the questions which arise in connection with the performance of coupling beams of shear wall structures may be formulated as follows:

a) To what extent does the conventional flexural analysis, based on the Bernoulli-Navier hypothesis, used for reinforced concrete flexural members, assess the situation in coupling beams?

b) What are the principal modes of shear resistance in such deep beams?

c) Do the bending moments and the large shear force affect each other, i.e. does an interaction exist?

d) Do the elastic and postelastic deformation characteristics have any unusual features?

e) What is the stiffness of such beams upon which the elastic response could be based?

f) What are the principal modes of failure?

g) What are the effects of high intensity alternating loading, to be expected in a major earthquake, upon the strength and stiffness of coupling beams?

The testing program was planned in such a way as to yield some information related to the above questions. The coupling beams were cast together with two end-blocks, which simulated parts of the coupled shear walls. They enabled two equal moments to be simultaneously applied to both ends of the beams. Two such beams, with different aspect ratios, are shown together with the testing frame in Fig. 1 and Fig. 2. The load was applied by means of a centrally placed hydraulic jack. Through the two halves of the steel frame the load was transmitted to each end-block so that the point of zero moment remained always under the jack at the midspan of the beam. The crack pattern of a typical beam is shown for one-way loading in Fig. 3 and for alternating near ultimate loading in Fig. 4.

The beams contained an equal amount of mild steel flexural reinforcement in the top and the bottom of the beam. It consisted of # 7 or # 8 deformed bars which were anchored 3 feet into the end-blocks. Closed stirrups, made of # 3, # 4 and # 5 mild steel deformed bars, were placed at 4 in. centres. In some beams intermediate horizontal bars, of the same size as the stirrups, were used at 8 in. centres. The general arrangement of the reinforcement and the principal dimensions are shown in Fig. 5. Here the applied load is also indicated. The cylinder crushing strength of the concrete was in the range of 5000 to 6000 psi.

3. The Behaviour of the Flexural Reinforcement

By means of mechanical strain gauges the strain distribution in all flexural bars was determined over the length of the beam proper. From these the tensile force generated at each point at various stages of the loading could be derived. Typical tensile force distribution curves along the top reinforcement, for the beam illustrated in Fig. 3, are shown for four stages of the loading in Fig. 6. Because of the antisymmetrical load pattern similar curves were found for the bottom reinforcement. The figures in circles indicate the load intensity in terms of the failure load.

A comparison with the theoretical distribution of the tension force, shawn by the straight dotted lines, indicates that after diagonal cracking of the beam a radical change occurs in the pattern. When 35 % of the ultimate load was applied only a few flexural cracks were observed. At this stage a good agreement existed between the measured values of the tension and the values obtained from conventional analysis. When over 40 % of the ultimate load was applied diagonal cracks gradually spread over the whole beam. As a consequence of this a portion of the flexural reinforcement in the compression zone of the beam has also been subjected to tension. When the load intensity was in excess of 75 % of the ultimate load the top reinforcement was in tension over the entire span of the beam.

The curves in Fig. 6 supply important evidence, that was also verified on all other test beams, which indicates: that steel stresses in coupling beams of such relative dimensions can not be derived by conventional techniques; that plain sections across a beam do not remain plain at high loads, and that the beam does not behave as a doubly reinforced concrete beam. The steel is in tension in both faces of the beam at the same section. Therefore no increase in ductility can be expected from the presence of "compression steel".

Measurements also revealed that as the diagonal cracks developed, the moment arm of the internal forces was reduced. Near ultimate load the internal lever arm, z, has become as small as 60 % of the effective depth of the beam. This tendency is quite contrary to the observed behaviour of normal under reinforced concrete beams, in which the internal lever arm increases when yielding of the steel sets in. This phenomenon also explains the fact that not one of the test beams attained the ultimate load predicted by ultimate strength (such as Whitney's) theories and that yielding of the flexural reinforcement has occurred relatively early.

