# FRAME-SHEAR WALL ASSEMBLIES SUBJECTED TO SIMULATED SEISMIC LOADING

by

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## SYNOPSIS

The elasto-plastic response of two simple reinforced concrete coupled frame-shear wall assemblies, subjected to reversed cyclic static loading, was studied and the highlights of the experimental results are briefly reported here. In preliminary tests the behaviour of the beam-wall junctions, where large ductilities were imposed, was studied in particular to establish the influence of the shear force on stiffness degradation and energy dissipation. Two different arrangements of the flexural reinforcement were used in these preliminary specimens. In spite of the very severe displacement pattern imposed on the frame-shear wall assembly models, very satisfactory response was obtained when the adverse effect of the reversed cyclic shear on the concrete of the plastic hinge zones of the beams was eliminated.

## INTRODUCTION

The aims of this study were to examine energy dissipating characteristics, sources of stiffness degradation, ductility demands on component members of a simplified frame-shear wall assembly and its eventual failure mechanisms. The data obtained is being used to improve existing mathematical models which can then predict more realistically the postelastic dynamic response of such structures to a given ground motion.

The principal dimensions of one of the one quarter scale seven storey reinforced concrete frame-shear wall models can be seen in Fig. 4. As expected the overall deformations during reversed cyclic loading were dominated by the deformations of the shear wall. The column-beam joints, which were expected to be likely sources of weakness, were reinforced in a special way to ensure that energy dissipation at large displacements would occur in other areas of the structure. The reinforcement pattern used in the joints satisfactorily achieved this aim. However, the arrangement was considered impractical for full size construction and for this reason is not discussed here.

Apart from the base of the shear wall the plastic hinges in the beams were expected to be the major energy absorbing devices. To study their behaviour four isolated beam-wall junction specimens were constructed and tested before the frame-shear wall models were made. The behaviour of the anchorage zones of the beam bars in the wall and the response of the plastic hinges, as a consequence of shear load, was of particular interest. The more important features of the behaviour of these beams are reported in the next section.

The loading used in the frame-shear wall models simulated the equivalent lateral static loading commonly prescribed by the seismic provisions of building codes. This load pattern could be achieved by applying

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approximately equal point loads at the 7th, 5th and 3rd floor levels of the structure.

## ISOLATED BEAM - WALL JUNCTION SPECIMENS

The dimensions of the beam specimens, which were similar to those used in the major tests, and the reinforcing details are shown in Figs. 1a and 2a. These cantilever beams represented one half of the prototype beams, extending from the wall to the point of contraflexure. The prototype beams had a clear span to effective depth ratio of approximately 8. In the conventionally reinforced beam (Fig. 1a) ample stirrup reinforcement was provided in the plastic hinge area to resist the shear generated at the development of the maximum expected flexural strength. The stirrups were thus not expected to yield.

The cylinder strength of the concrete at the time of testing was of the order of 7650 psi (52.5 MPa). The steel used for the flexural steel had an observed mean yield strength of 43.8 Ksi (302 MPa) and the stress-strain curve exhibited a well defined yield plateau up to a strain of approximately 10 times the strain at yield.

In the second type of beam specimen the flexural steel was carried through the potential plastic hinge area at an angle that enabled a resolution of the applied force at the end of the cantilever into two sloping components that coincided with the line of action of the flexural steel. (See Fig. 2a). It was expected that, if and when the need arises, the external load, i.e. the ensuing moment and shear, would be resisted in the plastic hinge zone entirely by reinforcement without the assistance of the concrete. Therefore any damage that the concrete would suffer during the cyclic reversed loading should have negligible effect on the response of these beams once the flexural top and bottom bars have yielded.

As the load-deflection relationships, presented in Figs. 1b and 2b, indicate, after two half load cycles in each direction below yield level, increasing tip deflections were applied to both types of specimens. At the end of the loading sequence these deflections corresponded with imposed displacement ductilities of ±12 to ±16.

Both types of beams satisfactorily developed flexural capacities that were in excess of the theoretical value $P_u$ because of strain hardening and strain ageing, which was significant in the type of steel used. To enable numerous displacement and strain measurements to be made, occasionally several days passed between the application of a load cycle and the following one. The testing of the seven storey models extended over a number of weeks. Over several days the increase of the yield strength of the bars, that had been subjected previously to yielding, was of the order of 8%.

