DESIGN ASPECTS OF SHEAR WALLS FOR SEISMIC AREAS

by

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SYNOPSIS

The presentation, taking the form of a State of the Art report, considers several aspects of the behaviour of tall and squat shear walls. In particular the problems of brittle and ductile failure modes, diagonal tension, construction joints, alternating plasticity, sliding shear, stiffness degradation and strength loss under reversed cyclic loading are discussed. The behaviour of coupled shear walls is examined in some detail and the problems related to the components of this structure are reviewed. The principles rather than techniques of design for earthquake resistance are stated. Wherever possible the issues are studied against the background of experimental evidence. The material presented indicates that carefully detailed shear walls, designed to possess an intelligent hierarchy in their failure mechanisms, can be made to possess the properties so desirable in earthquake resistant structures.

1. INTRODUCTION

In many tall buildings shear walls will provide the major if not all the required strength for lateral loading resulting from gravity, wind and earthquake effects. Their incorporation into an overall plan, dictated by various functional requirements, will usually determine their geometry. For this reason it is not possible to specify a unified treatment for analysis or the evaluation of the limit of ultimate strength for shear wall structures. However, the fundamental behaviour of typical shear wall structures has been identified in numerous studies (1) (2) (3) (4) in which the techniques of elastic analysis have been used or suitably modified to evaluate internal load disposition, stresses and deformations. Unfortunately at present only limited experimental evidence is available from which the range of validity of such analysis, as applied to reinforced concrete shear walls, could be gauged.

For wind loading generally the governing design criteria will be deflection. When drift limitations in tall buildings are satisfied it is usually found that it is not difficult to provide for strength requirements using appropriate load factors specified by codes (5).

In addition to the limit states for strength and deflection the requirement for ductility arises for shear walls used for seismic resistance. Only in exceptional cases will it be possible to resist earthquake generated inertia forces within the elastic range of behaviour. In the case of a very large earthquake it is generally accepted that energy dissipation, involving considerable excursions into the postelastic range, will have to be relied upon. This nonlinear response of shear walls is further complicated by the fact that dynamic response to random vibratory motions is involved. Several specific seismic problems such as the effects of variable repeated and reversed loading, rate of loading, strength and stiffness degradations of reinforced concrete structures have been discussed and illustrated with experimental observations by Bertero (6). Many issues related to analysis, proportioning and detailing of reinforced concrete shear walls, to be faced in the design office, were raised by Popoff (7).

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The main purpose of this contribution is to review the strength criteria for shear walls when large intensity reversed cyclic loading is involved and when a demand for large ductilities in the shear wall structure and its components may exist. The source of ductility, i.e. energy dissipation, is briefly discussed with the intention of drawing designers' attention to desirable and possibly energy dissipating mechanisms and to point out the modes of potentially brittle failures under seismic load conditions. Attention was drawn to several issues raised here by Allen, Jaeger and Fenton in Canada (8), who critically examined the lack of consideration given by codes to ductility. They also proposed a classification for shear walls in terms of potential ductility. Some of their recommendations may be compared with experimental evidence offered in this contribution.

Shear walls, when carefully designed and detailed, hold the promise of giving the greatest degree of protection against non-structural damage in moderate earthquakes while assuring survival in case of catastrophic seismic disturbances on account of their ductility. This was clearly demonstrated for example by the "Banco de America" building which responded very successfully to intense shaking during the Managua Earthquake in 1972 (9).

2. POTENTIAL FAILURE MODES AND GEOMETRY

A single cantilever shear wall, such as shown in Fig. 1, can be expected to behave the same way as a reinforced concrete beam. The narrow cross-section suggests that problems of lateral instability may arise. However, floor slabs of multistorey buildings, when effectively connected to the wall, acting as stiffeners, provide adequate lateral strength.

Such shear walls, as large cantilevers, will be subjected to bending moments, and shear forces originating from lateral loads and to axial compression induced by gravity. Accordingly the flexural strength of the critical section, normally at the base of the structure, can be evaluated from first principles of flexure-axial load interaction. The vertical reinforcement in the web portion of such a wall may be considerable and its contribution to flexural strength must be taken into account to ensure that a proper margin between the shear strength and the flexural strength of the critical section is provided. As essential prerequisites, adequate foundations and sufficient connection to all floors, to transmit horizontal loads, must be assured.

2.1 The Flexural Strength of Tall Shear Walls

In shear walls with moderate height, particularly in areas not affected by earthquakes, the flexural steel demand may not be large. Consequently the vertical reinforcement is often uniformly distributed over the whole section. Such arrangement does not efficiently utilise the reinforcement when developing the ultimate moment. Moreover, the ultimate curvature, hence the curvature ductility may be seriously reduced (10). Fig. 2 illustrates, by means of an example wall section (10),that the ductility properties improve when the bulk of the flexural reinforcement is placed near the edges of the wall section. This will also enable a considerable portion of the flexural compression to be resisted by reinforcement when the wall is subjected to alternating reversed loading causing yielding. Where a shear wall carries also considerable gravity load in addition to large lateral forces, it may be necessary to improve the ductility properties of the critical region, normally at foundation level, by providing confinement to the concrete in the compression zones. Wall ties can be arranged the same way as in tied columns. Closely spaced transverse ties are required around vertical flexural bars, which may be subjected to alternating yielding and hence Bauschinger effect, to ensure that local buckling of individual bars does not occur. Tie spacing for such situations need be considerably less than the distance recommended by codes for columns receiving static gravity loads (11).

2.2 The Shear Strength of Tall Shear Walls

The shear strength of shear walls, with a height to depth ratio of more than 3, can be assessed the same way as that of beams. Due allowance can be made for the contribution of the axial compression (5) in boosting the share of the shear resisting mechanisms, other than web reinforcement (12). This share is usually referred to as the contribution of the concrete. In doing so the adverse effect of vertical accelerations, induced by earthquakes, should be considered. At the base of the wall, where yielding of the flexural reinforcement in both faces of the section can occur, the contribution of the concrete towards shear strength should be neglected where the axial compression on the gross cross section is less than 12% of the cylinder crushing strength of the concrete. This relatively small compression may be offset by vertical accelerations inducing tension. Moreover, cyclic reversed loading, causing diagonal cracking in two directions, is known to diminish rapidly the shear strength of the concrete. This implies that in most cases horizontal stirrups need be provided to resist the whole of the shear force generated by the lateral forces near the base of the wall. This shear reinforcement must extend at least over the possible length of the plastic hinge, where yielding of the flexural bars will affect also the width of diagonal cracks. The length for full shear reinforcement should not be taken less than the overall depth (D in Fig. 1) of the shear wall section. Tests show that the existing requirements of the American Concrete Institute (5) predict conservatively the shear strength of shear walls under monotonic load conditions (13).

Where it is essential that the lateral and gravity strength of a tall shear wall be maintained in a ductile manner, as is the case in seismic areas, every attempt must be made to suppress a shear failure. This is only possible if the shear force, associated with the maximum possible flexure strength of the critical section, taking into account the increased yield strength of the flexural reinforcement due to strain hardening, is provided for in such a way that the shear (web) reinforcement will not yield. It is emphasized that all vertical reinforcement, including nominal temperature steel, need be included in the evaluation of the flexural capacity of the critical wall section.

2.3 Construction Joints Across Shear Walls

Earthquake damage in shear walls of high rise buildings has often been observed at construction joints along which sliding movements occurred (9) (14). For monotonic loading the failure along a construction joint may be quite ductile. However, after load reversal very large slips and a reduction of shear resistance result (15). The failure mechanism dissipates diminishing energy under cyclic loading when the reinforcement crossing the joint yields. Fig. 3 shows the shear stress-shear slip relationship for a construction joint specimen which was subjected to a few cycles of reversed static loading. It should be noted that after yielding slips in excess of 0.1 in each direction have been recorded. These involve damage almost beyond repair. It is evident that this mechanism is unsuitable in earthquake resistant structures. For this reason a failure along construction joints should be suppressed the same way as diagonal tension failure due to shear is to be prevented by all means, in order to allow a ductile flexural energy dissipating mechanism to develop when required.

The dominant mode of shear transfer across a horizontal construction joint is by aggregate interlock action, also called shear friction. Dowel action of the reinforcement, crossing a horizontal construction joint, once believed to be the major component of shear resistance, is relatively insignificant. The reason for this is that the full strength of dowel action can be mobilised only after a significant slip along the joint has occurred. By that time the shear friction resistance is greatly diminished. Fig. 4 shows the shear stress-slip relationship for construction joint specimens subjected to monotonic loading. The top curves show the shear stress transferred by shear friction only, using different surface preparations. The bottom curve on the other hand gives the contribution of the dowel action of the vertical bars only in terms of average shear stress. It is seen that up to a slip of 0.01 in. the contribution of the dowel action is negligible (15) and that dowel strength becomes significant at a very large sliding movement of 0.1 in. or more.

To ensure efficient shear transfer the potential crack along a construction joint must be prevented from opening so that the contact between the rough surfaces at either side of the crack is not lost. Thus the prime function of the vertical reinforcement, passing across a construction joint, is to supply the necessary clamping force and to enable friction forces to be transferred.

It does not seem to matter how the surface of the old concrete at a construction joint is prepared. As long as it is rough and clean and the freshly placed concrete, placed upon it, is well bonded to it, a shear force corresponding with a friction factor of unity can be repeatedly transferred with insignificant slip (15). The basic strength of a construction joint can be assessed as follows:

$$v_{uf} = \left(A_{vf} f_y + N\right)/A_g = \rho_v f_y + N/A_g \qquad \ldots \quad (1)$$

where v_{uf} = average ultimate shear stress transferable across a well prepared rough construction joint.

A_{vf} = total vertical steel to be utilised for the required clamping force.

A_g = gross area of a rectangular shear wall section.

N = axial force on the section taken as positive when producing compression.

f_y = the yield strength of the reinforcement used.

ρ_v = reinforcing steel content.

It is normal practice to provide a nominal minimum amount of vertical reinforcement equal to ρ_v = 0.0015 to 0.0025 and this steel will provide a considerable clamping force across construction joints. However, in the lower parts of shear walls, where large shear forces may be carried, this steel content may have to be considerably increased, in accordance with Eq. (1), if a sliding shear failure is to be suppressed. It is important that the required vertical reinforcement be provided at close spacing because the clamping force, supplied by each bar, is effective only in the bars' immediate vicinity. Reinforcement provided for flexure and situated near the extreme vertical edges of the shear wall, should not be included in the evaluation of the clamping force to be developed across the core of the section.