On another test beam the load was increased until yield commenced in all four bars of the flexural tension reinforcement at the supports. Then the load was reversed and it was increased until yield occurred in the bars situated in the opposite faces of the beam. Before the flexural compression force in the concrete could develop during this load reversal, the cracks, which were produced in the previous load cycle, had to close. However the yielded reinforcement which passed through these cracks in the compression zone delayed their closure and rendered the surrounding concrete less effective in carrying compression. The distribution of the concrete compression stresses and the relative positions of the internal forces are qualitatively shown in Fig. 7. It must be noted that the tensile resultant in these beams is the sum of the tensile forces generated in the top and bottom reinforcement. This diagram indicates that after the reversal of the near ultimate load the internal lever arm is further reduced. The load-strain relationship at the points of maximum tension is shown for the first and the reversed load in Fig. 8. The considerable increase of the steel strains at all stages during the application of the reversed load and the earlier onset of yielding is evident.

The maximum steel strains do not only increase at the support sections with cyclic loading. Increases have also been observed in the central portion of the beam, over which the flexural reinforcement always performed within the elastic range. Fig. 9 shows the distribution of tension force along the top reinforcement in the vicinity of the point of zero moment for three load applications in each direction and for three different load intensities.

4. The Behaviour of Stirrups

The web reinforcement was varied in the test beams so that a flexural or a shear failure could be expected. This also enabled a comparison to be made with the appropriate recommendations of the current Building Code of the American Concrete Institute (ACI). Some beams were deliberately underreinforced so that the web reinforcement was, according to the conventional truss analogy, capable of resisting only about 50 % of the shear force when the theoretical ultimate flexural load was to be attained.

As expected these beams, such as the one shown in Fig. 3, failed along the major diagonal crack across which all stirrups yielded. In spite of the insufficient strength of the web reinforcement a high load was carried by means of arch action. The geometry of the beams allowed this to occur. In all cases the flexural reinforcement commenced to yield prior to failure.

In another beam the web reinforcement was increased so that 74 % of the computed ultimate shear was to be carried by stirrups and, according to the ACI recommendations, which are generally considered conservative when applied to beams of such aspect ratios, about 19 % would have been resisted by the concrete section. This beam was subject to six cycles of alternating loading. Each time the load was gradually increased to 85 - 92 % of its theoretical ultimate intensity.

Finally beams were examined in which the strength of the web reinforcement was equal to 102 - 120 % of the estimated ultimate shear force on the beam.

Stress-strain relationships, obtained from a large number of measurements, indicated that during the first cycle of the loading a satisfactory agreement existed with the ACI recommendations. This is illustrated in Fig. 10. The heavy line in each of the three diagramsindicates the stress-strain relationship at the region where the major diagonal crack crosses the stirrup. However when cyclic loading was applied it became evident that the contribution of the concrete towards shear resistance gradually diminished and that more of the shear force was thrown on to the stirrups. Fig. 11 which shows the load (shear)-strain relationship for stirrups illustrates this observation.

In spite of the premature yielding of the web reinforcement high alternating loads could be carried by arch action in the coupling beams. Fig. 12 shows the load-strain relationship for the centre stirrup of a beam which was subjected to six cycles of near ultimate load. The curves indicate appreciable ductility in shear. It was found that these large shear deformations considerably affect the load-rotation characteristics of coupling beams.

High percentage of web reinforcement was provided in test beams when it was desired that the full flexural strength of the beam should be attained without yielding occurring in the stirrups. Indeed yielding was not observed even during cycling loading in any of the stirrups, which crossed the usually critical main diagonal crack. The failure in such beams was confined to a section at the support where the maximum moments occurred. At this section however one or two stirrups, which crossed a steep diagonal crack adjacent to the end of the beam, always yielded before the beam failed in diagonal compression and sliding shear.

5. The Load-Rotation Characteristics

The test specimens were instrumented so as to enable the rotations of the rigid end-blocks to be determined. From these the rotations at the supports of the beam proper could be derived. The rotations at both ends of the beams were evaluated independently and this indicated little deviation from antisymmetrical behaviour as long as the beams behaved elastically.

Fig. 13 shows the load-rotation relationship for a 31 in deep beam containing heavy web reinforcement and in Fig. 14 the behaviour of a similar beam is illustrated. In this latter beam the stirrups were theoretically capable of resisting only 74 % of the ultimate shear.

