# CYCLIC LOAD TESTS ON SHEAR WALL PANELS

by

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## SYNOPSIS

This paper presents the results of a series of cyclic lateral load tests conducted on one-half scale reinforced concrete shear wall panels. Parameters investigated include the effect of vertical load and the effect of varying the panel aspect ratio. Results are given in terms of load-displacement hysteresis curves and stiffness degradation. The energy absorption characteristics and panel ductility factor capacities are also discussed.

## INTRODUCTION

A number of investigations into the performance of reinforced concrete shear walls subjected to seismic loading have been conducted previously to determine the suitability of this type of construction in seismically active areas. This particular investigation is concerned with the experimental determination of the in-plane behaviour of low aspect ratio (height/width) reinforced concrete shear wall panels subjected to cyclic lateral loads. The specimen size (a typical specimen is shown in Figure 3) represents a one-half scale "model" of a single storey shear wall panel which could be a complete wall or an inter-floor element in a multi-storey wall. The panels were subjected to cyclically reversing horizontal loads to failure. The parameters of interest are the shape of the load-deflection curves, stiffness degradation, cracking patterns, energy absorption, and ductility capacity.

## TEST PROCEDURE

The models tested had a constant height of 54 ins., a thickness of 4 ins., and lengths of 9 ft., 6 ft. and 3 ft. These were tested under various conditions of vertical loading and reinforcing steel arrangement. The horizontal loading programme, applied in the controlled displacement mode, consisted of several cycles at low displacements until cracking was observed, a cycle to produce yielding, a number of cycles at below-yield displacements to investigate energy absorption and stiffness stabilization, a cycle to 1.5 times the yield displacement, a number of cycles at a lower displacement and then several cycles at increasing displacement until failure occurred or as near to failure as the test apparatus would allow. The horizontal loads were transmitted to the panel through heavy reinforcing bars located in the top flange beam. These bars were wrapped in a staggered manner in order to produce a reasonably uniform transfer of load along the length of the panel. The reactions were transmitted

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from the panel to the test floor through a base flange beam with bolts torqued to the test floor.

The basic reinforcement pattern common to all the models consisted of a single layer of deformed 1/8 in., diameter bars (yield stress = 52000 p.s.i.) spaced at 4-1/2 in., centres in both directions. The resulting steel ratio of 0.003 simulates minimum code requirements for steel to control temperature and shrinkage cracks. Large amounts of flexural steel at the panel edges were not used because the primary interest was in the capacity of a single panel in shear. Minor variations in the reinforcement arrangement are noted in the discussion of the individual panel test results.

## RESULTS

Panel 1 (length of 9 ft.) was not subjected to any vertical load and failed along a flexural crack at the panel-base beam interface with little propagation of useful cracking into the panel itself. Panel 2 was basically the same except for modifications to include a single vertical bar at the panel edges and starter bars from the base beam into the panel. This led to improved ductility and energy absorption due to an improved cracking pattern, thereby justifying current construction practice. The additional edge reinforcing reduced the tendency for the interface flexural crack to widen without internal cracking and thereby produced inclined cracks within the panel. Inclined cracks in the panel propagated primarily from the base to the edges of the panel, and occasionally at higher magnitudes of lateral displacement from the edge of the panel. No perceptible trends were observed. At large displacements, a rotational block and wedge action along the inclined cracks produced crushing and further cracking of the concrete in the lower third of the wall. The starter bars tended to lift the level of the initial cracks above the base beam interface, but subsequent inclined cracking propagated through to this interface.

Panel 3 was the same as Panel 2 but was subjected to a constant vertical load (producing an axial compression stress of 185 p.s.i.) during the lateral loading programme. This panel sustained a maximum horizontal load of 157 Kips, 26 percent higher than the maximum for Panel 2. The vertical load produced a more uniform distribution of cracking in the panel, with the final inclined cracks propagating right up to the top flange beam interface. The maximum crack height reached in Panel 2 was 3/4 of the panel height. This improved crack distribution decreased the stiffness degradation, compared with Panels 1 and 2 which had basically the same stiffness degradation, as shown in Figure 1. The energy absorption was 70 percent higher than that in Panel 2 but, as would be expected, reduced ductilities (range 2 to 4) occurred because of the axial load.

