SP 42-26

Diagonally Reinforced Coupling Beams Of Shear Walls

By
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Synopsis : A study of conventially reinforced coupling beams, containing horizontal flexural reinforcement and vertical stirrups for shear resistance, revealed that failure after high intensity reversed cyclic loading is in the form of sliding shear. These beams were found to possess inadequate ductilities to satisfy the demand in ductile coupled shear wall structures which are expected to survive large earthquakes. In a search for improved performance, three further short and relatively deep coupling beams were tested under simulated seismic loading. In these the principal reinforcement was placed diagonally to enable the beam to act as a cross bracing with equal diagonal tension and compression capacity. The tests revealed that such beams, when adequately protected against instability, possess excellent ductility and energy absorption properties. They are likely to satisfy the ductility demands imposed upon them by adjacent shear walls under severe seismic load conditions.

Keywords: beams (supports); buckling; compression; cracking (fracturing); cyclic loads; deep beams; diagonal tension; ductility; dynamic loads; earthquake resistant structures; reinforced concrete; research; shear strength; shear tests; shear walls; stiffness.

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INTRODUCTION

The major part or all of the lateral load on multistorey buildings, resulting from wind or earthquakes, is often resisted by shear walls. These may have openings for windows or doors, generally in a regular pattern. A short and relatively deep structural element is usually formed between two such openings situated above each other. Because they provide structural connection between two adjacent walls on either side of a vertical row of openings, the elements are customarily referred to as connecting or coupling beams. The definition also conforms with the mathematical idealisation utilised in the "laminar analysis" of coupled shear walls. (1, 2) This analytical technique has been greatly developed in the past 10 years so as to deal with a variety of load and boundary conditions for linear elastic coupled shear wall structures, and it has been extended to assess behaviour in the elasto-plastic range of response.(3) The latter is particularly relevant to the performance of shear wall structures in seismic areas. The experimental study reported here was primarily directed towards seismic aspects of coupling beams.

Theoretical case studies indicated that, in the process of attaining full strength with adequate overall ductility in coupled shear walls, the ductility demand on the coupling beams could be very large. (3, 4) A comparison with conventionally reinforced coupling beam specimens (5), subjected to alternating cyclic loading at the University of Canterbury, showed that ductilities theoretically required could not be attained.(6) To appreciate this the main features of the behaviour and failure mechanism of conventionally reinforced coupling beams are briefly reviewed.

In a search for improving the ductility of coupling beams and in particular for suppressing the shear mode of failure, the project reported here was undertaken. Three beams have been generously instrumented and tested under reversed cyclic loading so as to impose large postelastic deformations, i.e. ductilities. The conventional flexural reinforcement, consisting of a group of horizontal bars in the top and the bottom of the beams, has been omitted. Instead, groups of diagonal bars were used which intersected at midspan (see Fig.4). The aim was not to supplement the strength of the conventional web reinforcement, consisting of vertical stirrups, but to provide reinforcement consistent with an entirely different internal resisting system. It was intended to simulate the behaviour of a cross bracing.

The stiffnesses of the shear walls are, as a rule, much larger than those of the coupling beams. For this reason, under lateral load the walls impose equal rotations at the built-in ends of beams. The symmetrically reinforced coupling beams are thus subjected to equal maximum moments at the supports, zero moment at midspan, in the presence of constant shear over the full clear span.

CONVENTIONALLY REINFORCED COUPLING BEAMS

To simulate the load conditions to be encountered in laterally loaded coupled shear walls, test beams were made with large end blocks. These enabled equal bending moments to be introduced in the presence of constant shear. The overall dimensions of two types of coupling beams, to be discussed in some detail, are reproduced in Fig.1. The forces shown were introduced by means of a massive steel frame, specially designed for this purpose, details of which have been previously reported.(5)

A typical conventionally reinforced coupling beam contained two No.7 and two No.8 (22 and 25 mm) continuous horizontal deformed bars in the top and bottom of the beam, adequately anchored in both end blocks. The shear reinforcement consisted of vertical No.5 (16 mm) stirrups placed at 4 in (10 cm) centres along the full span. This relatively large amount of web reinforcement was required to ensure that failure by diagonal tension, often observed in buildings damaged by earthquakes, would not occur even after several cycles of reversed cyclic loading imposing yielding on the flexural reinforcement. Nominal small diameter bars were also placed horizontally near the middle of the beam section. The steel arrangement over the clear span of such a coupling beam is shown in one of the photographs of Fig.3.

Theoretical considerations predicted and strain measurements verified that after the onset of diagonal cracking the flexural steel in beams, with a span to depth ratio of less than 1.5, is subjected to tension over the entire span of the beam. Tension exists not only at midspan, i.e. point of zero moment, but also in the theoretical compression zones for both the top and bottom flexural reinforcement. The ultimate strength of the critical section cannot, therefore, be assessed with the principles applicable to doubly reinforced concrete sections. The tension force in the bottom and top reinforcement must be balanced by an equal compression force resultant situated between the positions of the bottom and top steel. Thus the internal lever arm available to resist the external load is smaller than what would be encountered in a slender reinforced concrete beam. Indeed the observed ultimate moments in such beams were always less than what the currently accepted flexural theory predicts.

It is evident that as the load increases and as the top and bottom steel is in tension, the length of the beam increases with the load. Moreover as yielding occurs at the supports in the top and subsequently, after load reversals, in the bottom reinforcement, the vertical crack extending over the full depth of the member becomes large. Residual tensile yield in the flexural steel is never fully recovered but it accumulates as the high intensity reversed load cycles continue.