The onset of diagonal cracking during the first load cycle is clearly evident in both cases. The evaluation of several beams indicated that the stiffness of these coupling beams was reduced, after cracking, to about one quarter of the value which was based in a conventional way upon uncracked concrete sections. The theoretical load-rotation relationship, which allowed for flexural and shear deformations of the uncracked concrete beam, is shown by the straight dotted lines. They indicate a satisfactory agreement. Analytical studies and deflection measurements both indicated that shear deformations in cracked coupling beams greatly predominate flexural deformations.

Fig. 14 illuminates the behaviour of a beam in which large yielding occurred also in the web reinforcement. Consequently the diagonal cracks became wide and remained wide upon removal of the load. When the load was reversed these cracks, which now crossed the line of diagonal compression, had to close first before significant compression forces could be transmitted. Thus considerable rotations had to occur as the load was brought on to the beam. Only after these large initial rotations had taken place did the beam stiffen. This "soft range" of resistance at low loads is clearly manifested in Fig. 14. The phenomenon is likely to affect significantly the elastic response of a shear wall structure.

6. Beam Distortions

For the sake of fuller understanding of the behaviour, the distortions of these beams have also been examined. Only two features, which are entirely insignificant in normal reinforced concrete beams, are mentioned here.

Because of the large lengths of the stirrups, relative to the span of the beam, significant transverse expansions of the beam occur when the stirrups are highly stressed. The displacement of one horizontal edge of the beam relative to the other is shown in Fig. 15. for three cases. These transverse expansions are generally so large that they completely overshadow the deformed shape of the beam's axis. (Such as shown in the upper left hand corner of Fig. 14.)

It was previously stated that after extensive diagonal cracking the top and the bottom reinforcement is subjected to tension over the entire length of the beam proper. Consequently the whole beam besomes longer as the load increases. By measuring the displacements of the end-blocks relative to each other the beam elongation could be determined quantitatively. Fig. 16 shows curves for one-way and for cyclic loadings of beams in which only the web reinforcement was different. These beam elongation curves give a good picture of the yield deformations of the flexural reinforcement. It is likely that this type of beam distortion affects the behaviour of the shear walls which are being coupled.

7. The Principal Modes of Failure

As can be expected beams which are underreinforced for shear fail along a diagonal tension crack. Such coupling beams are separated into two triangular halves. Coupling beams of the two buildings in Anchorage, Alaska(1) and the test beams shown in Fig. 2 and Fig. 3 belong to this type. Clearly this type of failure is undesirable.

The large shearing force generated in a coupling beam is largely responsible for the failure, even if very generous web reinforcement is provided. This is particularly the case when simulated earthquake loading is applied. The compression zone of a coupling beam is seriously disturbed by the flexural cracks produced during a previous load cycle. Appreciable shear displacements occur along such cracks and the permanent shear displacement may be significant if the load responsible for this was of high intensity. These displacements contribute to the deterioration of the concrete at each face of a crack when the same closes. The compression zone near the supports also has to transfer a significant portion of the shear because of pronounced arch action at high loads. The disturbed and partly pulverised compression zone however is not capable of efficiently transferring the vertical component of the diagonal compression. Considerable sliding displacements may occur at this stage. A few stirrups and the flexural reinforcement which are crossed by this near vertical crack are unable to arrest the sliding movement. The stirrups yield, the flexural bars bend due to dowel action and the beam fails in sliding shear and diagonal crushing of the concrete. The latter sometimes manifests itself by lateral bursting. Fig. 17 shows a close-up of such a failure section.

This type of failure is restricted to a relatively narrow vertical band near the supports. In one beam the destroyed concrete was removed and the gap filled with new concrete while the beam was left in the testing frame. Upon reloading the beam, in the same sense as the previous failure load was applied, it failed at the unrepaired end. This time however the failure load was 22 % higher. The flexural reinforcement at both critical sections operated in the strain hardening range. The large yielding is also revieled by the extensive elongation of the beam as indicated in Fig. 16. The repaired beam after failure is shown in Fig. 18.