Tests have clearly shown (15) that when the reinforcement across the construction joint commences to yield the control over crack width is lost and hence a sliding shear failure is imminent. This point coincides with the top of the upper curves shown in Fig. 4. Sliding movement dislodges aggregate particles and drastically reduces the effective surface roughness so that the shear strength of the joint, particularly for reversed loading, is greatly diminished. For this reason mechanisms relying on shear friction for energy dissipation, sometimes advocated in precast panel construction for shear walls, are unsuitable for earthquake resistant structures.

2.4 Squat Shear Walls

In shear walls with a height to depth ratio of less than two it may be more difficult to ensure ductile flexural failure mechanisms. Fortunately such structures seldom occur in tall buildings. Their performance is sometimes compared with that of deep beams. Such comparisons must be evaluated with great care because the introduction of load, generating shear stresses, is generally very different in these two types of structures (16).

2.4.1 The behaviour of low-rise shear walls

In some low-rise buildings the height of the cantilever shear walls may be less than their length, i.e. their structural depth. In such situations the assessment of flexural and shear strength cannot be based on the conventional techniques applicable to tall walls. It is no longer possible to discuss separately flexure and shear as the two actions are more intimately interrelated in squat shear walls.

Low-rise shear walls usually carry small gravity loads and for this reason their beneficial effect, derived for shear strength, is best ignored. In most cases the demand for flexural steel will also be small because a relatively large internal lever arm is available. It will be more practical therefore to distribute the vertical (i.e. flexural) reinforcement uniformly over the full length of the wall with only nominal increase at the vertical edges. From Fig. 2 it is evident that such arrangement may mean some loss of ductility. This, however, is not likely to be of great importance because of three reasons. Firsly, the low steel requirement is often satisfied by a steel content close to minimum, i.e. 0.25%. As Fig. 2 reveals, this provides ample energy absorption in the postelastic range. Secondly, properly detailed squat shear walls can be made to absorb the whole, or at least the greater part, of the seismic shock in the elastic range without great demand for reinforcing content. Thirdly, the limit to the lateral load capacity of low rise shear walls is often set by the inability of the foundations to provide stability against overturning moment.

It has been attempted to predict the likely behaviour of squat shear walls from tests carried out on deep beams. Geometric similarities suggested such procedures. Usually the load and reactions are directly applied to the faces of the specimen the same way as in the case of the simply supported beam shown in Fig. 5. (a). This form of load application considerably enhances arch action and hence stirrups, crossing the potential diagonal failure crack, forming between load and support points, do not engage in load transfer. This type of test does not disclose the proper functioning of web reinforcement in squat shear walls.

In the common shear wall the load is introduced along the joint between floor slabs and walls as a line load. A more appropriate beam test, simulating this condition is shown in Fig. 5. (b). Clearly no effective arch action can develop with this type of loading. The failure of this beam in diagonal tension can be prevented only by sufficient vertical stirrups or bent up bars in the shear span. Leonhardt and Walther demonstrated this convincingly in tests with wall-beams (17).

The crack patterm, shown somewhat idealised in Fig. 6, shows the formation of diagonal struts and the engagement of the wall reinforcement in the shear resistance of a low-rise wall. From considerations of equilibrium in the triangular free body, marked 1, it is evident that horizontal stirrups are required to resist the shearing stress applied along the top edge. The diagonal compression forces, set up at 45^{o} in the free body, also require the same amount of vertical reinforcement.

In the free body bound by two diagonal cracks, marked 2, only vertical forces, equal to the shear intensity, need be generated in order to develop the necessary diagonal compression. This steel is often referred to as shear reinforcement even though its principal role is to resist the moment which tends to overturn free body 2.

2.4.2 Experimental evidence of squat shear wall behaviour

The failure modes and associated ductilities in square shaped shear walls were studied in a small project at the University of Canterbury (16) (18) and the more important findings are briefly reviewed here. Crack patterns in the three specimens, which were subjected to reversed cyclic static transverse loading only are presented in Fig. 7. The load was applied in such a way that the total shear was distributed along the top edge of the wall.

The top row of photographs in Fig. 7 shows Wall A which was deliberately underreinforced for shear. Stirrups were provided to resist the theoretical shear force P_u^* that would have developed if flexure governed the design. In the 12th load cycle the development of a diagonal failure crack became evident, as seen in Fig. 7.a. The effectiveness of stirrups in this squat shear wall was demonstrated by the fact that at failure one stirrup fractured.

Wall B, shown in Fig. 7.b, was identical in every respect to the previous specimen except that horizontal stirrups were provided in excess of the flexural capacity P_u^*, which corresponded with the application of moderate nominal shear stresses, i.e. $5.6\sqrt{f'_c}$ psi. The specimen carried loads in excess of its ultimate design capacity P_u^* in each of the "plastic" cycles and exhibited considerable ductility. As Fig. 7.b. shows, a flexural mode of failure became evident in the 13th load cycle. The response of the specimen to the applied cyclic loading is illustrated in Fig. 8.a. which shows the load-rotation relationship for Wall B. (The rotation is defined as the lateral deflection at the top of the wall divided by the height of the specimen.)

By providing more vertical reinforcement, particularly at the edges, the flexural capacity of Wall C was made approximately twice that of Wall B. This required the development of large shear stresses, i.e. approximately $10\sqrt{f'_c}$ psi. Horizontal stirrup reinforcement was provided for a shear force in excess of the flexural load capacity P_u^* and hence a flexural failure could be anticipated. As Fig. 7.c. shows a sliding shear failure occurred in the 12th cycle at 39% of the theoretical shear capacity of the specimen. This type of failure, typical of deep members when high intensity reversed cyclic shear is applied, cannot be prevented by additional stirrup reinforcement. Sliding shear failure would not normally be observed in tests with monotonic loading because the flexural compression zone remains relatively intact up till crushing and hence it is capable of transmitting the diagonal compression forces generated between diagonal cracks. As Fig. 8.b. shows, Wall C is not only less ductile than Wall B but a distinct loss of strength with "plastic" cycles is also evident.

The superior performance of Wall B suggests two important conclusions for design:

(a) If a ductile (i.e. flexural) failure mechanism is desired in a low-rise shear wall then the nominal shear stresses, associated with the flexural overcapacity the wall (also including allowances for strain hardening of the flexural steel) must be moderate, say $v_u < 5\sqrt{f'_c}$ psi. This is normally not difficult to achieve.

(b) Because the flexural failure mechanism is associated with large cracks, no reliance should be placed on the concrete in contributing towards shear strength. Consequently the whole of the shear force should be resisted by stirrups.

Compressive axial stresses cause diagonal cracks to be more evenly dispersed. Alexander and Heidebrecht found (19) that as a result less stiffness degradation is obtained in low-rise panels. Compressive load on the other hand was found to reduce the walls' capacity for ductility (19).

2.4.3 Shear walls with boundary elements

Shear walls are sometimes surrounded by peripheral frames that may contain substantial reinforcement. The behaviour of these, also containing openings, has been studied experimentally by Benjamin and Williams (20), Umemura and others (21)(22). Some researchers have attempted to evaluate the behaviour of such walls from the superposition of the frame action and the diaphragm action of the wall infill. The approach is justified when a homogeneous connection does not exist between the two structural components. This is the case with steel or reinforced concrete frames with masonry infill panels. A monolithically cast reinforced concrete shear wall with boundary elements, however, will tend to act as one unit. Every effort should be made in the process of designing and detailing to encourage this most efficient interaction.

Boundary elements, such as flanges, may considerably boost the flexural capacity of walls, even with a small steel content in these flanges, and the danger of non-ductile shear failure may arise. Barda, who studied particularly the shear mode of failure, found that the compression flange in squat shear walls is quite ineffective in carrying load (23).

2.5 Moment Axial Load Interaction for Shear Wall Sections

There is no reason to expect that tall shear walls with flanged cross sections would behave differently. In symmetrical sections usually the internal "steel couple" can carry the external moment and the contribution of the concrete is governed by the magnitude of the gravity load. When significant gravity compression is present the whole area of flanges may be subjected to compression when the tension steel is at yield. In such cases it may be necessary to provide secondary confining reinforcement in the compression flanges. In flanged walls much greater ductilities are obtainable than in walls with rectangular cross sections (9).

Flanges may boost the moment of resistance of shear walls so that shear stresses in the webs, causing diagonal tension cracks or possible slip along construction joints, may become critical. Appropriate horizontal and vertical shear reinforcement must be provided so that at no stage of the loading should yield be expected in such reinforcement.

The moment capacity of unsymmetrical wall sections, in the presence of axial load, needs to be assessed for each possible direction of the loading. As the overall sectional dimensions of tall shear walls remain usually sensibly constant over the height of the building, it is sometimes expedient to derive the complete moment-axial force (M_u-P_u) interaction for such sections. This enables the selection of the appropriate steel content at any level to be readily made. Fig. 9 shows such a chart for a channel shaped shear wall section with a sectional aspect ratio of 3, in which the vertical reinforcement is uniformly distributed (24). Positive moments are considered to cause tension and negative moments will induce compression in the web portion of the wall section. The radiating straight lines indicate the position of the neutral axis C, measured from the compression edge, as a fraction of the depth D of the section. This shows the extent of compression area at the development of full strength. In this region special transverse reinforcement for confinement of the concrete and for stability of compression bars, subjected to Bauschinger effect, may be required.

3. SHEAR WALLS WITH OPENINGS

Windows, doors and service ducts require that certain shear walls be provided with openings. It is important that at the early stages of the planning suitable decisions are made with respect to the positioning of openings so that a rational structure results. One may define a rational shear wall structure as one whose essential behaviour can be assessed by bare inspection.

Irrational shear wall structures usually defy solutions by routine techniques of structural analyses and in such cases model investigations or finite element studies may assist in the evaluation of the internal forces. This may be sufficient when only wind loading is to be resisted. However, only special experimental studies can disclose the important aspects of ultimate strength, energy absorption and ductility demand in irrational shear walls which have to survive severe seismic disturbances.

It is imperative that the openings to be provided should interfere as little as possible with the moment and shear carrying capacity of the structure. A good example of an irrational shear wall is shown in Fig. 10.(a). The flexural resistance of the cantilever structure is drastically reduced at the critical section near the base. The staggered arrangement of openings seriously reduces the contact area between the two walls where shearing forces should be transmitted. The sloping legs of the wall, illustrated in Fig. 10.(b) could lead to the undesirable situation whereby the above wall tilts in a direction opposite to the forces which tend to bring about the sway. Observations after earthquakes indicate that such structures invite disaster (25).