Irrespective of the amount of web reinforcement used, the whole of the shear force must be transferred by the concrete between the last stirrup and the built-in end of the beam, and by the dowelling action of the flexural reinforcement. The latter, however, can only be effective if vertical shear displacements, mobilising dowel action, can occur.(7) Prior to such displacement the concrete in the compression zone of the beam, heavily cracked during previous cyclic loading, is required to transfer the whole of the shear force. This involves the mechanism of interface shear by aggregate interlock action. (8)

When the load is removed, after a substantial excursion into the postelastic range, large cracks remain, due to residual tensile yielding of the reinforcement. Upon renewed load reversal this reinforcement will prevent the immediate closure of the cracks. Consequently coarse aggregate particles will not be able to establish contact across the crack, a prerequisite for shear transfer by aggregate interlock. Thus, as the load is being introduced, a sliding movement, engaging the flexural bars in dowel shear resistance, will occur. The sliding can be large enough to produce a gross lack of fit between the two faces of the previously formed crack. Consequently, upon closure of this crack, numerous stress concentrations will occur where protruding aggregate particles locally bear against the other displaced face of the concrete. Moreover the shear displacement, associated with the shear force to be transferred, will cause aggregate particles to be dislodged from the face into which they were embedded. As loading progresses the interface, instead of being a clean, well-defined, rough textured crack, becomes a zone of loose partly pulverised and crushed concrete with progressively reducing frictional resistance. With the breakdown of the aggregate interlock mechanism, a sliding shear failure terminates the capacity of the beam. The photograph of Fig.3 shows the fully developed pattern of cracks of such a beam and the zone of the sliding shear failure at the right-hand support at the 7th cycle of loading.

The response of the beam is best illustrated by the load-rotation relationship, presented in Fig.3. The drastic loss of stiffness, caused by diagonal cracking, is evident during the first four cycles of loading below yield level. The rotation is defined in Fig.2.

Other coupling beam specimens, web reinforced for the maximum possible shear load, with slightly smaller or larger span to depth ratios failed in the same manner by sliding shear, exhibiting only a small amount of ductility. (5, 6)

DIAGONALLY REINFORCED BEAMS

Beam Properties

The first two diagonally reinforced coupling beams (Beam 316 and 317*) had overall dimensions as shown in Fig.1.(a) and the third (Beam 395) was made to the dimensions of Fig.1(b). All beams were only 6 in (15.2 cm) wide and therefore it was difficult to accommodate symmetrically two sets of diagonal bars, intersecting at midspan, and to maintain adequate cover to these. To overcome this, four #7 (22mm) bars in two layers were placed in one direction and in between these three #8 (25 mm) bars were placed in a single layer in the centre of the 6 in (15.2 cm) width. The main bars together with nominal basketing reinforcement can be seen in Fig.4. No.2 (6 mm) ties at approximately 4 in (10 cm) centres have been placed around the diagonal groups of main bars. Short No.4 (12 mm) bars were required in the four corners of the ties around the No.8 bars which were in the middle of the beam's thickness. These are visible in Fig.4. No ties were provided in the first beam (316) tested.

The properties of the concrete and reinforcement used are assembled in Table 1. The test specimens lay flat on the ground when cast. The concrete, with a certified minimum strength of 3250 psi (228 kgf/cm^2), generally used for building construction in the Canterbury province of New Zealand, was supplied by a commercial plant.

Loading and Instrumentation

The loading was applied in approximately 10 increments in each cycle till the desired maximum intensity was attained and then it was reduced in 2 to 3 increments to zero. This is referred to as one load cycle. Hydraulic jacks were then rearranged so that the load could be applied with a reversed sense and another load cycle was applied. In the first four "elastic" cycles the applied load, P_i, did not exceed 80 % of the theoretical maximum load P_u^*. This was followed by one or two cycles of loading in each direction imposing very substantial yielding. To examine the effect of damage so inflicted, 2 to 3 "elastic" cycles were applied which were followed again by large imposed yield deformations till the beams showed a drastic strength reduction. Beam 316 maintained full strength till the deformations could not be followed any more with the test rig.

At selected load levels, particularly in the elastic range of behaviour, strain readings along all four No.7 diagonal bars were made. The steel studs, giving 4 in (10 cm) gauge lengths for the demountable mechanical strain gauges, are visible in Fig.4.

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* The first two digits refer to the beam's length in inches and the third digit is the number of the specimen in that series. Seven 31 in deep beams were tested.

The rotations and displacements of both end blocks were followed by numerous dial gauges. From these readings the rotations of the supports of the coupling beams, as shown in Fig.2, were derived. The ductilities were expressed in terms of the mean beam end rotations at first yielding. Simultaneously the horizontal movements of the end blocks (Fig.1) could also be evaluated, giving the elongation or shortening of the beams during loading.