In one of the beams the four corners, which are subjected to large diagonal compression, were reinforced with closely spaced # 2 transverse reinforcement to enable the effect of confinement to be studied. In spite of the small confined width a considerable increase in ductility was obtained. After 9 cycles of high intensity of load reversals, the concrete in the compression zone crushed owing to steep diagonal compression, as can be seen in Fig. 19. The sliding shear displacement associated with this failure is illustrated in Fig. 20 by the dowel deformation of the flexural bars in the upper right hand corner of the beam.

8. Summary and Conclusions

The behaviour of short and relatively deep, symmetrically reinforced concrete coupling beams, which frequently occur in shear wall structures, was briefly described. The test specimens were subjected to cyclic static loading of near ultimate intensity. The more important conclusions of this experimental investigation may be summarised as follows.

The tensile stresses along the flexural reinforcement of coupling beams radically deviate from the values derived by conventional techniques of analyses.

After the development of diagonal cracking, which is always inevitable, the flexural reinforcement was found to be in tension over the entire span of the beam. Therefore the "compression steel" in the doubly reinforced coupling beams of this investigation can not be relied upon to improve ductility.

The lever arm of the internal forces is always smaller than in conventional reinforced concrete beams. This is more pronounced after the first repetition of high intensity loading.

Web reinforcement provided according to present practice seems inadequate when alternating load is expected because the contribution of the shear resisting mechanisms of the concrete diminishes.

Large shear forces produce large diagonal compression because of arch action, and this reduces the flexural capacity. The interaction of shear and flexure must be taken into account when assessing the strength of coupling beams.

There is a drastic loss of stiffness after the formation of diagonal cracks. Shear distortions predominate flexural deformations in the cracked state. The yielding of the reinforcement, particularly of stirrups, is responsible for a "softening" of the beams at small loads. This is likely to affect the elastic response of coupled shear wall structures.

Confining reinforcement placed in areas of critical diagonal compression increase the ductility of coupling beams.

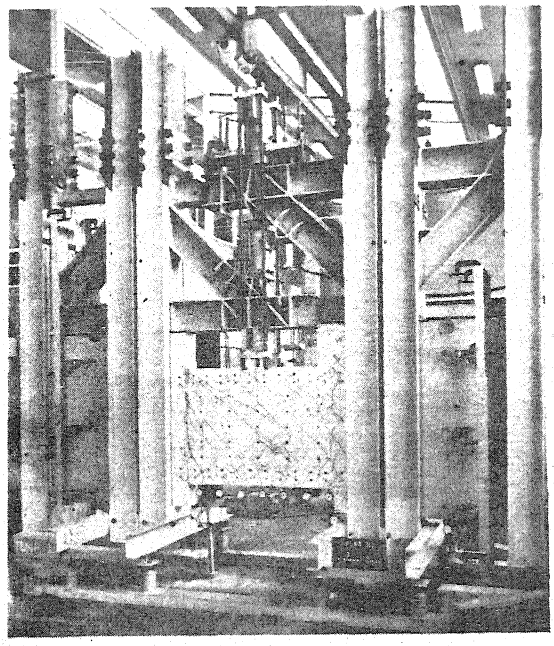
A full account of this research project together with analytical studies and design redommendations will be published later.

9. Acknowledgements

The support of Professor H.J. Hopkins, Head of the Department of Civil Engineering, and the financial assistance of the University of Canterbury are gratefully acknowledged.

