DUCTILE BEHAVIOUR OF SHEAR WALLS SUBJECTED TO REVERSED CYCLIC LOADING

by

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SYNOPSIS

Possible failure modes in cantilever shear walls and their suitability as energy dissipating mechanisms are reviewed. From the analogy of a ductile shear fibre the potentials of coupled shear walls, as earthquake resistant structures, are discussed. The development of ductile coupling beams and their beneficial influence on the overall elasto-plastic response of two relatively large experimental reinforced concrete models are presented.

INTRODUCTION

Shear walls have been traditionally used in high rise buildings, largely to control deflections caused by lateral loads. The advantage that reinforced concrete shear walls present during moderate seismic disturbances, by reducing drastically the non-structural damage that could occur in rigid jointed frames, is now well appreciated. The satisfying of the requirements of ductility and energy dissipating capacity, to be utilised during very large ground excitations, has been open to much criticism. The notion that shear walls are likely to fail in shear or in a mode with restricted ductility, possibly with a consequent loss of gravity load carrying capacity, is still prevalent. During the last few years, however, much progress has been made in identifying the possible sources of brittleness and in devising techniques that would ensure adequate ductility in shear walls.

A cantilever shear wall, when subjected to gravity loads and to earthquake induced lateral forces, such as illustrated in Fig. 1a, must be designed in such a way that energy dissipation in the postelastic range of response will occur in flexure at a plastic hinge. This is shown in Fig. 1b. The associated shearing forces may induce diagonal cracking and may lead to a shear failure, shown in Fig. 1c, if the horizontal shear reinforcement is inadequate. Clearly, because of limited ductility and possible strength reduction with reversed loading, this failure mechanism is unacceptable. Interconnected flexural cracks or cracks at construction joints may lead to the mechanism of sliding shear depicted in Fig. 1d if the clamping force, N_f , is insufficient. Because of its poor energy dissipating characteristics [1] this potential failure mechanism must also be suppressed. After several large alternating excursions into the plastic range of response it may not be possible to prevent a sliding failure across the damaged concrete of the plastic hinge zone, shown in *Fig. 1e, unless some device is provided that can effectively transmit* shear stresses without much reliance on the shear strength of the concrete.

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Cantilever walls, when designed and detailed to dissipate energy in flexure can satisfactorily meet all the requirements of earthquake resistant design. It is to be noted, however, that there is only one line of defence when overload occurs. Yielding will be confined to a single plastic hinge, usually at the base, and thus a moderate excursion beyond the elastic limit may result in misalignment of the wall.

In a homogeneous isotropic beam, such as a thin webbed rolled steel section, one may have to consider also longitudinal shear stresses. If one were to decide that this critical shear fibre, illustrated in Fig. 2a, be made sufficiently weak, one could ensure that failure by shear would precede the development of the flexural capacity of the beam. If in addition one could assure that this shear fibre is sufficiently ductile and that it can sustain its full shear capacity over the entire height of the structure, shown in Fig. 2b, one could rely on an efficient energy dissipating device. The analogy to a coupled shear wall, such as given in Fig. 2c, is thus obvious.

COUPLED SHEAR WALLS

By utilising the discrete beams that connect coupled shear walls at each floor one could transform the previously discussed cantilever shear wall into a superior earthquake resistant system that would have advantages such as:

(1) Energy can be dissipated, when required, over the full height of the coupling system in addition to the potential plastic hinges at the base of the walls. (See Fig. 2c.) Energy dissipation is thus dispersed over the entire structure. During a catastrophic earthquake the major part of the energy could be dissipated by the coupling system. As a consequence the ductility demand on the walls is reduced. Sufficient ductility in the coupling beams, to be discussed subsequently, is a prerequisite to this desirable behaviour.