The Behaviour of Test Beams

As expected, the steel stresses along the diagonal bars were nearly uniform after the onset of diagonal cracking. In the 31 in (78.7 cm) deep beams, the concrete still carried some tension while diagonal cracks developed. However, in the 39 in (99 cm) deep beams the diagonal steel was stressed rather high at low loads and this indicated that the corresponding diagonal compression force acted on an angle which was smaller than the angle of the tension steel. At maximum load level, the tension force distribution corresponded very closely in all three beams with that obtained from an equivalent (symmetrical) bracing system. The theoretical strength capacities, P_u^*, were based on this model. Due to strain hardening, all beams developed strengths in excess of the theoretical ultimate. Details of steel strains are fully reported elsewhere. (10)

The response of Beam 316 is shown in Fig.5. It shows the characteristics of a steel member with distinct Bauschinger effects and good energy absorption capacity with no loss of strength. It is to be noted that, even though a distinct reduction of stiffness after the first excursion into the postelastic range is evident, there is an immediate response in this beam to imposed deformations upon load reversals. This is in contrast to conventionally reinforced coupling beams which become very "soft" at low loads after previous substantial postelastic deformations (6), (see cycle 6 for Beam 315 in Fig.3). In such beams large cracks previously formed need to be closed first before the concrete can transmit diagonal compression. Only a small force is required to do this. The associated rotation may result in very large lateral deflections in a shear wall structure before the structure "hardens" again.

In diagonally reinforced beams the role of the concrete is relatively minor. Once the diagonal tension reinforcement has been subjected to substantial yielding it will resist the whole of the compression force upon load reversal. This steel will prevent the closure of previously formed diagonal cracks up till unrestricted yield level. Thus the beam's behaviour is governed by the diagonal steel bracing members which are expected to be equally effective in tension and compression. It may be noted that this system on its own satisfies the static requirements of shear and flexure.

The appearance of the beam at the termination of the test can be seen in Fig.5. The average end rotations of the beam at this stage were more than 12 times the yield rotation.

To ensure satisfactory performance of the steel cross-bracing, the bars must be adequately anchored in the shear walls (and blocks) and buckling of the compression bars must be prevented. The first requirement can be easily met. However, lateral support for the compression bars is more difficult to provide. At the final stages of the loading the concrete at the corners fell away (Fig.5) and the three # 8 bars in the centre of the wall showed a distinct buckle. For this reason in the other two beams ties, surrounding each set of diagonal bars, were provided at a nominal distance of 4 in (10 cm) centres, as can be seen in Fig.4. The purpose of these ties was to hold the concrete in position around and in between the diagonal bars and so to ensure some flexural rigidity with respect to out-of-plane (horizontal) displacements of a diagonal strut.

The response of Beam 317 is presented in Fig.6. It is seen that much more severe rotations were imposed on this specimen than on the previous one. The average rotations of the two ends, which after yielding may differ considerably, have been shown here. When these large rotations occurred it was difficult to maintain the position of the load. This introduced an unintended shift of the point of zero moment, away from the centre line of the specimen, and caused larger moments at one support of the beam. The maximum rotational ductility obtained in the 8th cycle was 21.1 . Towards the end of the test severe buckling, particularly of the No.8 bars, reduced the capacity of the beam. These No.8 bars were in compression when the odd-numbered load cycles were applied. The buckling and subsequent straightening of the reinforcement destroyed the concrete at the corners of the beams so that with the progression of cyclic loading less and less lateral support was available to the compression bars. Fig.7 shows the heavily damaged beam at two stages of the test.

The behaviour of Beam 395 is presented in Fig.8. The heavy bias towards positive loading, entirely unintentional, was due to the very large yielding imposed in the 5th cycle. Beyond the 8th cycle the effects of buckling of the compression bars are evident. The maximum ductility imposed on this beam was in excess of 12. The larger strength achieved in the specimen is due to the increased structural depth.

To check the performance of these beams under moderate loading, numerous crack width measurements were also made before the first yielding has occurred.(10) No unusual features were observed in this respect.

A COMPARISON OF TEST BEAMS

By comparing the performance of conventionally reinforced coupling beams reported here (Beam 315) and elsewhere (5, 6) with the performance of diagonally reinforced beams, the superior behaviour of the latter, with respect to seismic type of loading, becomes evident. As opposed to a sliding shear failure, diagonally reinforced coupling beams fail in a more ductile manner when the compression bars buckle. It is necessary to point out that the test set-up employed did not offer favourable conditions with respect to instability. The end blocks could not effectively restrain the beam against torsional rotations or against lateral displacement. In coupled shear wall multistorey structures, more favourable conditions would prevail because:

(1) wider beams would be used which enable the incorporation into the coupling beam of more effective diagonal tied compression members with larger flexural rigidity,

(2) the beam would be stiffened over its full span by a floor slab which would frame into it at least from one side,

(3) the coupled shear walls, stiffened at each floor by slabs, would provide a more effective support against lateral movement at the built-in ends of the coupling beams.

In both conventional and diagonally reinforced coupling beams, the principal reinforcement is in tension over the full span. Consequently beam elongation must occur. As Fig.9(a) shows, this is relatively small for a conventional beam, but it can assume very large proportions in the diagonally reinforced specimens, see Fig.9(b). Only a small proportion of the relative movements of the end blocks, shown in Fig.9, results from elastic deformations at, and slip of, the anchorages. The main source of elongation is the large yield strain, extending well into the strain hardening range, of the main reinforcement. When compression, i.e. instability, begins to limit the capacity of a beam, no further elongations occur. As the bars begin to buckle the beam reduces its length. This can be seen after the 8th cycle in Fig.9(b). It is not likely that such deformations could occur in shear wall structures without introducing substantial restraining forces from the slab diaphragms.

One current requirement for ductile earthquake resistant buildings in New Zealand is that the structure be capable of moving four times in each direction through lateral displacement at least four times as large as that which occurs at first yielding, with a strength loss not exceeding 20%. For individual components, such as beams, the ductility demand is usually in excess of this overall ductility factor of four.(3, 11, 12) However, the maximum ductility demand is not likely to be imposed on all such members simultaneously at any one instant of the elasto-plastic dynamic response to the seismic excitation. Thus a larger, 30%, loss of strength is considered acceptable after 8 cycles of loading, imposing ductilities consistent with the member's behaviour as part of the whole structure.