10. References

1. Glen, G.V. and Stratta, J.L., "Anchorage and the Alaskan Earthquake". American Iron and Steel Institute, New York.



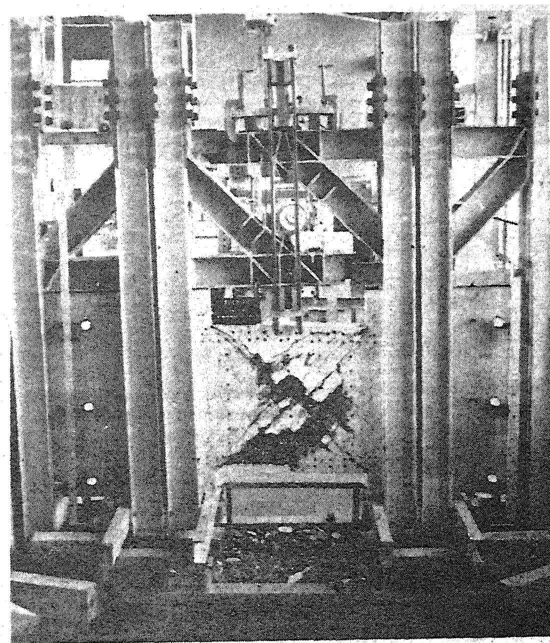
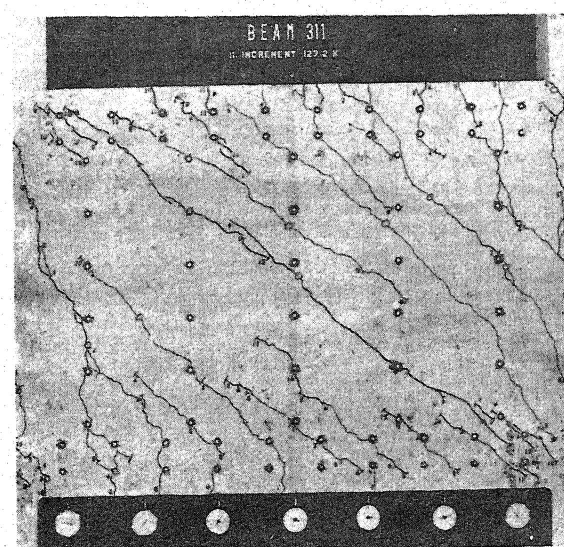


Fig. 1 Shear Wall Beam in Testing Frame. A Diagonal Compression Failure.

Fig. 2 Diagonal Tension Failure After Alternating Loading.



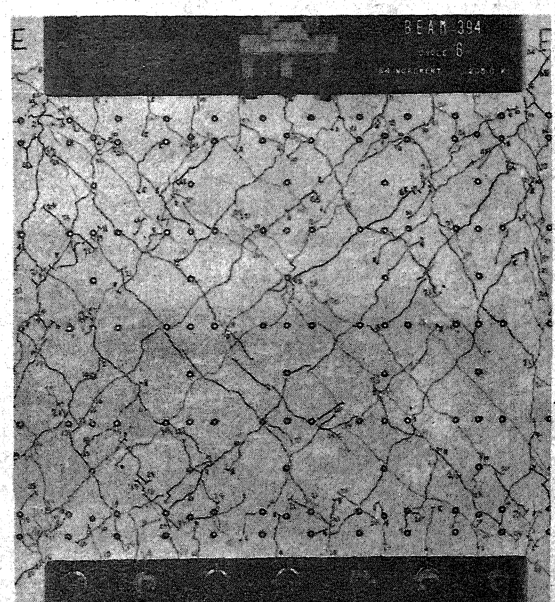


Fig. 3 Failure is Imminent Along the Main Diagonal. One-way Loading.

Fig. 4 Crack Pattern After Six Cycles of Near-Ultimate Loading.