(2) Common shear wall structures are not unduly sensitive to the stiffness of the coupling system. As Fig. 2c indicates, the total overturning moment, M_o , at the base of the structure is resisted by wall moments, M_1 and M_2 , and an internal couple, Tl , where T is the axial force in the wall that results from the accumulation of the shear forces across the entire coupling system. Fig. 3 shows a comparison of those three internal moment components in a 20 storey example coupled shear wall structure [2]. It is seen that, when rectangular coupling beams are used, in this case with a depth to clear span ratio of $D/l_s > 0.33$, at least 80% of the overturning moment, M_o , is resisted by the axial forces, T , operating with the available internal lever arm, l . Thus a deliberate increase in stiffness of the coupling system beyond $D/l_s = 0.33$ will not improve the efficiency of the structure.

(3) When coupling beams of sufficient stiffness are chosen the reduction of stiffness in the coupled shear wall structure, in comparison with a cantilever wall with the same overall dimensions and without openings, is usually insignificant.

(4) With an intelligent choice of stiffness and strength properties it is possible to optimally reinforce the various components in such a way that under increasing load the strength of all or at least most of the coupling beams is developed before the onset of yielding in the coupled walls. This implies that considerable energy could be dissipated in the coupling system before any serious damage would occur to the walls.

(5) With the preferred sequence and spread of plastification the

more serious damage may be confined to the coupling beams, the contribution of which to gravity load carrying capacity is usually negligible. Moreover, the beams lend themselves more easily to repair [3]. The walls, which must carry their appropriate share of the gravity load also after a catastrophic shaking, are thus protected to a much higher degree than in uncoupled cantilever systems.

DUCTILE COUPLING BEAMS

While deep coupling beams provide the necessary stiffness and strength, they are usually not ductile enough to fulfil the prerequisites set out in the previous section. In these conventionally reinforced beams with continuous horizontal top and bottom flexural bars, large shear forces may be generated at full flexural capacity that inhibit the development of the required ductilities. Vertical stirrups can be provided to control the consequent diagonal tension but they will not prevent a sliding shear failure after relatively few cycles of reversed loading [3].

If, instead of the above steel arrangement, two sets of diagonal bars are provided, as shown in Fig. 4, the necessary shear forces can be resolved into the two components. This statically admissible load transfer relieves the concrete from having to resist bond and shearing stresses. Moreover, after one excursion in each direction well past yield, the concrete does not need to resist compression forces either. Extremely large ductilities were achieved in such beams [4] provided that sufficient ties were used in the diagonal cages to prevent instability of the compression bars that must sustain large plastic compression strains. Fig. 4 shows a typical beam used in construction. No difficulty was encountered with the placing of such beams.

COUPLED SHEAR WALL MODELS

To verify the performance of the previously discussed two types of coupling beams, two one quarter full size seven storey reinforced concrete shear walls were constructed and tested under simulated seismic loading. This consisted of static forces, (Fig. 5), equivalent to that specified by the seismic provisions of most building codes, applied several times in both directions with gradually increasing imposed displacement ductilities. Gravity loads were also suitably simulated. Several hundred locations were instrumented to allow the observation of steel strains in the beams and the walls, crack widths, rotations, member elongations and overall deflections at each floor to be made. Some of the results were reported recently [1] [5]. Only the most important findings are briefly stated here.

Both shear walls exhibited considerable ductility. Due to strain hardening of the reinforcement the load exceeded in both cases the theoretical predicted capacity. However, because of excessive shear deformations in the conventionally reinforced coupling beams, stiffness degradation and loss of energy dissipating capacity were considerable in the first specimen [5]. Eventually all coupling beams failed in that model, shown in Fig. 5, by sliding shear.

The performance of the other shear wall model, with diagonally reinforced coupling beams, was in every respect superior to that of the previous specimen. Because of the more effective control of shear deformations in these beams, particularly after excursions into the plastic range, increased overall stiffness was attained after the full development of cracking in all components. Fig. 6 shows the load-roof deflection relationship in terms of the theoretical ultimate load P_u^*. Yielding in the extreme fibre of one wall was observed after 5 of the 7 beams had yielded. Complete yielding of the walls occurred only after all beams had extensively yielded. It is seen that this reinforced concrete structure was extremely stable both in terms of stiffness and strength. The displacement ductility factors relevant to the last two load cycles were approximately 6.7/0.5 = 13.4 and 9/0.5 $\doteq$ 18.0. (The yield deflection, based on line II at $P/P_u^* = 1.0$, was taken as 0.5 in). The hysteretic characteristics obtained are similar to those of a steel member.