A convenient quantitative measure of a member's performance under such conditions is the cumulative ductility at the load level maintained. The horizontal axis of Fig.10 shows the accumulation of imposed ductilities during cyclic loading of the test specimens. The ordinate records the maintained load level in terms of the theoretical strength of the coupling beams. The superior performance of the diagonally reinforced coupling beams in maintaining a high strength with large ductilities is clearly demonstrated in this Figure. Similar results were obtained when the cumulative absorbed energy for all specimens were compared.(10)

CONCLUSIONS

1. Conventionally reinforced coupling beams, with a span to depth ratio of less than 1.5, will fail by sliding shear after a few reversed load cycles which will cause yielding of the flexural reinforcement. Conventional web reinforcement, consisting of closely spaced vertical stirrups, cannot prevent this type of failure.

2. The ductilities obtained experimentally for such coupling beams are considerably less than what theoretical studies of coupled shear walls indicate as desirable.

3. Diagonally reinforced coupling beams were capable of sustaining a much larger fraction of their theoretical strength capacity during cyclic test loading which imposed large cumulative ductilities.

4. The cause of failure in diagonally reinforced beams was buckling of the compression bars after the surrounding concrete had broken away. The conditions for instability of the compression bars were particularly unfavourable in the test specimens. Because of the transient nature of the load during seismic disturbances, buckling of compression bars does not result in complete loss of strength. Upon reversals, such bars straighten and contribute fully towards tensile strength.

5. To prolong the effective contribution of coupling beams during catastrophic earthquake, the confinement of the concrete within the cage of the diagonally placed group of bars is imperative. Only closely spaced ties or spiral winding, particularly at the four corners of the beam, are likely to contain the heavily cracked concrete during cyclic loading. The purpose of confinement is to preserve the integrity of a section and thereby to provide flexural rigidity, as in the case of tied columns, required to prevent lateral instability.

6. The omission of conventional flexural (horizontal) reinforcement in diagonally reinforced coupling beams enables a relatively easy assembly of the, preferably prefabricated, cage of the complete beam reinforcement and the placing of this into the coupled shear wall structure without undue congestion of bars where wall and beam intersect.

7. The suggested design procedure for diagonally reinforced coupling beams, which follows from first principles, is outlined in the Appendix and in Fig.11.

ACKNOWLEDGMENT

The financial assistance and the facilities provided by the Department of Civil Engineering of the University of Canterbury, Christchurch, New Zealand, where the experimental work was carried out, are gratefully acknowledged.

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APPENDIX

The analysis and hence design of diagonally reinforced coupling beams can be based on the statically determinate model illustrated in Fig.11. Initially the location and hence the magnitude of the compression force is not known. However, after first yielding and load reversal the diagonal reinforcement will resist all the forces. Accordingly at the development of yield strength

$$T_u = C_u = A_s f_y \text{ and } V_u = 2T_u \sin\alpha, \text{ hence } A_s = \frac{V_u}{2f_y \sin\alpha}$$

where $\tan\alpha = (h-2d')/\ell$

The resisting moment at the supports of the beam may be found, if desired, either from the shear force thus;

$$M_u = V_u \ell/2 = \ell T_u \sin\alpha$$

or from the horizontal components of the diagonal forces, i.e.

$$M_u = (h - 2d')\, T_u \cos\alpha$$

The theoretical ultimate capacities referred to in this paper are based on this simple concept.

It is suggested that the basketing reinforcement, shown with broken lines in Fig.11, consisting of small diameter horizontal bars and similar vertical closed stirrups, should give a steel content of at least 0.25% in each direction. The mesh size should not exceed 12 in. (30 cm) .

TABLE I - MATERIAL PROPERTIES

Beam No.	Nominal Bar Size		Yield Strength		Bar Area		Concrete Strength*	
	No.	mm	Ksi	kgf/cm^2	in^2	cm^2	psi	kgf/cm^2
315	7 8	22 25	44.7 43.0	3140 3030	0.590 0.740	3.81 4.77	5500	387
316	7 8	22 25	41.8 41.7	2940 2930	0.595 0.785	3.84 5.06	4825	339
317	7 8	22 25	44.4 39.2	3120 2760	0.580 0.760	3.74 4.90	7348	517
395	7 8	22 25	37.6 41.9	2640 2940	0.592 0.758	3.82 4.89	5150	362

* Cylinder crushing strength at time of testing.