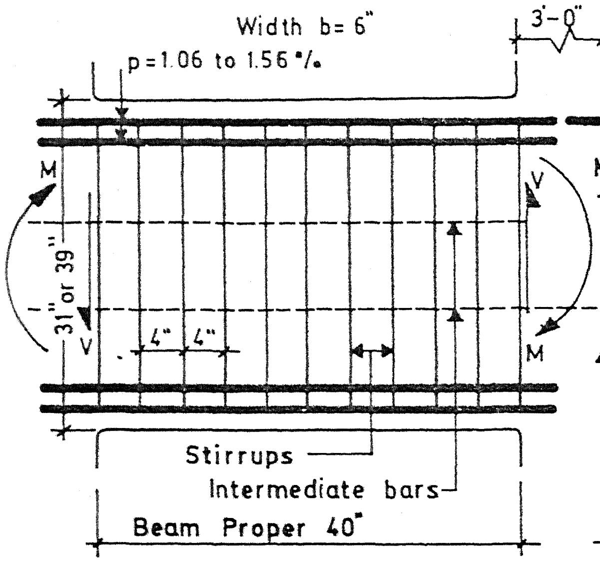


Fig. 5 The Dimensions, Reinforcement and Loading of Coupling Beams

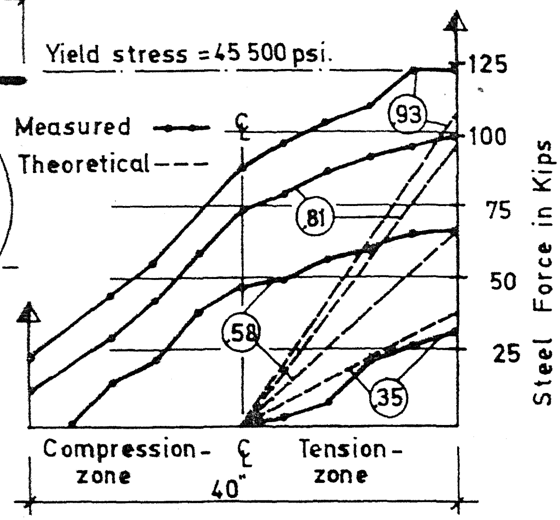


Fig. 6. The Distribution of Tension Force Along the Top Reinforcement

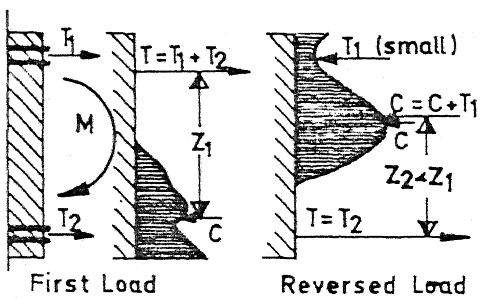


Fig. 7 The Position of the Internal Forces Before and After Load Reversal

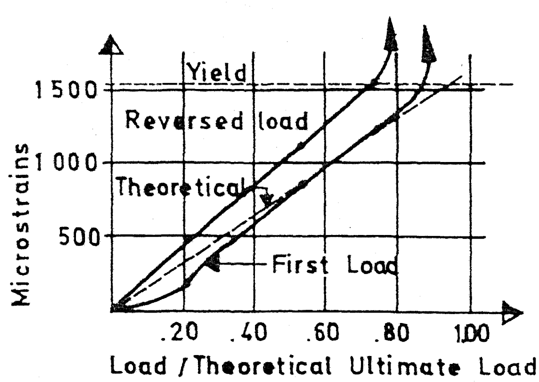


Fig. 8 Load-Steel Strain Relationship at the Supports During the First Two Load Cycles