4. COUPLED SHEAR WALLS

Many shear walls contain one or more rows of openings. Common examples are "shear cores" of tall buildings which accommodate elevator shafts, stair wells and service ducts. Access doors necessarily pierce the walls of such cores. Therefore the walls are interconnected only by beams, often short and relatively deep, left between openings. It is customary to refer to such walls as being coupled. An idealised coupled shear wall structure and its deformations caused by lateral loading are illustrated in Fig. 11.

4.1 The Assessment of Behaviour and the Effectiveness of Coupling

Because of the great difference in the stiffnesses of the walls and coupling beams and the significance of certain deformations, normally neglected in ordinary first order frame analyses, manual techniques of structural analyses, when applied to coupled shear walls, will rarely provide sufficient accuracy. Apart from flexural deformations of the various components, the axial deformations of the walls and the shear deformations of the coupling beams need be considered. Standard computer programmes, which include the appropriate terms for these deformations, can be used to assess the linear elastic response of these structures.

In a mathematical model, much better suited for coupled shear wall structures, the discrete beams are replaced by an equivalent continuous set of connecting laminae. This idealisation enables the shear forces across the coupling system to be expressed as a continuous function of the height, provided that no discontinuities in structural properties or the external load pattern occur. The Beck-Rosman laminar analyses (26) (27) utilises this. It has been extended and slightly modified so as to deal with a number of load and boundary conditions. The technique is efficient and is well suited for sensitivity analyses if a designer happens to have the opportunity to select structural dimensions.

The overturning moment, M_o, generated at any level by the external lateral load is resisted by three internal actions. With reference to Fig. 12 these will be (a) a moment induced in Wall 1, M_1, (b) a moment induced in Wall 2, M_2, and (c) equal and opposite axial forces, T, generated in both walls. The corresponding equilibrium statement

$$M_o = M_1 + M_2 + \ell T \qquad \ldots (2)$$

shows the interaction between these three components of the internal moment of resistance. The axial force induced in the walls at any level results from the summation of the shear forces in the coupling system above that level. It is evident that efficient shear transfer between the two coupled shear walls will generate large axial forces and hence the ℓT component of the internal moment of resistance, defined by Eq. (2), is large. This is desirable because a full utilisation of a large internal lever arm, ℓ, normally determined from functional requirements for a building, will result in minimum internal forces and consequently a minimum in vertical wall reinforcement. Moreover, an efficient coupling system provides the greatest stiffness and hence ensures minimum lateral deflection and interstorey drift. It may be argued that this will tend to reduce the natural period of the structure and hence generally it will invite larger forces during a seismic disturbance.

Surprisingly, above a certain minimum stiffness of the coupling beams, the efficiency of shear transfer is not significantly affected by the stiffness of the coupling system. Fig. 13 illustrates this phenomenon. Here the shear core of a 20 storey building is examined. The two channel shaped walls are interconnected by 6'-0" long rectangular coupling beams. It is seen that the mode of internal moment of resistance is hardly affected when beams in excess of 24 to 30 in depths are used, in all cases very effective shear transfer being assured. The outermost curve gives the total external overturning moment, M_o, at any level. One may speak of efficient coupling if at least 50% of the external moment M_o is resisted by the ℓT component of Eq. (2).

In apartment houses it will not be possible to use deep connecting beams. However, the floor slab can provide some degree of coupling. The effective width of the slab will influence the stiffness of the coupling system. The elastic response of slab coupling has been studied and reported (1) but little is known about strength of the slab-wall junction under repeated reversed loading.

The pattern of the internal actions in coupled shear walls may be significantly affected by cracking (28). In particular the stiffness of deep coupling beams, shown as 1 in Fig. 11, is drastically reduced when, as a result of cyclic reversed loading, diagonal cracking in two directions sets in (29). For this reason in an elastic analysis allowance should be made for the loss of stiffness, at least in an approximate way, if meaningful results for high intensity loading are to be obtained.

4.2 Elasto-Plastic Behaviour of Coupled Shear Walls

The strength of a coupled shear wall structure under lateral load is developed when a satisfactory admissible collapse mechanism is formed. This involves two plastic hinges in the coupling beams and one plastic hinge in each of the walls. The sequence of hinge formation during the non-linear response of the structure to monotonic loading will depend upon the relative stiffnesses and strengths of the components. In earthquake resistant structures the spreading of yielding will be of great importance. Therefore the designer must aim at establishing an intelligent hierarchy in the formation of collapse mechanism. In fact collapse is not expected. Instead the coupled shear wall structure and its components must possess adequate ductility so that significant excursions beyond the yield level of loading can be made, several times in each direction if necessary, without significant loss of strength and capacity for energy dissipation. These desirable properties can be achieved if the coupling system is made to yield first. When the coupling beams have yielded over say 90% of the height of the structure the walls can be made to sustain the large load just below yield level. This will ensure that at the development of the near full strength of the structure no permanent damage in the walls will occur and hence no misalignment of the building results. The coupling beams, subjected at this stage to extensive yielding, lend themselves to repair. Moreover they are normally not part of the gravity load carrying system and thus even serious damage of them will not jeopardise the stability of the building after a disturbance.

When very large seismic excitations are encountered, further lateral deflections will occur and these will cause plastic hinges to develop also in the walls, normally at their base. The elastic laminar analysis can be conveniently extended to deal with this situation including partial (30) or full plastification of the laminae (4). At this stage large ductility demands may be imposed upon the coupling system (31).

Fig. 14 shows the results of an elasto-plastic analysis (32) for the structure illustrated in Fig. 13. Six different stages during monotonic loading, well into the plastic range, are shown here. Stage 1 represents full elastic response corresponding with a code specified lateral load, W , of the type shown in Fig. 13. At Stage 2 the laminar shear intensity, q , causes the first lamina to yield. By Stage 3 all laminae in the upper 90% of the height of the building have developed their full strength, q_u . Only a small increase in load is required to cause plastic hinges in the tension wall (Stage 4) and the compression wall (Stage 5) to develop. With this the theoretical full strength of the structure, W_u = 1332 Kips, is developed. Further entirely plastic deformations are assumed to increase so that the deflection at the 20th floor increases to over 25 in. A series of curves indicate the laminar ductility factors, in terms of the total rotation of the laminae at their boundaries, for these 6 stages of the loading. The wall moments, M_{1u} , M_{2u} , the axial force T_u developed at Stages 5 and 6 and the vertical steel content in each wall, ρ , are shown in the insert of Fig. 14.

It is evident that in earthquake resistant coupled shear wall structures the ductility of the coupling beams must be assured, unless the structure is designed for much higher lateral forces, consistent with an elastic dynamic response. If the coupling system can be made ductile enough and also efficient in shear transfer, so that the ℓT component of the internal moment resistance remains large as shown in Fig. 13, a very great proportion of the total energy to be dissipated will allocate itself to the coupling beams. With such dispersion of energy dissipation over the whole structure, a greater degree of protection of the wall bases will ensue.

4.3 The Strength and Ductility of Coupling Beams

Because coupling beams, to be efficient, are usually short and relatively deep, they can develop a large flexural strength with relatively small flexural steel content. This is associated with large shear forces, which in turn may dominate the behaviour of these beams. (See area 1 in Fig. 11). Observations after earthquakes, for example in Alaska (1964) and Managua (1972), have repeatedly shown that coupling beams containing insufficient web reinforcement fail by diagonal tension. (See Fig. 15.(a)). Such failures, reproduced in tests (34), are brittle and they follow a high rate of stiffness and strength degradation under reversed cyclic loading. (See Fig. 15.(b)). If a ductile failure mechanism is essential, as in seismic areas, this mode of failure must be suppressed.

It was found that the critical diagonal tension crack, along which a separation of a beam into two parts can occur, forms along a diagonal from one corner to another, not necessarily at an angle of 45° (29). Under cyclic loading the contribution of the shear resisting mechanisms, other than stirrups, quickly diminishes. For this reason in coupling beams the whole of the shear force, developed when the flexural tension steel yields at the face of the coupled walls, need be resisted by stirrups. Again, due attention must be paid to strain hardening of the flexural bars to ensure that at the development of the maximum yield strength in these the stirrup reinforcement will operate in the elastic range.

The disposition of internal forces in coupling beams, with a span to depth ratio of less than 1.5 and conventional top and bottom flexural bars, is different from that encountered in normal reinforced concrete beams (29). Interaction between flexure and shear does exist and the development of flexural strength, based on the location of the internal concrete compression resultant at limit strain of the extreme fibre, is somewhat inhibited. The customary assumption that plain sections remain plain does not hold. The flexural reinforcement at the top and the bottom of a coupling beam may be in tension over the entire clear span. Ductility in such beams under cyclic reversed loading can be expected to be available only if the average shear stresses at the development of flexural yielding is low, i.e. of the order of $v_u < 5\sqrt{f'_c}$ psi.

When the ultimate shear stress to be developed in coupling beams is higher, i.e. when it approaches the maximum value recommended for example by the current ACI Code (5), a sliding shear failure adjacent to the face of the walls will occur after a few load reversals at ultimate. A typical example of a test beam that failed in this manner is shown in Fig. 15.(c). Only limited amount of ductility is available in such beams even though considerable dowel forces can be transmitted by the flexural reinforcement. This is mainly due to kinking in these bars after large transverse sliding movement occurs. Hysteresis curves for shear-displacement relationships in such beams clearly show (34) large loss of stiffness at very low loads. Hence such beams must be considered as being unsuitable when large ductilities are expected to be developed in coupled shear walls.

To overcome the limitations of conventionally reinforced coupling beams the principal reinforcement can be placed along the diagonals of the beams. Fig. 16 shows the model for such a beam and from this diagram the disposition of external and internal forces and the mode of moment and shear resistance is self explanatory. It should be noted that this reinforcement replaces entirely the horizontal flexural bars and the vertical stirrups of conventionally reinforced coupling beams. At first loading the diagonal steel, consisting of a number of bars in a cage, can resist all the diagonal tension, while in the other direction the diagonal compression is resisted by reinforcement and the concrete surrounding it. The strength of the beam is limited by the yield strength of the diagonal tension bars. As yielding can develop over the entire length of these bars, extending across the beams, large yield displacement can occur. Upon load reversal the bars previously in tension must first yield in compression before the wide cracks in the concrete can close, and this will then enable the compression to be transferred also by the concrete. In spite of Bauschinger effect, at this stage, the diagonal compression reinforcement will carry the bulk of the diagonal compression force. For this reason it may be said that in such beams, after reversed cyclic loading causing yielding, all forces are essentially carried by the diagonal sets of reinforcing bars. Using steel which has a long yield plateau, very large ductilities can be achieved. Test beams have shown very stable hysteretic characteristics (35).