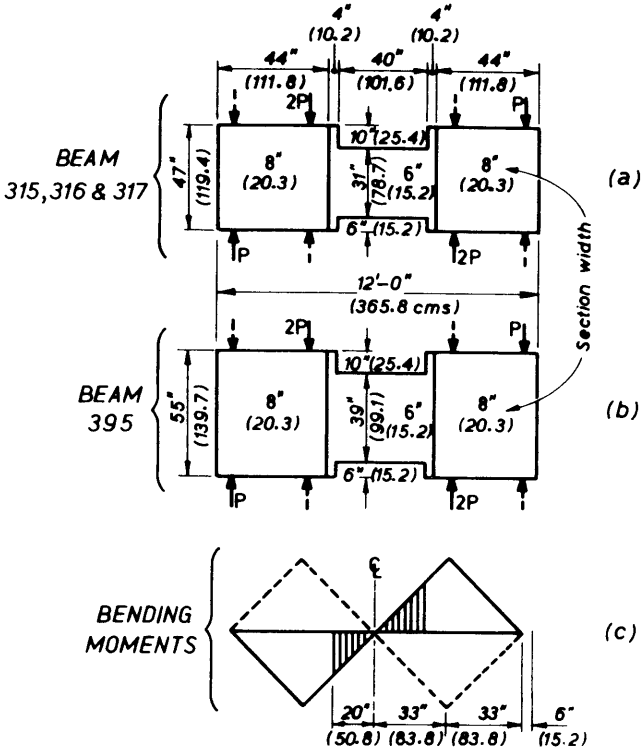


Fig. 1 - Principal dimensions and load pattern of test beams

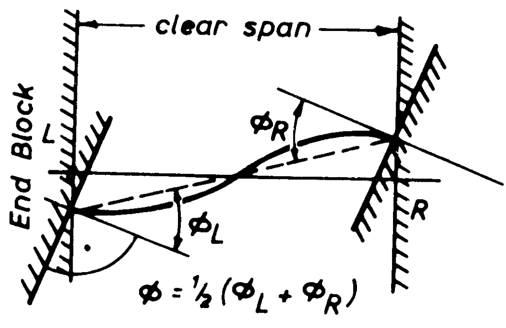


Fig. 2 - The mean support rotation of coupling beams

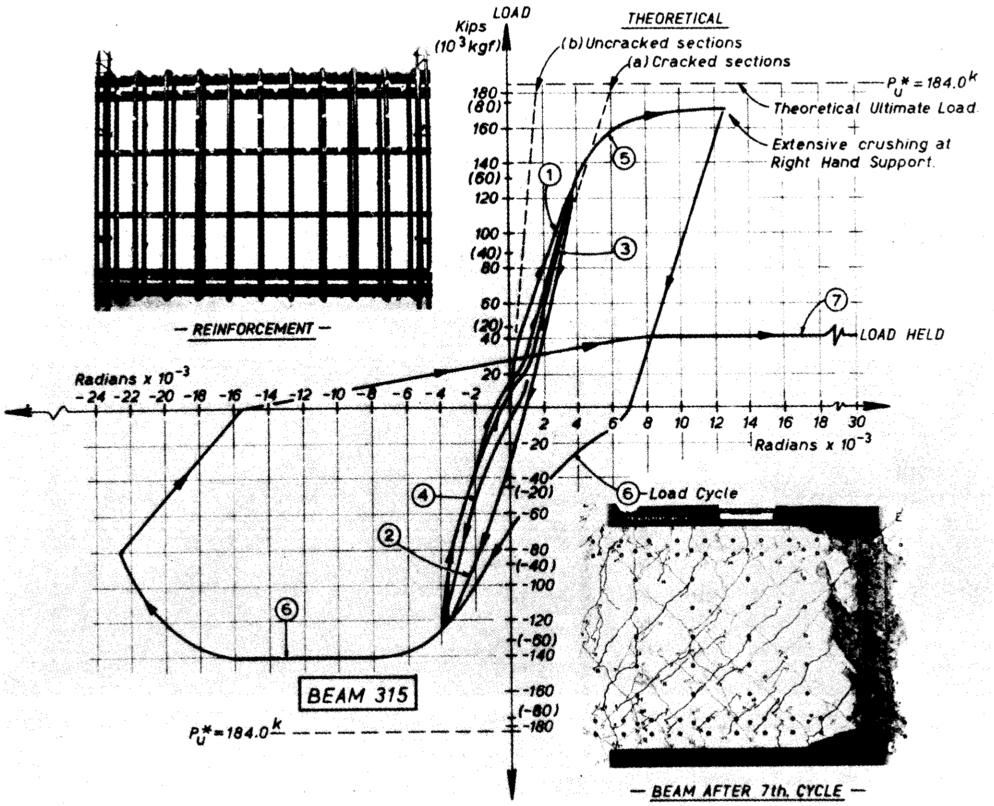
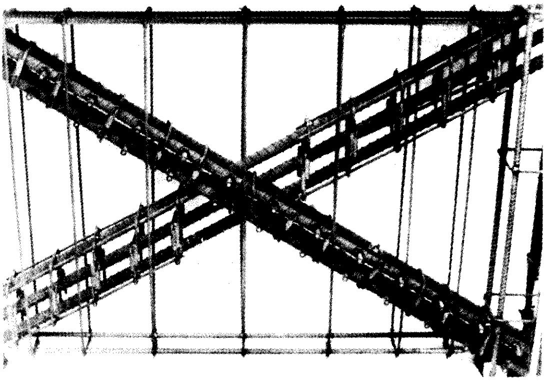
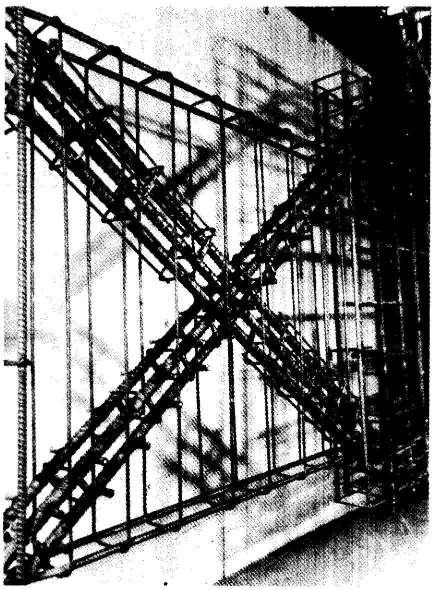