The major difference in the response of the two types of beams to similar cyclic loading sequence was in the stiffness degradation and energy dissipating capacities. In the plastic hinge zone the residual plastic strains in the top and bottom flexural reinforcement accumulate during reversed cyclic loading. The observed lengthening of the beams with successively imposed loading beyond yield verified this. As a consequence the widths of the cracks, that extended over the full depth of the section in the plastic hinge of the beam, increased so that the necessary shear transfer by aggregate interlock and dowel action of the horizontal flexural steel resulted in ever increasing shear displacements. The phenomenon was reported previously [1] and it was also observed in near full size specimens [2]. For example, an examination of the load-displacement relationship in the 13th half cycle (Fig. 1b) revealed that at low loads 85% of the total tip deflection was due to shear sliding. The contribution of shear deformations in the plastic hinge zone are relatively smaller near full load but they are still very significant. It is to be noted that the nominal shear stress at the development of full capacity in these beams was quite moderate, i.e. $v_u$ = 190 psi (1.3 MPa).

Similar beam elongations due to accumulated plastic strains in the flexural steel were observed in the second type of beams. However, because of the full shear resistance of the sloping flexural bars in these beams, as indicated in Fig. 2a, the widening of the full depth cracks did not affect the response of the beam. This can be seen in Fig. 2b. Another specimen with diagonally arranged flexural reinforcement was subjected to reversed cyclic load corresponding with displacement ductilities of ± 10. A very stable response, similar to that of a rolled steel member, with a well defined Bauschinger effect, was obtained. These tests showed that the undesirable features of degrading stiffness, loss of energy dissipating capacity and eventual sliding shear failure in the plastic hinge zones of beams can be avoided.

It should be noted that such beams can always be satisfactorily anchored to shear walls, a necessary condition for good behaviour that is much more difficult to achieve in the joint region of frames. The variation of the flexural steel strains were also followed in the anchorage zones of the flexural bars. This enabled the associated beam deflections to be evaluated. Surprisingly it was found that, irrespective of the magnitude of the imposed beam deflections, anchorage deformations within the shear wall consistently contributed to 27 - 34% of the beam tip deflections.

Fig. 3a shows a typical plastic hinge zone with seriously damaged concrete. The sliding displacements and the corresponding dowel deformation of the flexural bars are evident. The photograph shows convincingly that this type of failure cannot be prevented by additional stirrup reinforcement. The stirrups often determine the plane of significant sliding because they act as initiators of flexural cracks. In contrast Fig. 3b shows that after similar imposed ductilities the concrete in the diagonally reinforced plastic hinge zone was in much better condition, and that flexural curvature dominated the distortions in the plastic hinge zones.

## FRAME-SHEAR WALL MODELS

An attempt was made to model realistically prototype conditions in two reinforced concrete specimens that were tested in a horizontal position. These specimens did not relate to a specific full size structure. To simulate the gravity load from tributary floor areas, both the shear wall and the column were suitably prestressed with an adjustable ungrouted steel cable. Floor slabs, that would normally frame into the members at each floor, were omitted, as seen in Fig. 4, but provisions were made at every second floor to ensure that no buckling of the 4 in (102 mm) thin walls or columns would occur.

In the first specimen conventionally reinforced beams, such as shown in Fig. 1a, were used. A sliding shear failure in all the plastic hinges near the walls, similar to those seen in Fig. 3a, was quite evident at the end of the test. The other specimen, with diagonally reinforced beams, behaved very satisfactorily. This is reflected by the load-top floor displacement relationship, presented in Fig. 5. This shows stability of behaviour, good energy dissipating properties and no significant deterioration of stiffness. The eventual failure of the shear wall indicated that careful attention must be paid to stabilising the principal wall reinforcement at the extremities of the wall section if buckling of the plastified compression zone, individual bars or groups of bars in cages is to be prevented. Further details of this study will be published elsewhere [3].

## SUMMARY

The testing under simulated seismic conditions of one quarter full size reinforced concrete frame-shear wall assemblies demonstrated that, with suitable detailing of the reinforcement, seismic structural properties can be attained that are comparable with those of ductile steel structures.

When during cyclic reversed loading large ductilities are imposed on the plastic hinges of conventionally reinforced concrete beams, a failure by sliding is a possibility. Shear displacements in these areas can significantly contribute to stiffness degradation and hence to loss of energy dissipating capacity in the structure, particularly at low loads.

If the contribution of the concrete to the shear strength of a plastic hinge area is replaced by mechanisms that rely on the statically admissible resistance of the reinforcement only, then a plastic hinge with very large ductilities, even under severe cyclic loading, can be obtained. An example of such a solution was shown in the reported tests where the flexural reinforcement traversed the plastic hinge zone at exactly the required slope. Particular attention must be paid, however, to the splice that follows immediately the plastic hinge in such beams, as seen in Fig. 2a, to ensure that yield penetration into this area does not destroy the required bond transfer.

## ACKNOWLEDGEMENTS

The experiments were carried out in the laboratories of the Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand. The dedicated contribution of the technicians of the department and the financial assistance provided by the New Zealand University Grants Committee is gratefully acknowledge.