A typical arrangement of such reinforcement for an example coupling beam is shown in Fig. 17. If the compression yield capacity of diagonal reinforcement is to be sustained, and this indeed is a prerequisite for fully utilising the desirable qualities of such beams, the buckling of individual bars, after possible spalling of the cover concrete, must be prevented. Closely spaced individual ties or rectangular spirally wound continuous binding will ensure this. Allowing for the lower buckling strength of such bars, because of Bauschinger effect due to alternating plasticity, a maximum ties spacing of 4 times the diameter of the principal diagonal bars is suggested. This secondary reinforcement also serves the purpose of confining the concrete within the cage, formed by at least four diagonal bars. This concrete contributes towards the flexural rigidity and assists in preventing the buckling of this strut as a whole at right angles to the plane of the beam.

Additional nominal basketing reinforcement, consisting of stirrups and intermediate horizontal bars, as shown in Fig. 17, need be provided only for crack control and to prevent the dislocation of large concrete particles after disastrous seismic shocks. Fig. 15.(d) shows a test beam so reinforced, after 13 cycles of loading in each direction. The cracks in this beam are very large, but at this stage of the test it still sustained 94% of its theoretical ultimate shear capacity at an end rotation of 0.061 rad. (3.5°).

The diagonal reinforcement needs to be well anchored in the two adjacent shear walls which are to be coupled. Large tensile forces have to be transmitted to to the walls by a bundle of diagonal bars and for this reason the anchorage length to be provided should be larger (say by 50%) than that specified by codes for individual bars. Alternatively the bars may be splayed so that bond forces from individual bars are more widely dispersed in the anchorage zone.

Conventionally and diagonally reinforced coupling beams were subjected to the same kind of cyclic reversed loading and this enabled a comparison to be made with respect to ductility. The ductilities imposed in each load cycle, causing inevitably some loss of strength, together with the load sustained, were compared. Fig. 18 presents the results in terms of cumulative ductilities and shows the superior performance of diagonally reinforced coupling beams.

4.4 The Strength of Coupled Walls

Once the load, received by one of the coupled shear walls, at one hand from external loading and on the other from internal shear transfer across the coupling system at full strength, is evaluated, the wall can be treated as a cantilever. At the critical section, above foundation level, the reinforcement can then be determined, for example with the aid of an interaction relationship, such as shown in Fig. 9. Shear reinforcement, confining steel and other details need to be provided as outlined in Section 2. Particular attention must be paid to shear strength in the presence of possible net tension in the wall, and to construction joints. These areas are indicated at 2 and 3 in Fig. 11. There is evidence (36) that at the development of plastic hinges in the coupled walls substantial redistribution of the shear resistance occurs. A considerable portion of the shear force resisted by the tension wall before the onset of extensive yielding in the flexural reinforcement may be transferred to the compression wall. This wall, however, could accept the excess shear because at this stage of the loading it is under larger axial compression and thus a large portion of the shear force can be resisted by mechanisms other than stirrups.

4.5 Evidence of Ductility in Coupled Shear Walls

The behaviour of cantilever shear walls and that of coupling beams was briefly discussed in the previous sections. It was pointed out that with careful arrangement and detailing of reinforcement large ductilities can be achieved.

Some convincing evidence is now available to show that the ductilities experimentally obtained for individual shear wall test components can be relied upon in a complete coupled shear wall structure. This evidence strongly indicates that coupled shear walls may be designed so as to possess all the features so desirable when large lateral forces, such as occur during very severe seismic disturbances, are to be resisted. Therefore the highlights of two tests, carried out in 1973 at the University of Canterbury (36) are very briefly reported here.

Two one quarter full size seven storey reinforced concrete coupled shear walls were tested under simulated earthquake loading. The simulation consisted of three lateral static point loads, representing a triangularly distributed load similar to that shown in Fig. 1 and applied at the 3rd, 5th and 7th floors, in each direction several times. Some of the load was limited to produce stresses within the elastic range, but generally large yielding was imposed upon the structure in a cyclic fashion. To qualify as a ductile structure, design practice in New Zealand calls for the ability of the structure to deflect under lateral load, say at roof level, four times as much as the deflection which would occur at the onset of yielding or at the attainment of the specified ultimate lateral load. This deflection must then be sustained at least four times in each direction with a strength loss not exceeding 20% of theoretical ultimate. Consequently the test specimens were subjected to this or more severe lateral displacements.

Wall A, shown in Fig. 19, contained short and relatively deep coupling beams reinforced in a conventional manner. The wall and beam reinforcement was proportioned so that the coupling system was expected to yield before the full strength of the two walls was approached. The gravity load, corresponding with typical tributary floor areas in a seven storey building, was replaced by vertical prestressing in the walls, using a single central ungrouted cable. Shear reinforcement and anchorages were provided throughout the structure so that flexure dominated the behaviour in accordance with the principles discussed in the previous sections. After several cycles of high intensity reversed loading it became evident that sliding shear failure in the coupling beams was going to occur.

Fig. 19 shows the damaged Wall A after the completion of the test. The large sliding movements along one or both faces of the coupling beams and the development of well defined plastic hinges at the wall bases are evident. The load-displacement (roof level deflection) history of the structure may be seen in Fig. 20.(a). It is also seen that the structure would meet the overall ductility requirements as defined previously. The model structure, as shown in Fig. 19, maintained 80% of its theoretical ultimate load capacity, P_u^*, at a 7th floor deflection equal to one half of a storey height. This is evidence of very large ductility indeed. The hysteresis curves in Fig. 20.(a) show, however, the effect of progressive damage upon diminishing stiffness at low loads and the loss of energy dissipating capacity.

Wall B was identical in every respect to the previous specimen, except for the couplings. These were diagonally reinforced and were expected to behave as outlined in section 4.3. The reinforcement and the damaged wall is shown in Fig. 19. The stable hysteresis loops shown in Fig. 20.(b) demonstrate the remarkable ductility and the excellent energy dissipation capacity of this specimen. The load attained several times in both models P_i exceeded the theoretical ultimate strength P_u^* because of strain hardening effects in the mild steel used. Fig. 20.(b) shows the characteristics of a steel member subjected to reversed cyclic loading and it convincingly demonstrates that a suitably designed and detailed coupled shear wall can be ductile enough to sustain, without strength loss, the largest displacements expected in a very large seismic disturbance.

The effectiveness of floor slab coupling between two storey microconcrete shear wall models under monotonic loading was studied by Mirza and Jaeger (37) and it is hoped that this investigation will extend to include also simulated seismic load conditions.

4.6 Design Principles for Ductile Coupled Shear Walls

To ensure a satisfactory performance, when coupled shear walls are subjected to severe lateral loading, such as result from seismic shocks, it is necessary to be able to assess, at least approximately, their behaviour in both the elastic and plastic range of loading. Desirable behaviour can be expected only if the structure is made capable of following a preferred sequence of yielding. From the point of view of damage control and possible repair, it is desirable that the wall components be the last ones to suffer in the process of imposing incremental ultimate conditions.

Considerations of the strength of conventionally reinforced coupling beams indicate that full protection against diagonal tension failure, which is an unsuitable energy dissipating mechanism during cyclic loading, is required. Therefore, the flexural steel content in both faces of such beams must be moderate to avoid early failure by sliding shear. Deep coupling beams subjected to large shear forces cannot be expected to be ductile enough to sustain plastic deformations associated with an overall ductility of 4 for the structure as a whole. When conventionally reinforced coupling beams are used the ductility demand on the structure must be limited. Satisfactory performance of these beams will be assured for wind loads.

When diagonal reinforcement is used in coupling beams and adequate ties are provided to enable the compression struts to sustain yield load without buckling, very ductile performance can be expected.

The walls which are being coupled can be proportioned in accordance with the principles of reinforced concrete sections at limit states of strains. If necessary axial load-moment interaction relationships, taking into account lack of symmetry, can be used. Particular care in detailing the potential plastic hinge zones is a prerequisite to ensure that premature shear failure, as a consequence of alternating reversed flexural yielding, does not occur, and that adequate stability for yielding compression bars and confinement of compressed concrete is available. This will ensure that large ductilities at the base of coupled shear walls will also be available when required during extreme seismic conditions.

To ensure that the design axial load on the walls is not exceeded, mild steel with a well defined yield plateau (Grade 40), should be used in the coupling system and due allowance should be made for increased shear transfer due to strain hardening. In all other parts of a coupled shear wall benefit may be derived from early strain hardening of high yield (Grade 60) reinforcement.

The principles outlined in this section are equally valid and readily applicable when more than two walls are coupled by rows of beams between them.

5. CONCLUSIONS

(a) The behaviour of tall cantilever shear walls can be predicted from first principles of reinforced concrete behaviour. When seismic forces are to be resisted ductile response becomes a prerequisite.

(b) Shear, anchorage and instability failure mechanisms must be suppressed to ensure that shear walls can develop their intended lateral load capacity in a ductile manner.

(c) Ductile flexural response with large yielding can generally be assured in shear walls because a large proportion of the overturning moment is usually resisted by reinforcement only. When gravity loads become large, precautions may have to be taken to boost the deformability of compression zones by appropriate confinement.

(d) Shear stresses may dominate the behaviour of squat shear walls or wall elements. The behaviour of such walls has been extensively studied and reported by Tomii (22). With proper arrangement of reinforcement, which can relieve the concrete in carrying shear stresses, even low-rise shear walls can be made ductile (18). Squat shear walls should also be considered as flexural members. The shear force associated with the development of flexural capacity, the assessment of which must be based on all bars that can possibly contribute to flexure, should be carefully evaluated so that appropriate shear reinforcement can be provided. In many cases ductility will not be required in low-rise shear walls because the structure can comfortably resist the seismic forces within the elastic range.

(e) Reinforced concrete shear walls that are sometimes provided with boundary frames should be considered as one integral unit. Boundary frames act as flanges. They may accommodate the bulk of the vertical flexural reinforcement and they provide stability against possible lateral buckling. The separate treatment of boundary frames and infill panels in monolithic reinforced concrete construction is not in accord with the natural behaviour of such structures.

(f) Cantilever shear walls can provide excellent resistance against lateral load and can greatly reduce deflection. However, for seismic conditions they offer only a single line of defence. Should a large excitation require yielding this is likely to cause permanent deformation near the base and it may lead to early misalignment in the building.

(g) When openings are to be provided in a shear wall this should be done in such a way that a regular structure results in which flexure and shear can be rationally evaluated and effectively resisted.