SUMMARY

When non-ductile mechanisms in cantilever shear walls are successfully suppressed good energy dissipating properties can be expected. Cantilever shear walls, however, have a single line of defence when ductility is required.

In coupled shear walls suitably reinforced coupling beams can be made to dissipate a substantial part of the energy input during severe excitation. With intelligent selection of stiffness and strength properties, energy dissipation can be efficiently dispersed over the entire structure. Experimental evidence verified this.

Shear walls with diagonally reinforced coupling beams, in the design of which the desired hierarchy of the real member capacities has been carefully established, are considered in New Zealand to offer the highest degree of seismic protection for multistorey reinforced concrete buildings.

ACKNOWLEDGEMENTS

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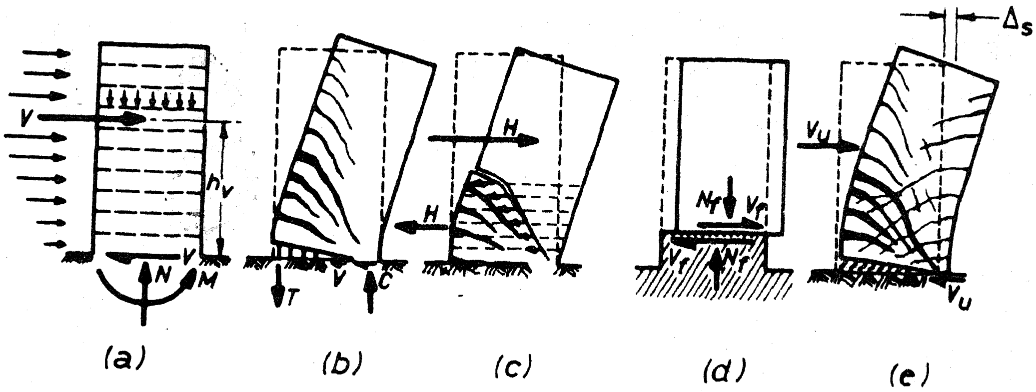
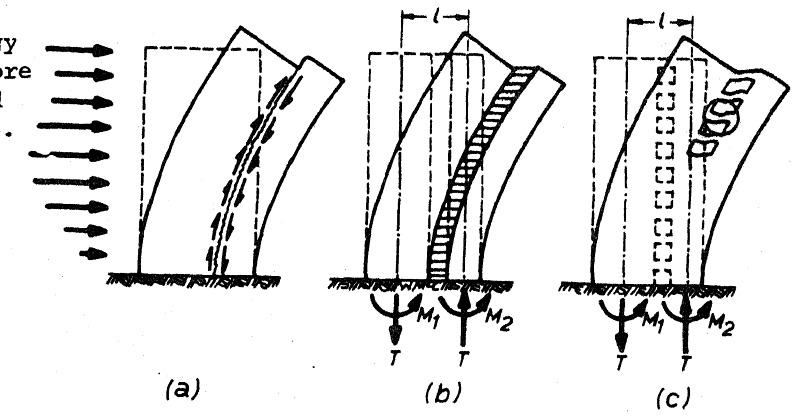


Fig. 1 Failure Modes in a Laterally Loaded Reinforced Concrete Shear Wall.

Fig. 2 The Analogy between a Shear Fibre of a Cantilever and Coupled Shear Walls.