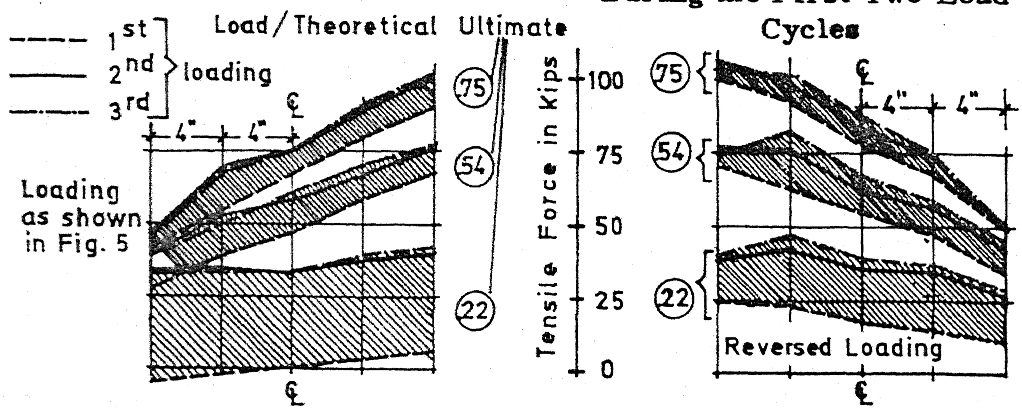


Fig. 9 The Change of Tension Force Distribution due to Cyclic Load

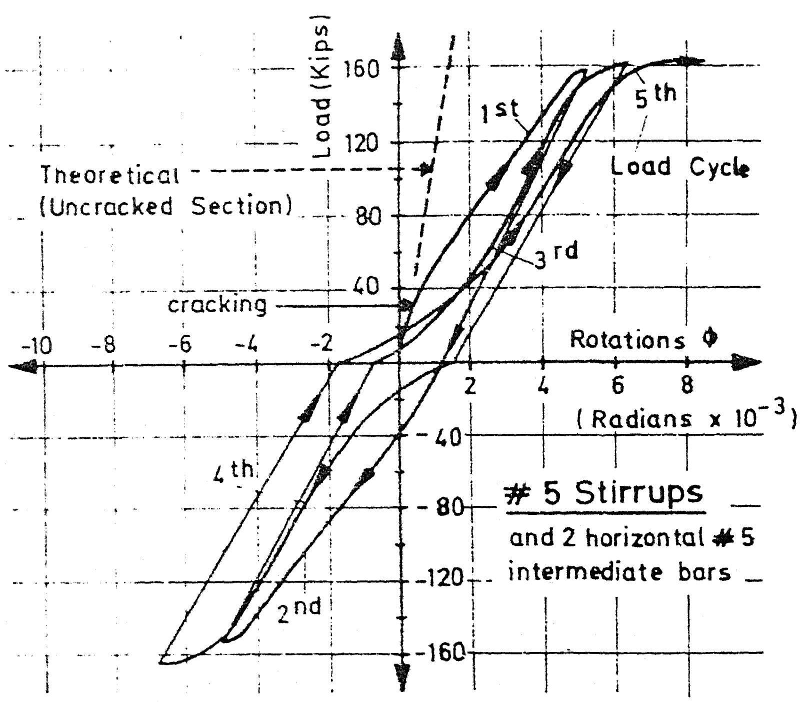


Fig. 13 The Load-Rotation Relationship for a Coupling Beam with Heavy Web Reinforcement

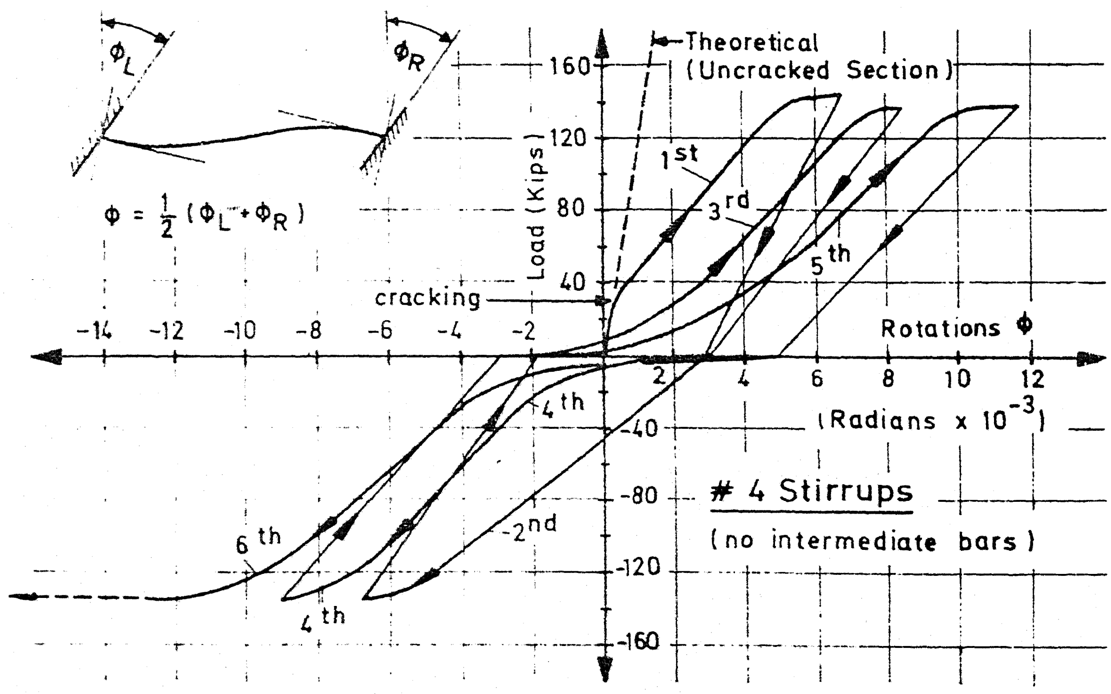


Fig. 14 The Load-Rotation Relationship for a Coupling Beam with Medium Web Reinforcement