(h) Regular arrangements of openings may enable coupled shear walls to be formed. In seismic areas it is essential that the coupling beams rather than the walls forms the weaker elements. With suitable detailing coupled shear walls hold a great promise of giving a large degree of protection by being both efficient in load resistance and sufficiently ductile. Energy dissipation when required, can be well dispersed over the entire structure and thus, as opposed to single cantilever walls, several lines of defence may be mobilised when extreme displacements are imposed upon a building.

(i) Horizontal construction joints may present potential weaknesses in all types of shear walls. By utilising clamping forces, provided by gravity and reinforcement, in accordance with the concepts of shear friction, this mode of failure can be eliminated. It is essential, however, that the maximum possible shear force, which may be induced, be evaluated from the most probable maximum flexural strength potential of the structure.

(j) The potential plastic hinge length at the base of a shear wall may be extensive. When detailing the reinforcement, the designer should ensure that no lapped splices of the possibly large size vertical bars occur in this area. Often it will be impossible to carry all the vertical wall steel continuously up from the foundations. In such cases welded connections or mechanical splices will be required to eliminate the necessity of having to transfer large bond forces to the concrete that is likely to become severely damaged in the plastic zone.

(k) In many buildings shear walls may interact with each other and with rigid jointed space frames. Problems related to the elastic response of such structures have been studied (38) (39) (40). However, only limited information is available with respect to the nonlinear response of such complex structural systems (41) and it is hoped that research in progress will contribute soon to the further understanding of the issues involved (42).

(l) The limit states of cellular, thin walled shear wall cores, particularly for high intensity cyclic reversed loading, still require further study. Because of their potential flexural capacity critical conditions may arise when large shear forces need to be transferred in thin webs that were thoroughly cracked during previous cyclic reversed yield loads.

(m) When shear wall structures with some static indeterminancy are available, it is desirable that an advantageous sequence in the propagation of yielding be established, so that damage in repairable and less critical areas will occur first and that the principal gravity load carrying units receive the greatest degree of protection. Therefore the designer must establish an intelligent hierarchy in the most probable strength levels which he intends to provide for each shear wall component. These are the properties that are recognised during seismic excitations.

(n) In the process of concentrating on the characteristics of the non-linear response of shear wall structures the designer must not overlook the limits set by the foundations. Many shear wall structures in seismic areas will never approach yielding because the overturning capacity at foundation level will limit the magnitude of the lateral forces that can be generated. The seismic response of shear walls with rocking foundations, involving superstructure-soil interaction, is an exciting challenge for further research.

6. ACKNOWLEDGEMENTS

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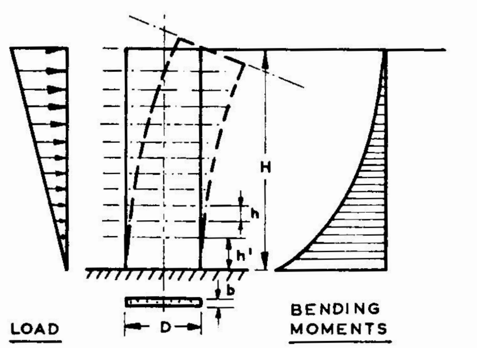


Fig. 1 - A CANTILEVER SHEAR WALL

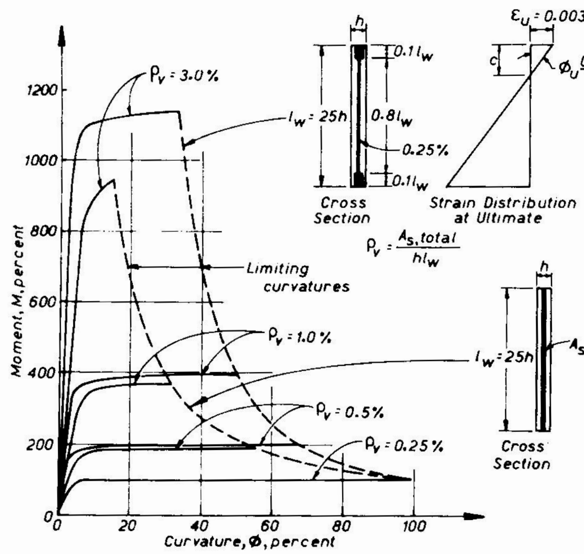


Fig. 2 - THE EFFECT OF THE AMOUNT AND THE DISTRIBUTION OF VERTICAL REINFORCEMENT UPON ULTIMATE CURVATURE (10)

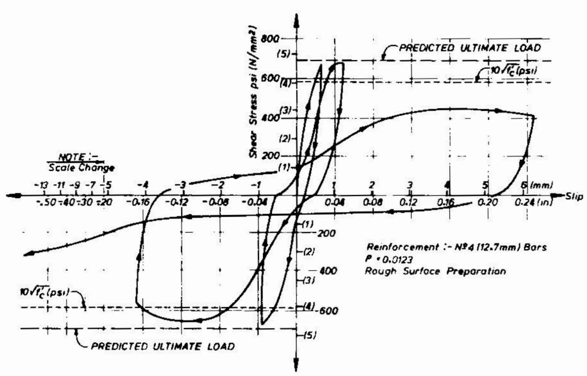


Fig. 3 - SHEAR STRESS-SLIP RELATIONSHIP AT A CONSTRUCTION JOINT TRANSFERRING CYCLIC REVERSED SHEAR FORCES (15)

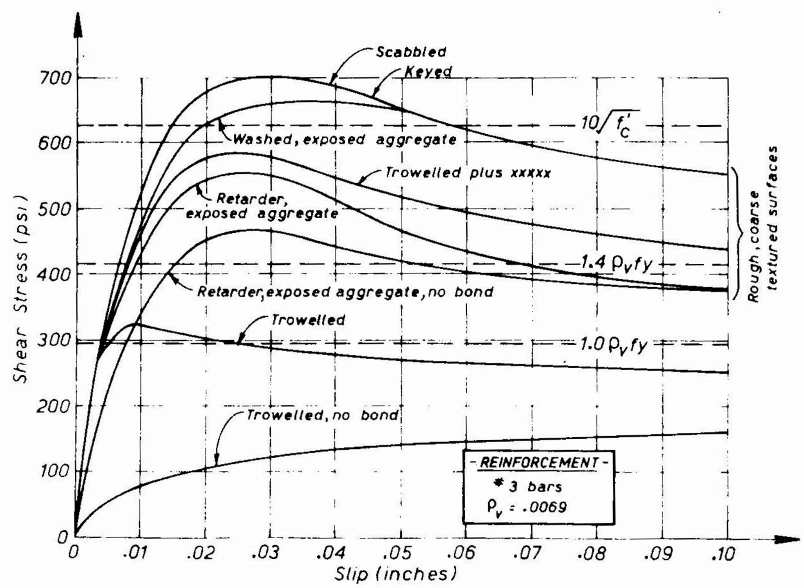


Fig. 4 - SHEAR STRESS-SLIP RELATIONSHIP FOR CONSTRUCTION JOINTS WITH DIFFERENT SURFACE PREPARATIONS (15)
Note that:

f'_c = concrete cylinder strength in psi,

f_y = yield strength of reinforcement crossing the joint,

ρ_v = reinforcing content

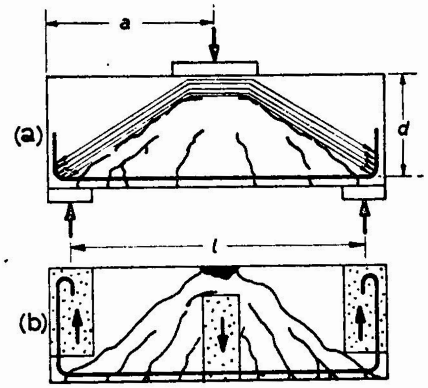


FIG. 5 - SHEAR TRANSFER IN DEEP BEAMS BY (a) EFFECTIVE ARCH ACTION (b) OTHER MEANS THAN ARCH ACTION (16)

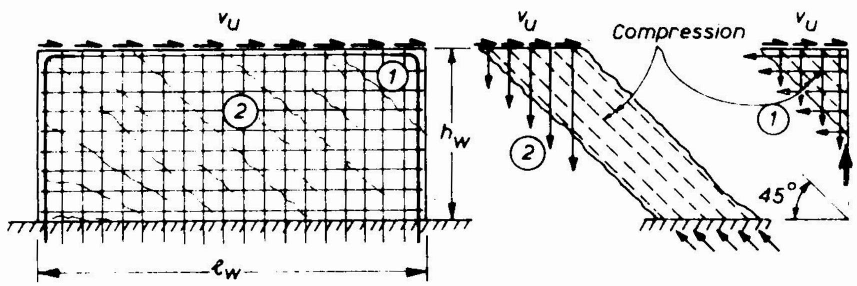


FIG. 6 - THE SHEAR RESISTANCE IN LOW-RISE SHEAR WALLS (16)