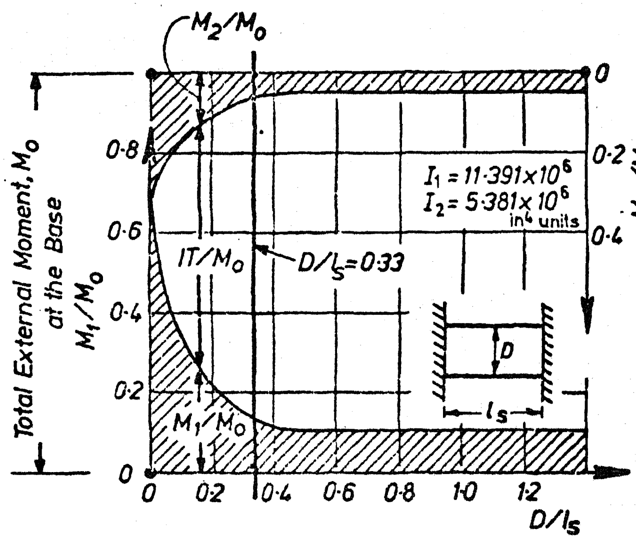
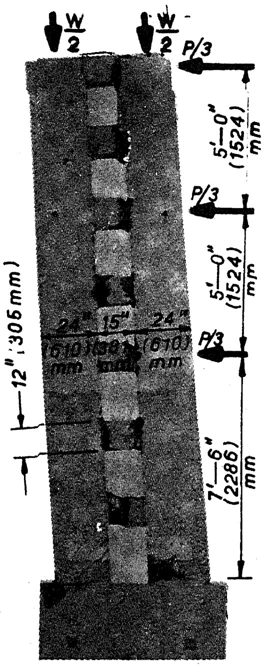


Fig. 3 The Variation of the Components of the Total Moment of Resistance at the Base of a Coupled Shear Wall Structure. ($M_o = M_1 + M_2 + lT$).



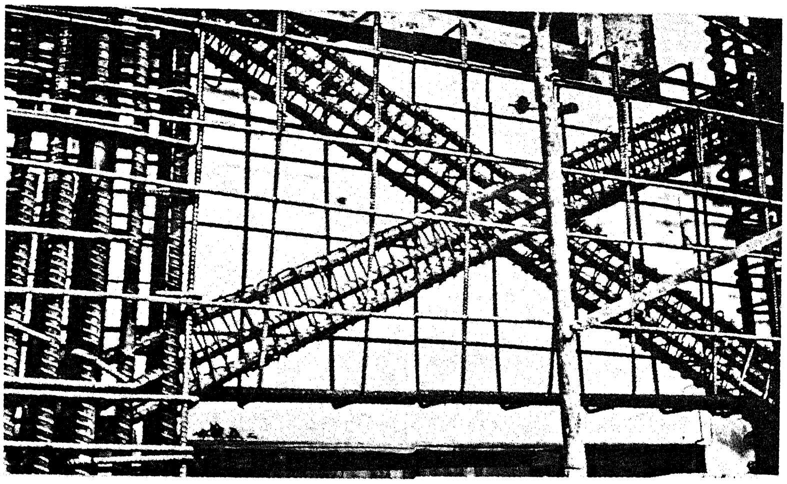


Fig. 4 Diagonal Arrangement of the Principal Reinforcement in Coupling Beams. (Note the closely spaced ties around diagonal bars and the nominal basketing reinforcement in the remainder of the beam.)

Fig. 5 One Quarter Full Size Shear Wall Model with Conventionally Reinforced Coupling Beams which Failed in Sliding Shear.