Fig. 3 - Load-rotation relationship for a conventional coupling beam



(a) Beam 317



(b) Beam 395

Fig. 4 - The reinforcing cages for diagonally reinforced coupling beams

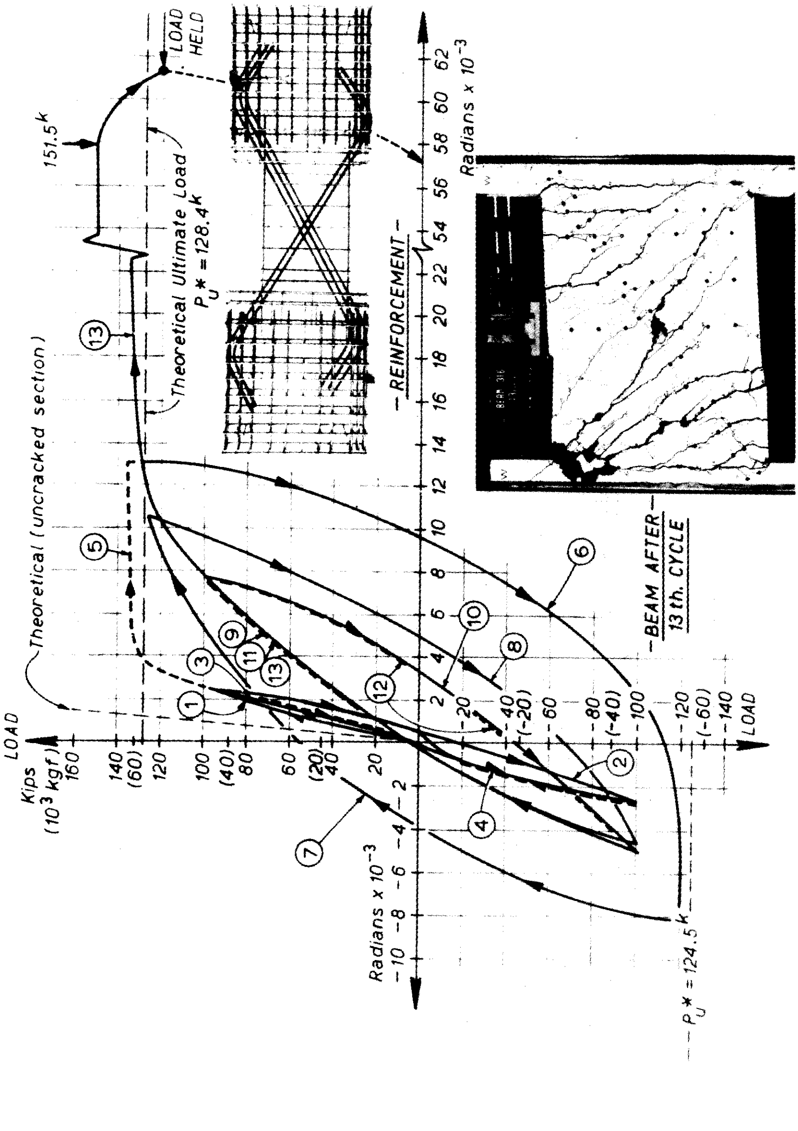


Fig. 5 - Load-rotation relationship for Beam 316

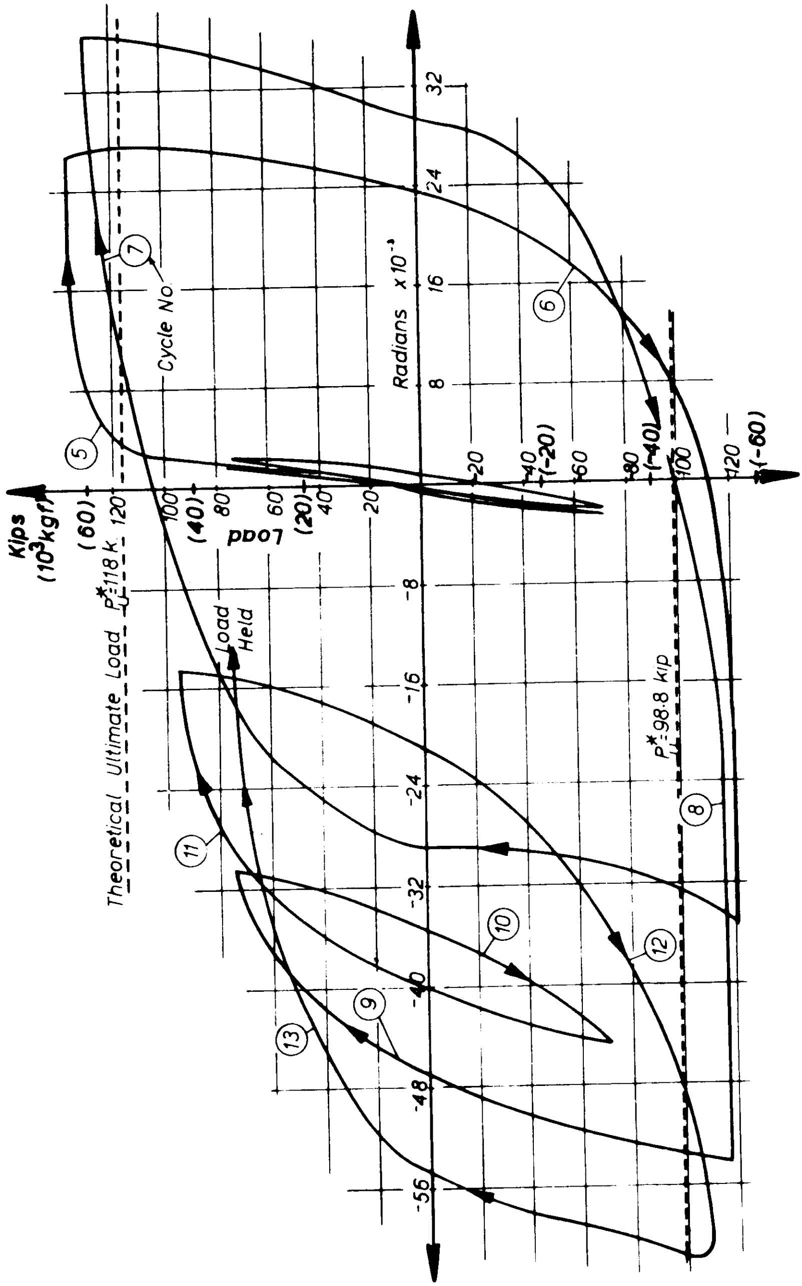