FIG. 7 - SEE PAGE 23

FIG. 8 - SEE PAGE 24

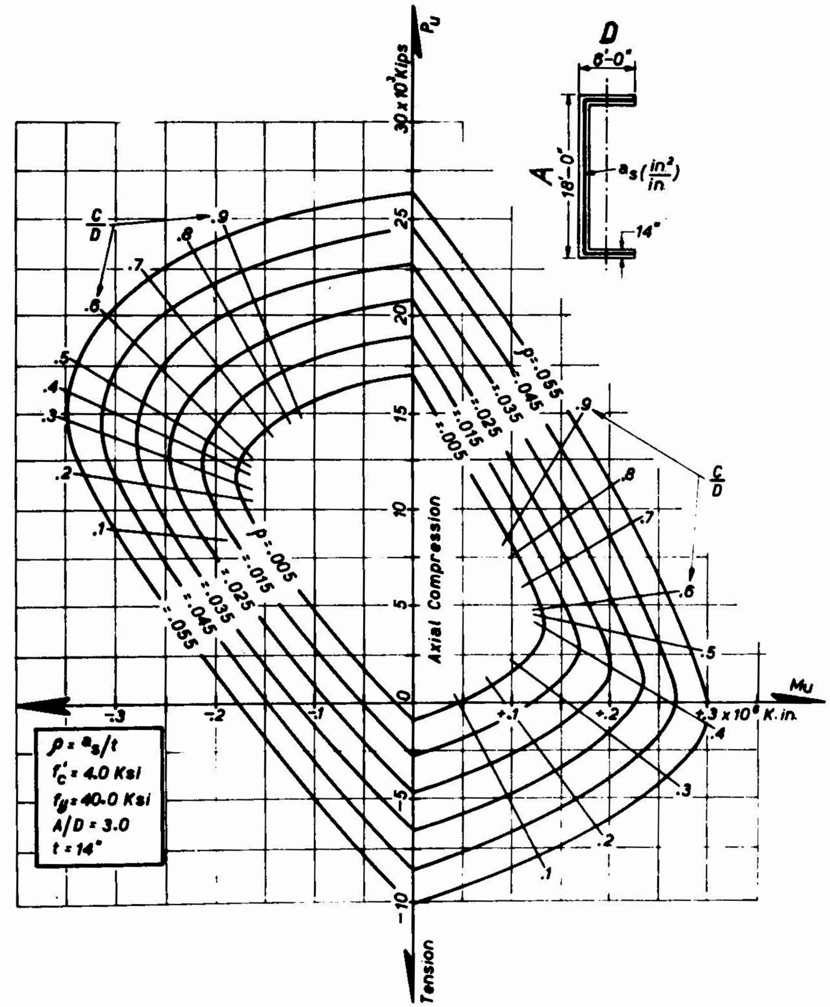
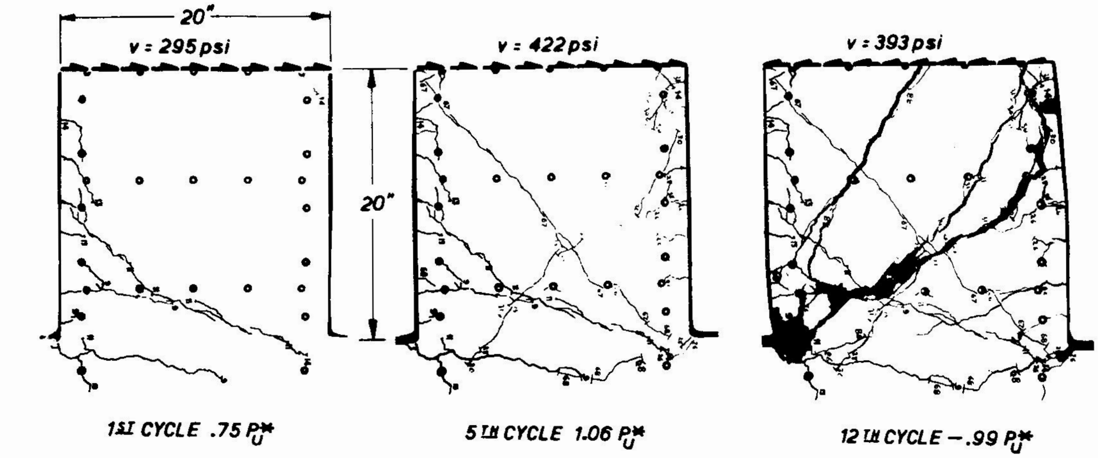
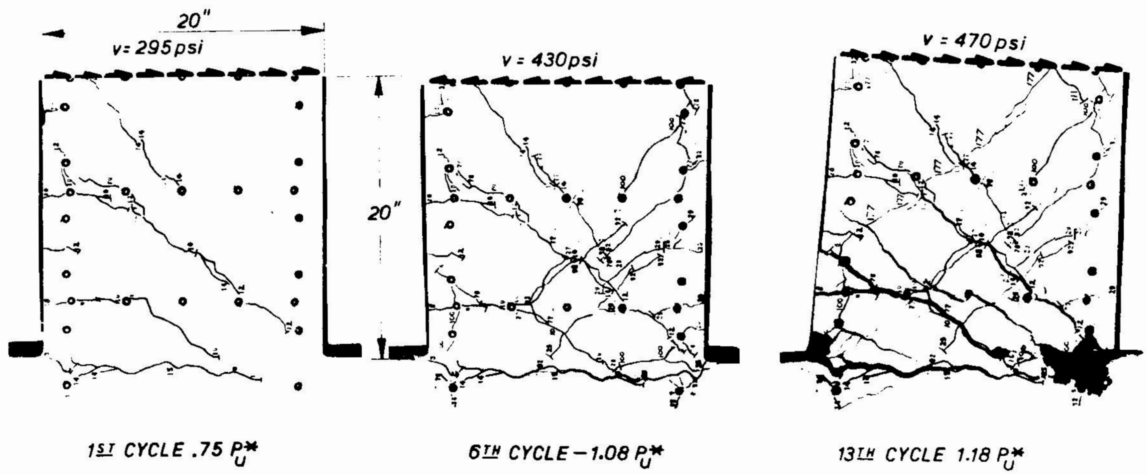


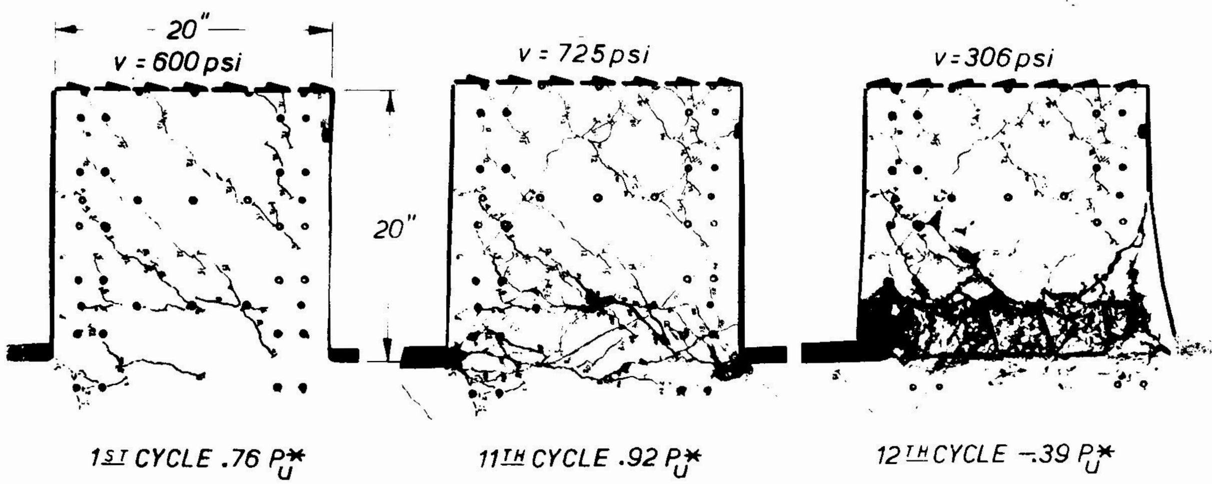
FIG. 9 - AXIAL FORCE-MOMENT INTERACTION RELATIONSHIP OF A CHANNEL SHAPED SHEAR WALL SECTION (24)



(a) Wall A - Diagonal Tension Failure

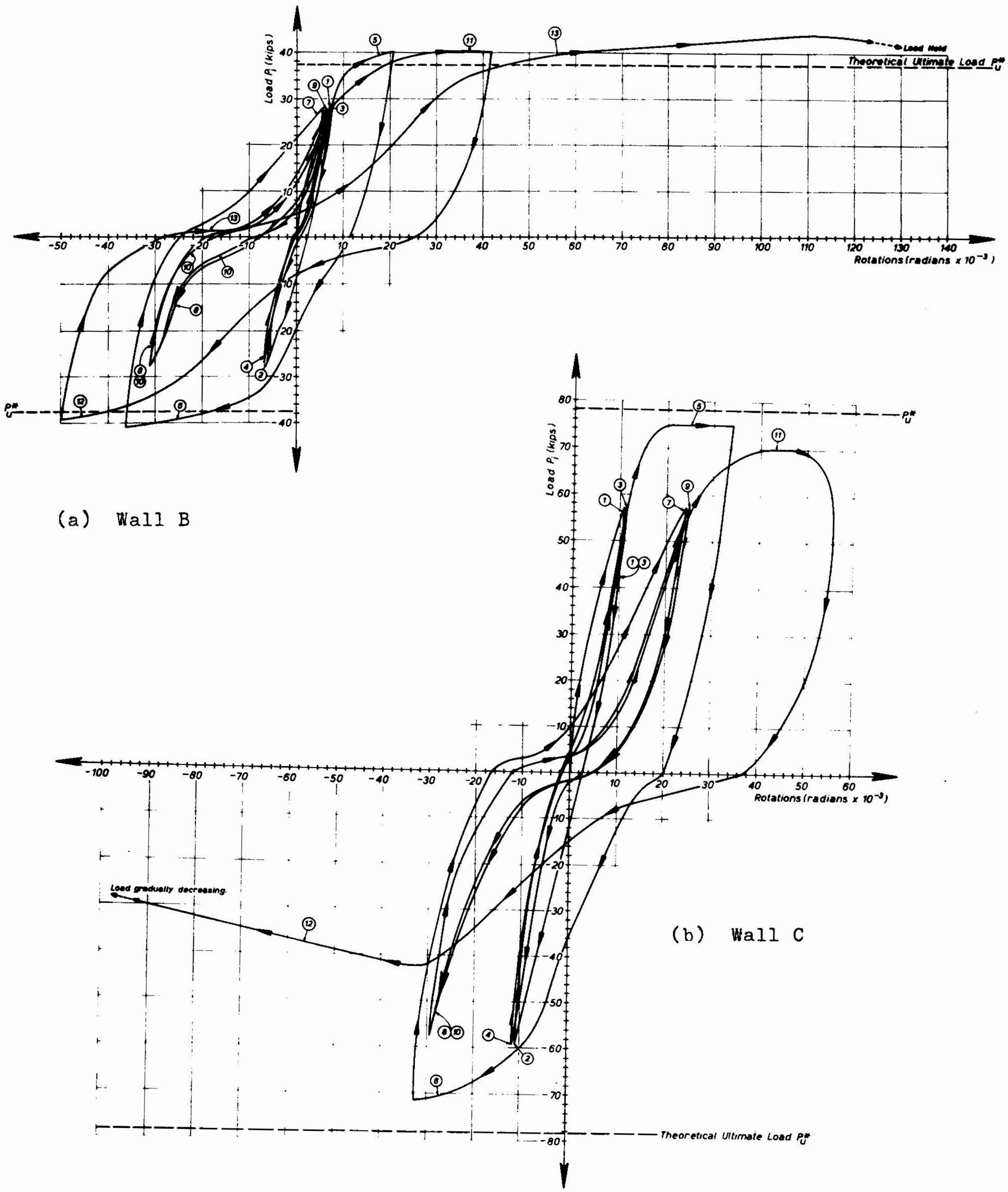


(b) Wall B - Flexural Failure



(c) Wall C - Sliding Shear Failure

FIG. 7 - THE FAILURE MODES OF THREE SQUARE SHEAR WALL MODELS (18)