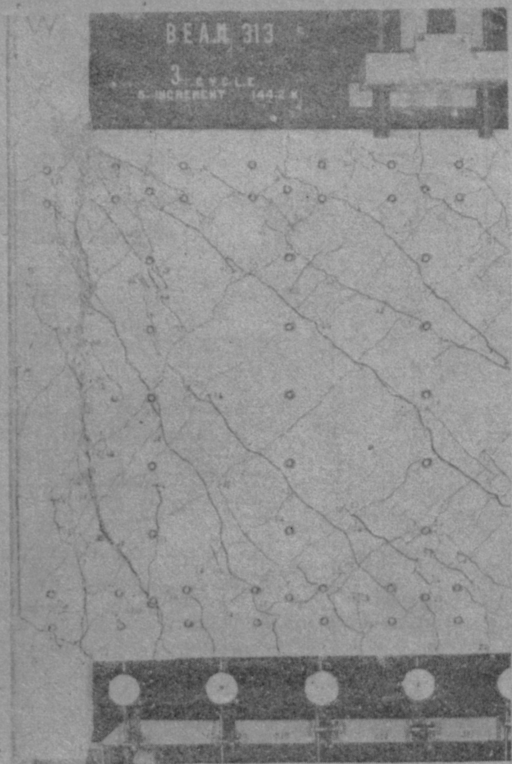


Fig. 17 A Failure Section. Diagonal Compression and Sliding Shear.

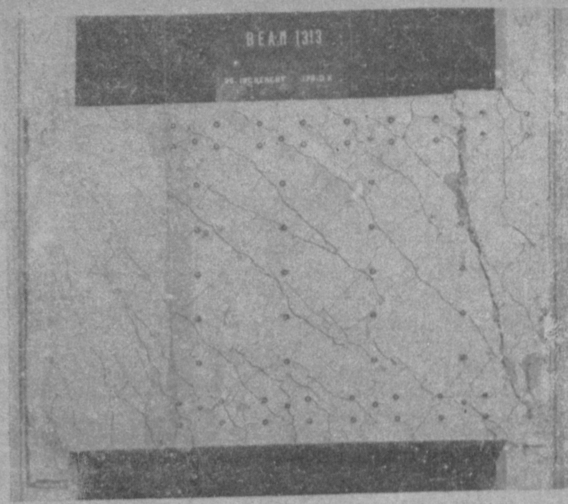


Fig. 18 Failure of Coupling Beam After Repair.

Fig. 19 Beam with Confining Reinforcement. Diagonal Compression Failure.

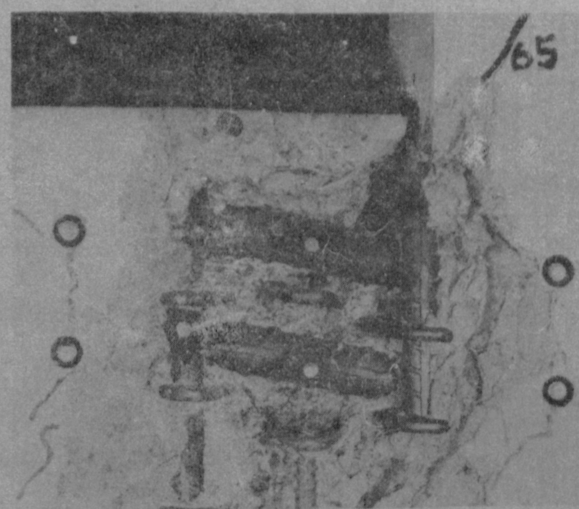


Fig. 20 Dowel Deformation of the Flexural Reinforcement due to Sliding Shear.