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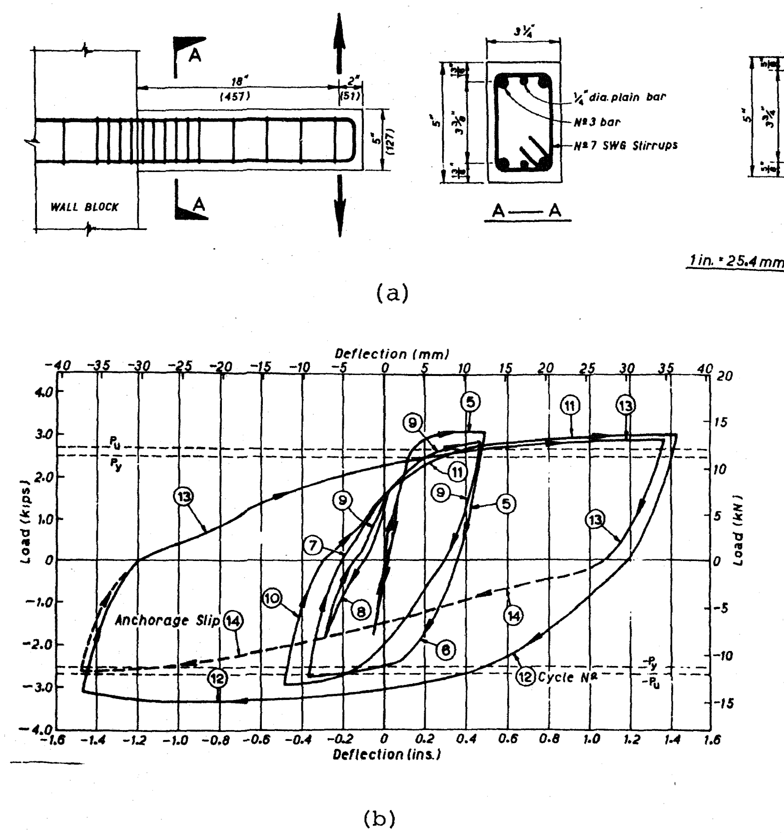



Fig. 1 Details of a Conventionally Reinforced Beam-Wall Junction Specimen.
(a) Dimensions and Reinforcement.
(b) Load-Deflection Relationship.

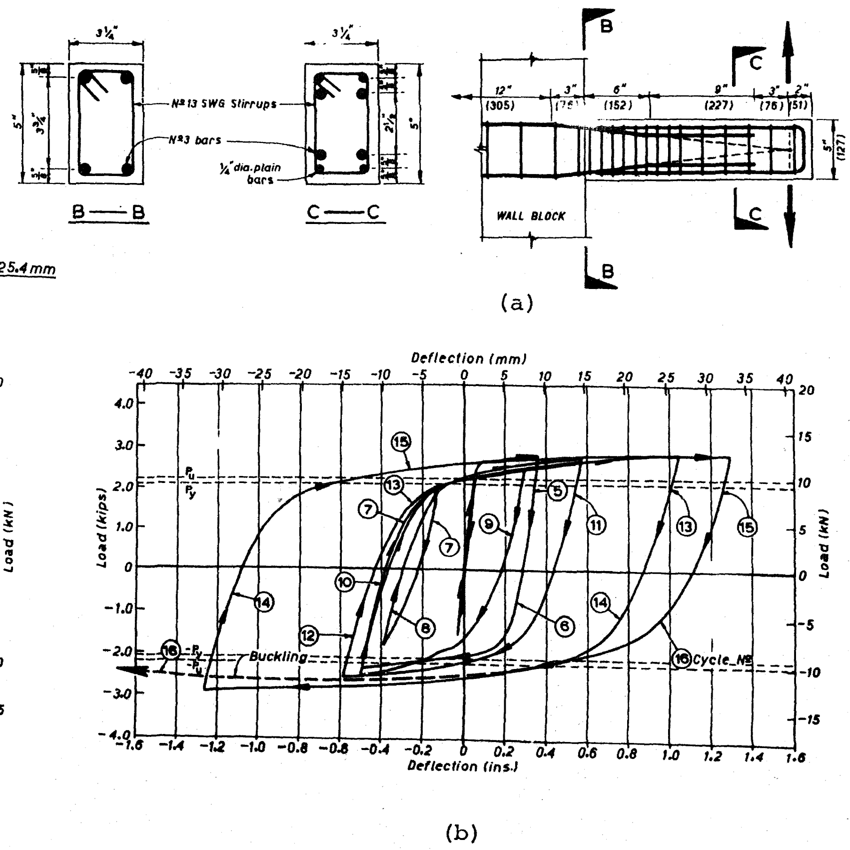
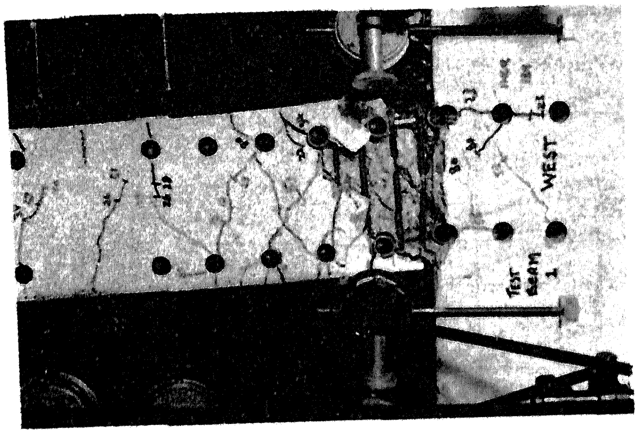