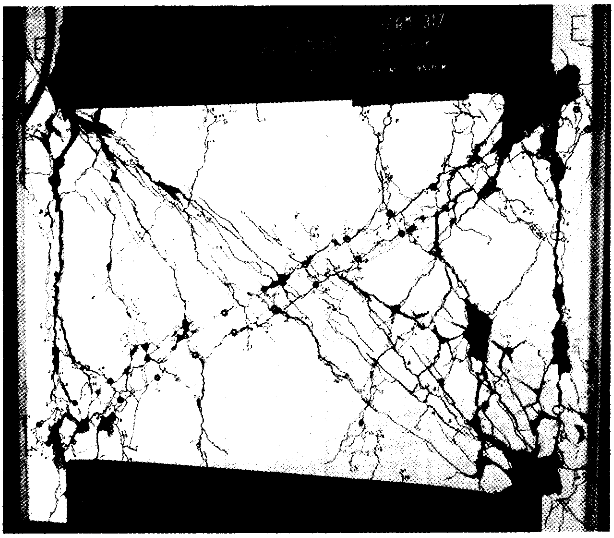


Fig. 6 - Load rotation relationship for Beam 317



(a) Beam 317

(b) Beam 395

Fig. 7 - Diagonally reinforced coupling beams at failure

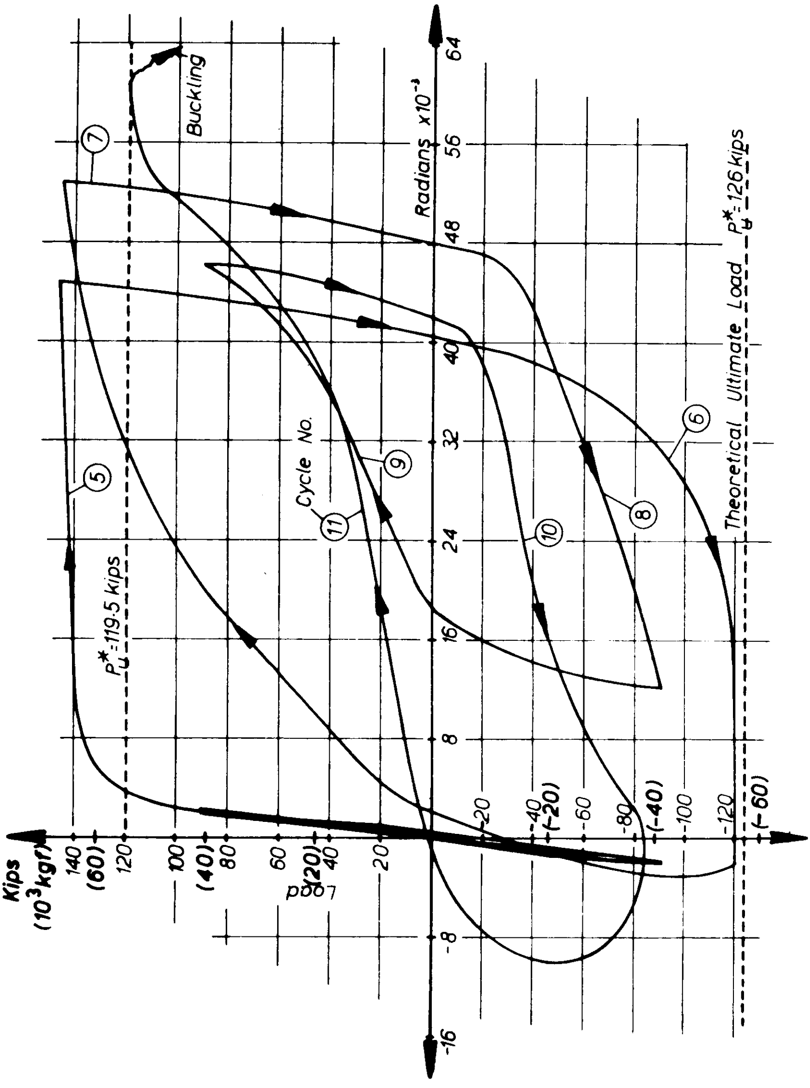
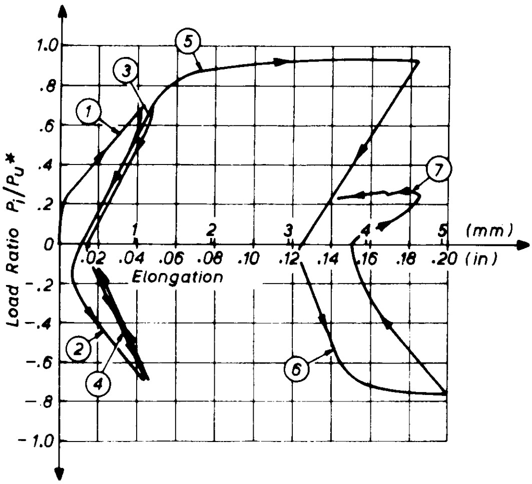
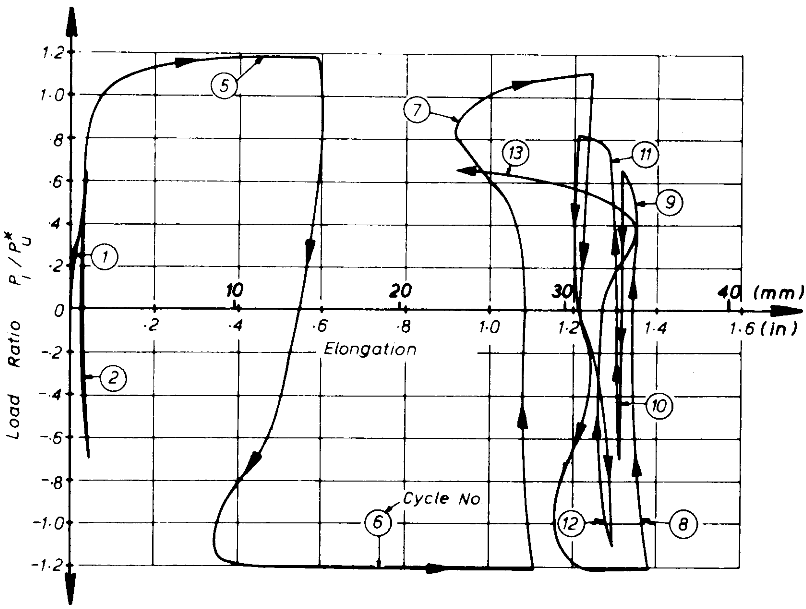