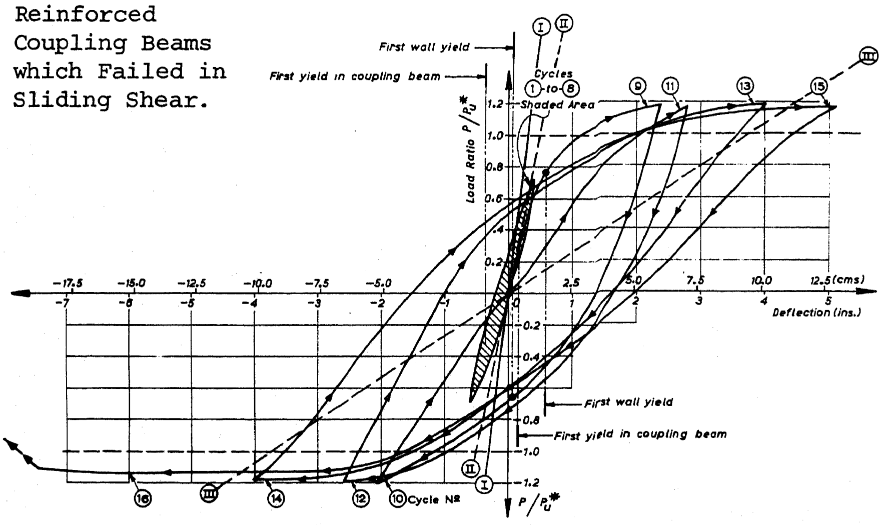


Fig. 6 The Load-Roof Deflection Relationship for a Coupled Shear Wall Model with Diagonally Reinforced Coupling Beams.
(P/P^*_u = Load to theoretical ultimate load ratio. Lines I, II and III represent the theoretical stiffness in the uncracked, fully cracked state and that of only two uncoupled cantilevers respectively.)

DISCUSSION

G.P. Saha (India)

The failure of the horizontal beams at the junction of the vertical walls shown in Fig. 5 has been termed by the authors as the failure by sliding shear and they have recommended the provision of diagonal reinforcements as shown in Fig. 4.

In an reinforced concrete member shear failure takes place in an inclined plane and not in a plane perpendicular to the longitudinal axis of the member. The type of failure which can be seen from the photograph of Fig. 5 indicates more a bending failure than a shear failure, the fracture mode being vertical in nature. It may be mentioned here that due to horizontal load on the shear wall the maximum bending moment will occur at the support of the beams and the fracture modes as are seen in Fig. 5 can be a likely mode of bending failure.

Stirrups with adequate horizontal bars are regarded by some engineers as a better form of shear reinforcement compared with the inclined bars mainly because the cracks can be of lesser width and evenly distributed. The stirups are usually distributed along the length of the beam and around a section whereas the inclined bars are concentrated within the section and hence from the point of distribution of cracks, stirrups are recommended.

It might be argued that the support section is subjected to bending moment and shear simultaneously and due to the bending moment vertical cracks will propagate into the section and lesser number of stirrups will be operative in resisting the shear. But it is also a fact that due to the bending moment the high compressive force developed in the compression zone of the beams enhances the shear resistance of the section substantially.

Hence the writer would like to know if the authors have done any investigation to compare the capacity of a R.C.C. section of the beam of a coupled shear wall by providing reinforcements (a) in the form of stirrups only and (b) in the form of inclined bars and stirrups so that the total quantum of reinforcements in the beam including stirrups, horizontal bars and inclined bars (for case b) remains same. For case (b) some minimum stirrups are to be provided for temperature and shrinkage consideration (as the authors have also provided in Fig. 4) which are also taken in the calculation to resist shear force. If it is found that by providing inclined reinforcement bars the total quantum of reinforcements are required less than by providing stirrups only to resist the same shear force, then this will certainly augment our knowledge on this subject.

Author's Closure

With regard to the question of Mr. Saha, we wish to state that at first sight it may appear from Fig. 5 that the shattered concrete near the supports of the coupling beams, where moments are maxima, was a consequence of a flexural failure. However, closer examination of these coupling beams, as shown in Fig. 7, shows enormous shear displacements and these leave no doubt about the nature of the failure.

Stirrups are indeed efficient in preventing diagonal tension failures even under severe reversed cyclic loading (1). It is evident, however, that apart from enabling the longitudinal bars to carry dowel shear, vertical stirrups cannot contribute to shear resistance along a full depth vertical crack across a beam. Such cracks develop at and near the support section because of the reversed cyclic nature of the loading. While large residual yielding occurs in both the top and bottom flexural reinforcement, such full depth cracks tend to widen as cyclic loading progresses. Eventually the only viable shear transfer mechanism, aggregate interlock action, breaks down as a result of grinding, and a sliding failure occurs. Because of the presence of equal top and bottom flexural reinforcement the moment is gradually being resisted by an internal steel couple. Therefore the concrete compression forces in the flexural compression zone, to which reference is made in the discussion, do not exist or they are insignificant.