Fig. 2 Details of a Diagonally Reinforced Beam-Wall Junction Specimen.
(a) Dimensions and Reinforcement.
(b) Load-Deflection Relationship.




(a)

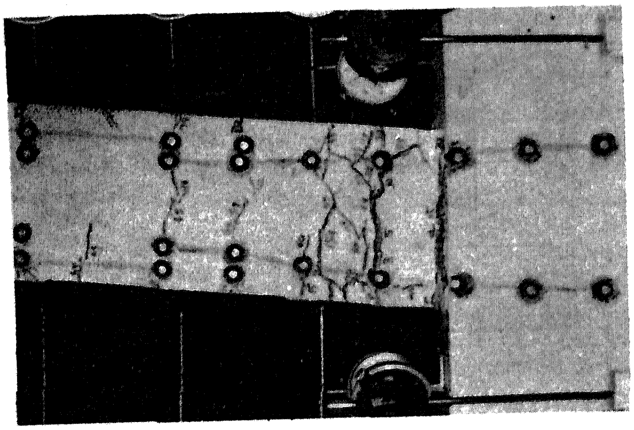


Fig. 3 The Plastic Hinge Zones of Beam-Wall Junction Specimens.

(a) Conventionally Reinforced Beam.
(b) Diagonally Reinforced Beam.

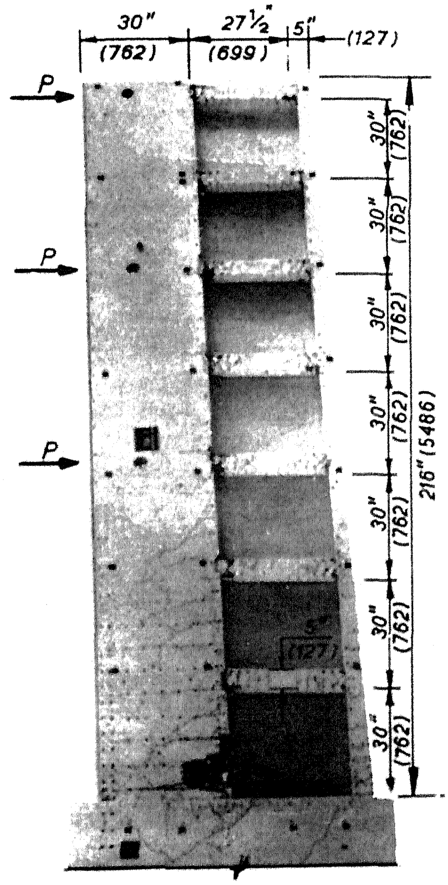



Fig. 4 One Quarter Full Size Reinforced Concrete Frame-Shear Wall Assembly After Testing.

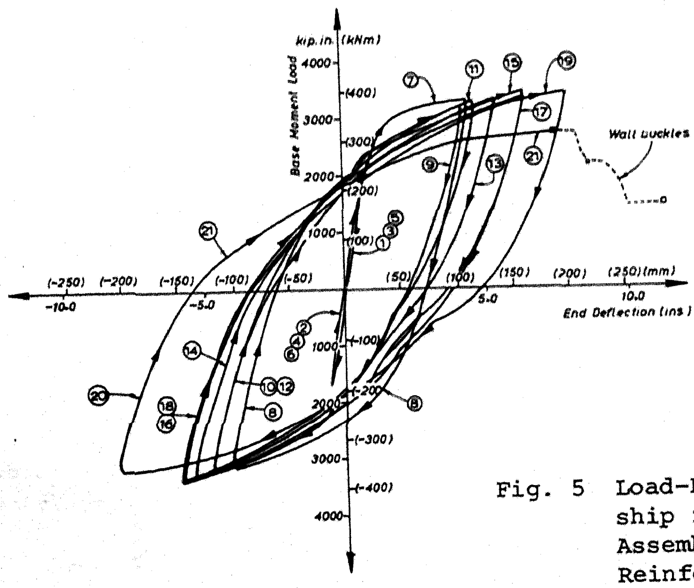



Fig. 5 Load-Roof Deflection Relationship for a Frame-Shear Wall Assembly with Diagonally Reinforced Beams.