Fig. 8 - Load-rotation relationship for Beam 395



(a) Beam 315



(b) Beam 395

Fig. 9 - The elongation of coupling beams

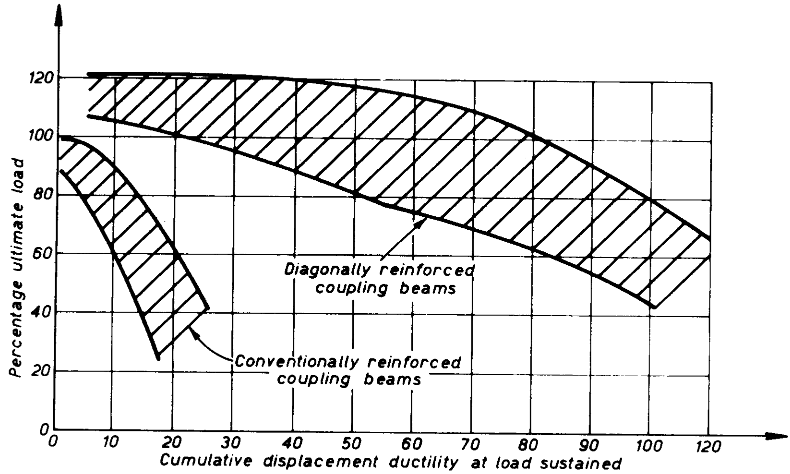


Fig.10 - Cumulative ductilities for conventionally and diagonally reinforced coupling beams

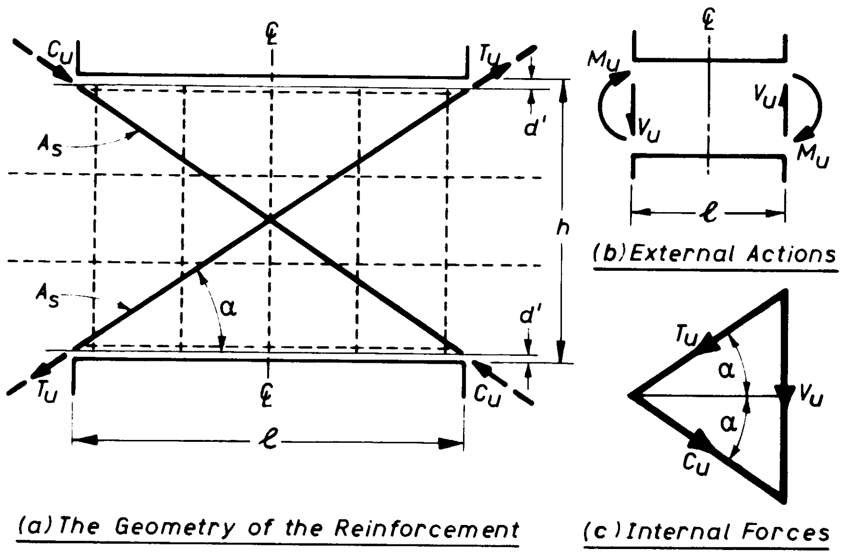


Fig.11 - Principal dimensions used in the design of coupling beams