(a) Wall B

(b) Wall C

FIG. 8 - LOAD-ROTATION RELATIONSHIPS FOR SQUARE SHEAR WALLS

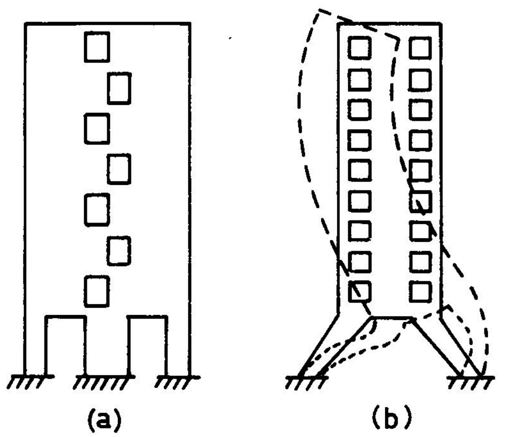


FIG. 10 - EXAMPLES OF IRRATIONAL SHEAR WALLS FOR SEISMIC AREAS

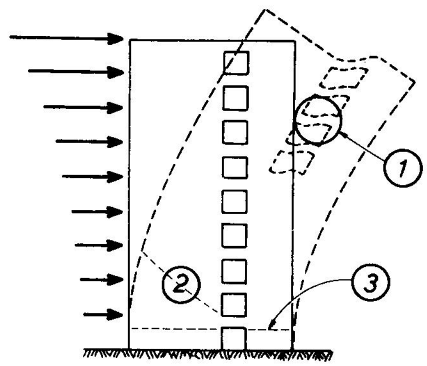


FIG. 11 - THE DISTORTIONS IN A LATERALLY LOADED COUPLED SHEAR WALL

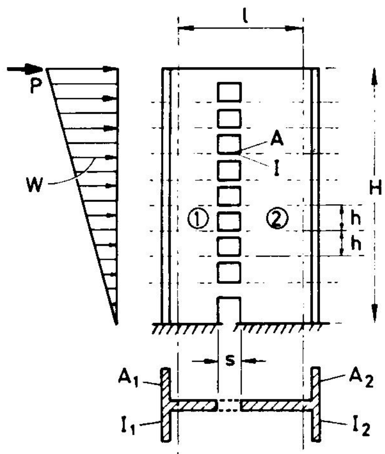


FIG. 12 - A TYPICAL COUPLED SHEAR WALL SUBJECTED TO LATERAL LOADING

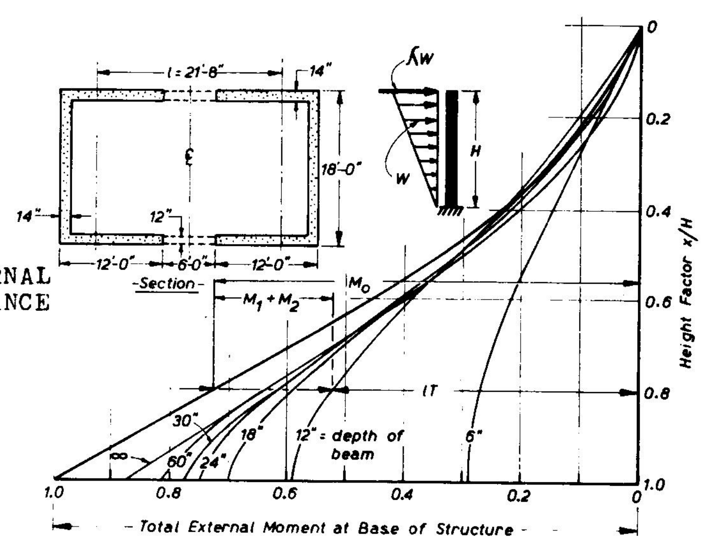
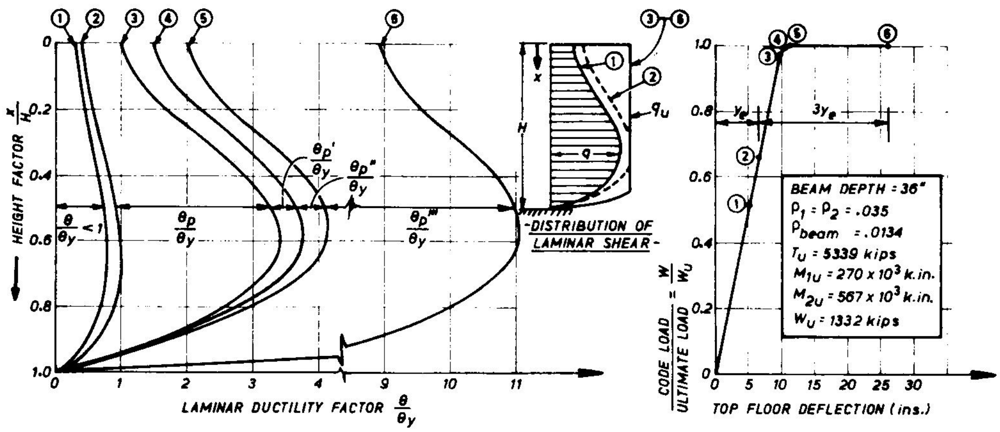


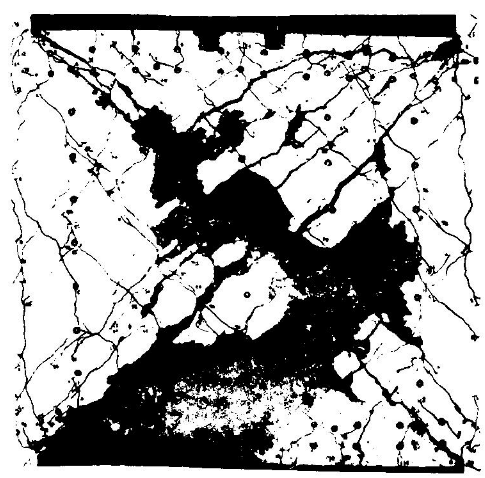
FIG. 13 - THE MODE OF INTERNAL MOMENT OF RESISTANCE IN COUPLED SHEAR WALI STRUCTURE



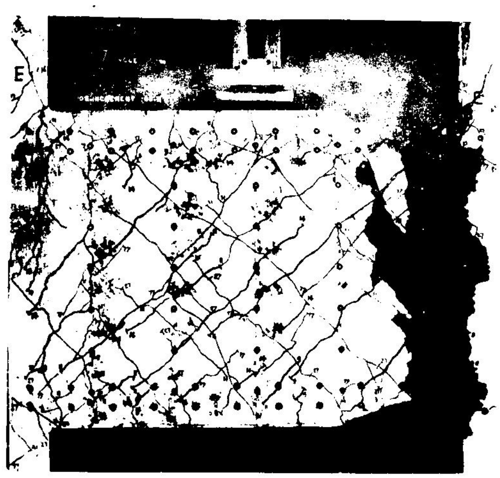
UNIFORM ULTIMATE SHEAR DISTRIBUTION

FIG. 14 - LAMINAR SHEAR FORCE DISTRIBUTION, DUCTILITY DEMAND AND TOP FLOOR DEFLECTIONS FOR A 20 STOREY COUPLED SHEAR WALL STRUCTURE WITH COUPLING BEAMS OF UNIFORM STRENGTH

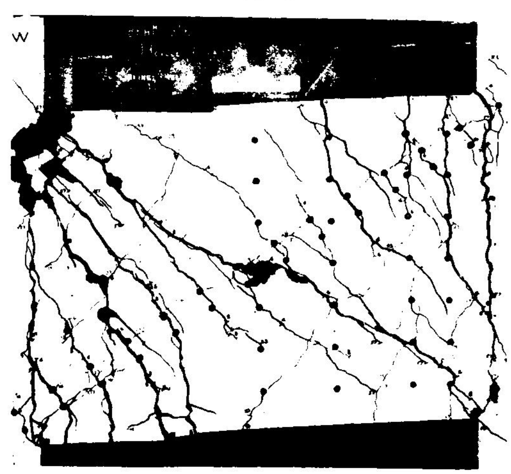
(a) Brittle Diagonal Tension, Alaska 1964 (33)



(b) Brittle Diagonal Tension in Test Beam (34)



(c) Sliding Shear (34)



(d) Ductile Diagonal Tension (35)