Previous to the study reported here, 13 approximately three quarter full size isolated coupling beams, reinforced with stirrups only, as suggested in the discussion, have been tested. In all specimens, where adequate stirrups were provided to prevent diagonal tension failure, a sliding shear failure occurred after relatively few cycles of reversed loading beyond the elastic limit (6,7). Beams having a combination of diagonal and stirrups shear reinforcement were not tested because it was considered that stirrups can make no worthwhile contribution to sliding shear resistance. The stirrups used in diagonally reinforced coupling beams, such as seen in Fig. 4, must be provided for basketing purposes primarily, i.e. to prevent the falling off of large concrete fragments after the development of intersecting diagonal cracks during a very large earthquake.

It is emphasised that the inelastic seismic response of reinforced concrete members in shear cannot be predicted from our knowledge which has been based on the traditional concepts of shear and diagonal tension and on the results of monotonic load tests. It should also be noted that the suggested form of diagonal reinforcement is not meant to augment the flexural and the stirrup shear reinforcement in a beam, but it is designed to replace simultaneously the role of both. Therefore the proposed arrangements of reinforcement is not only the most efficient one when the nominal shear stress at a section is considerable, but it is also economical.

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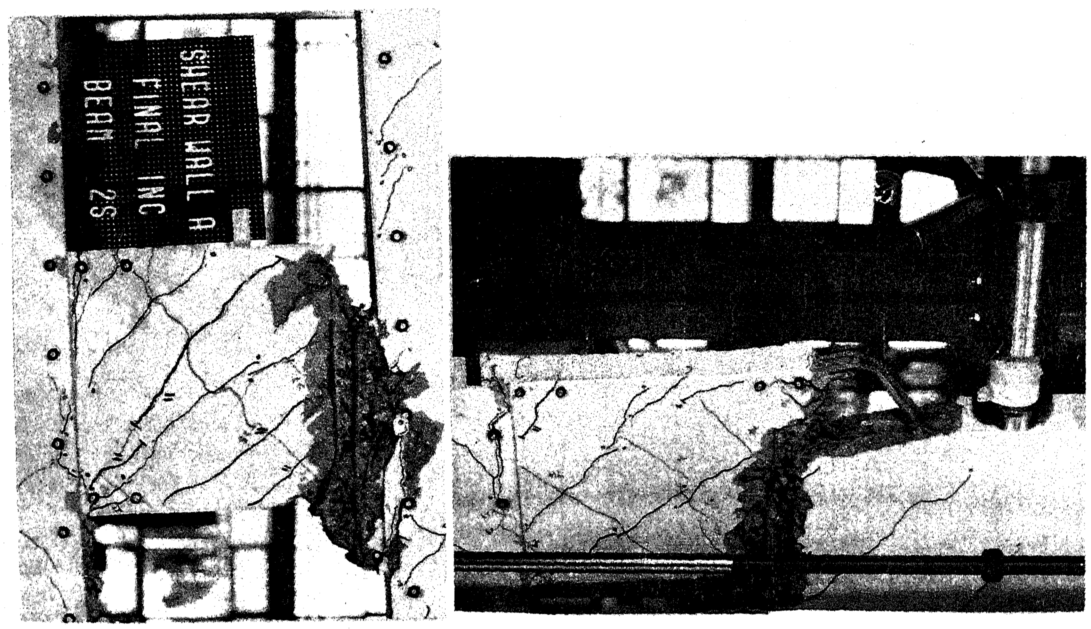

(a) At the 6th floor (b) At roof level

Fig. 7 Close-up of Coupling Beams of the Structure shown in Fig. 5.