FIG. 15 - FAILURE MODES IN DEEP COUPLING BEAMS

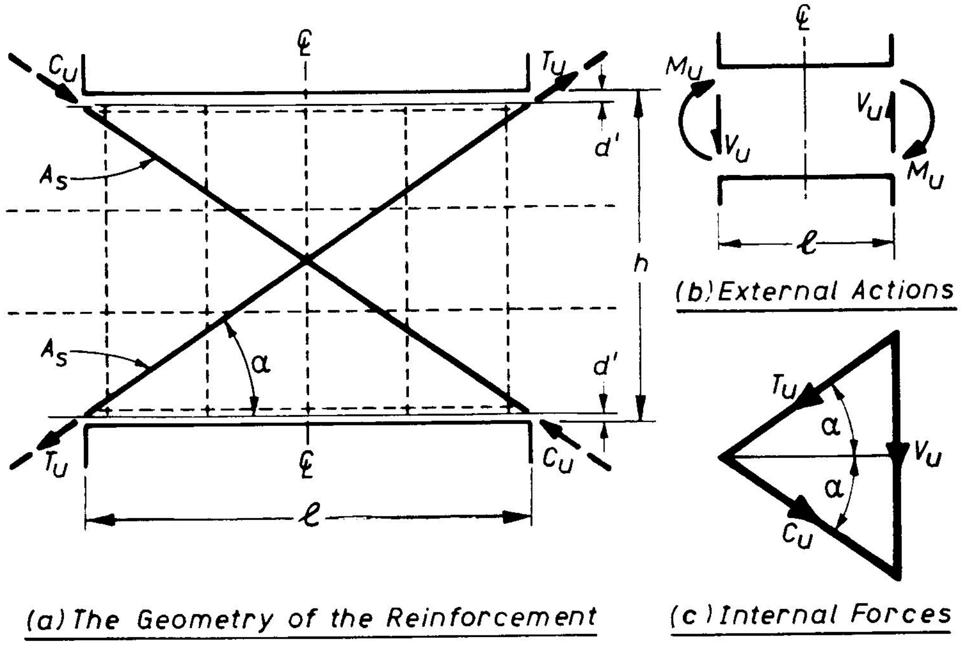


FIG. 16 - THE INTERPLAY OF EXTERNAL ACTIONS AND INTERNAL FORCES IN DIAGONALLY REINFORCED COUPLING BEAMS

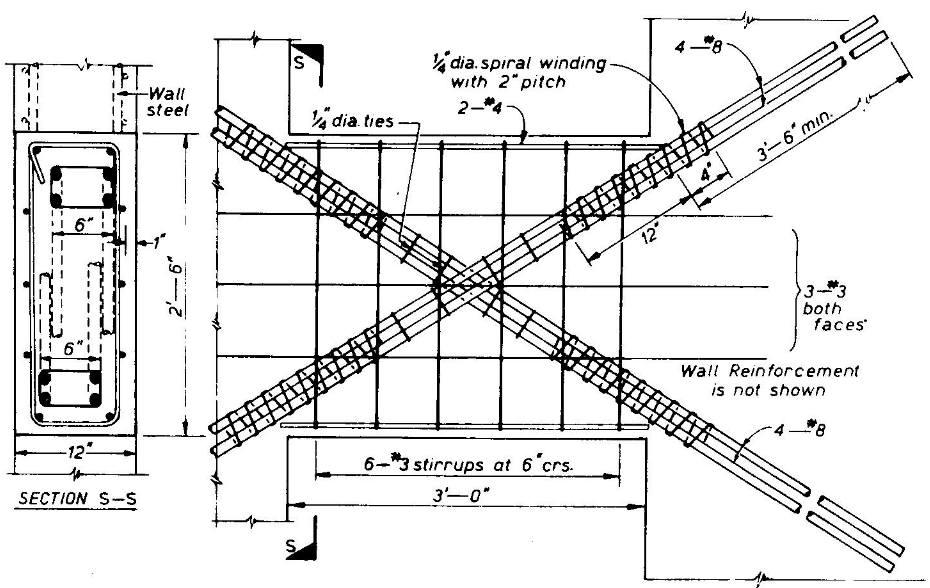


FIG. 17 - SUGGESTED STEEL ARRANGEMENT IN DIAGONALLY REINFORCED COUPLING BEAMS

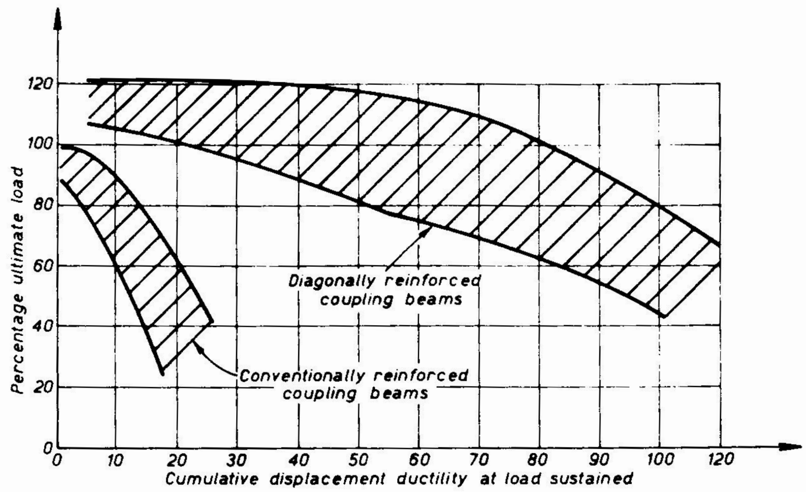
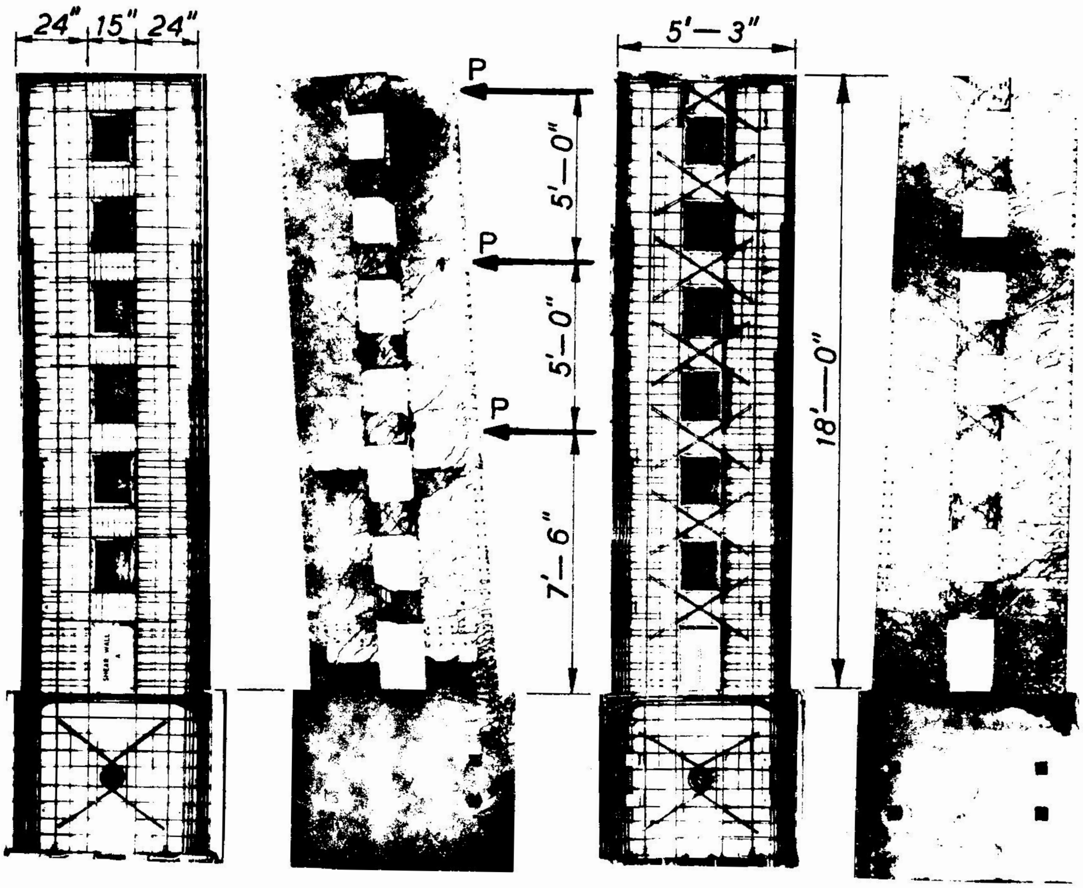


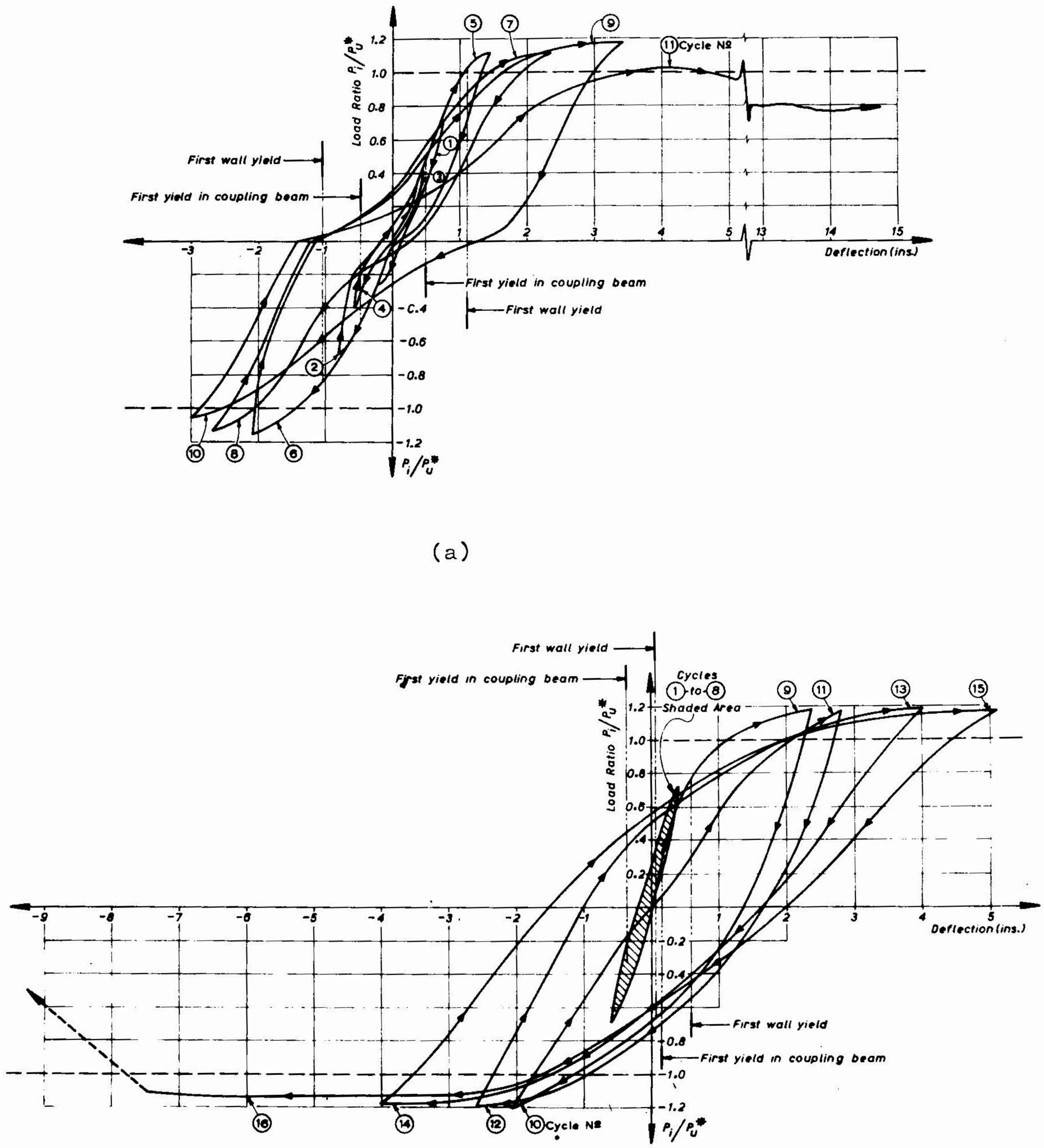
FIG. 18 - CUMULATIVE DUCTILITIES IMPOSED ON CONVENTIONALLY AND DIAGONALLY REINFORCED COUPLING BEAMS (35)



Shear Wall A

Shear Wall B

FIG. 19 - THE REINFORCEMENT IN AND THE CRACK PATTERN OF TWO QUARTER FULL SIZE SHEAR WALL MODELS SUBJECTED TO REVERSED CYCLIC LOADING (36)



(a)

(b)

FIG. 20 - THE LOAD-DISPLACEMENT RELATIONSHIP FOR A SEVEN STOREY SHEAR WALL MODEL WITH (a) CONVENTIONALLY, (b) DIAGONALLY REINFORCED COUPLING BEAMS (36)