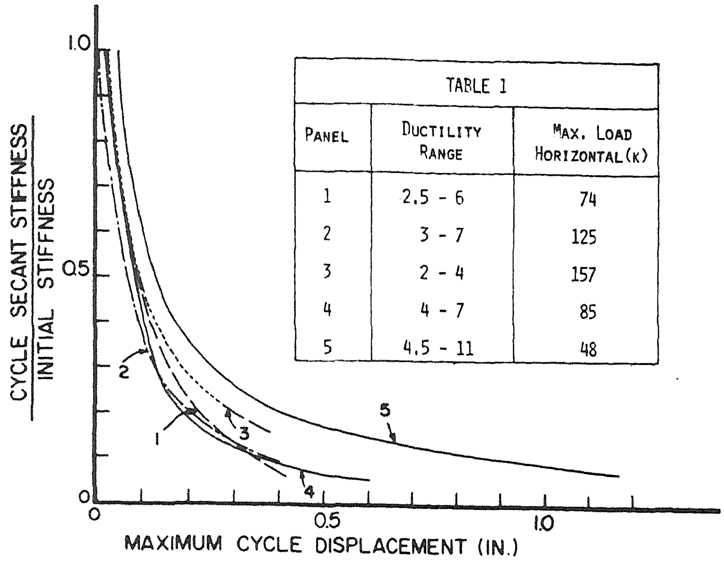
Panels 4 and 5 were 6 ft. and 3 ft. long respectively, but contained the same percentages of reinforced steel as Panels 2 and 3. Vertical loads were applied to both panels, producing compression stresses of 139 and 278 p.s.i. respectively. There were no fundamental changes in the cracking pattern. The lower maximum load capacities allowed failure to occur at progressively larger ductilities as the length of the panel decreased (see tabulated results in Figure 1). A portion of the load-deflection curves for Panel 4 are shown in Figure 2.

The effect of differing amounts of axial stress due to vertical load significantly affects the stiffness degradation characteristics, as shown in Figure 1. Because the degradation curves are presented in terms of the ratio to the initial stiffness, the effect of different panel sizes has been removed, except for differing maximum displacements. Consequently, it can be seen that increased axial stresses causes less stiffness degradation.

In all panels, failure occurred by opening of a major crack and the yielding of the steel bars across that crack. This major crack was usually one of those which formed during the first cycle to yield displacement, and extended across the panel approximately in a crescent shape. In the later stages of loading, preferential action begins on the major crack with only small expansions on the other inclined cracks. The effect of the major crack expansion can be seen in the hysteresis loops. The last cycles approaching the ultimate load show a trend for the stiffnesses to progressively decrease as yielding of the bars across the crack increased. On the return from maximum displacement, the gradient of the curve becomes parallel to that of previous cycles after the portion of the major crack on the loaded side of the panel had closed. Repetition of sub-peak level displacement cycles indicated stabilization of lateral panel displacements and energy absorption.

## CONCLUSIONS

Compressive axial stress reduces panel ductility capacities but leads to improved stiffness degradation, apparently independent of panel lengths. Energy absorption improves when inclined cracks are widely dispersed over the panel face; vertical loads assist in this crack dispersion. Aspect ratios also affect the panel ductility capacities, with the shorter panels having higher capacities.




| TABLE 1 | | |
|---|---|---|
| PANEL | DUCTILITY RANGE | MAX. LOAD HORIZONTAL (K) |
| 1 | 2.5 - 6 | 74 |
| 2 | 3 - 7 | 125 |
| 3 | 2 - 4 | 157 |
| 4 | 4 - 7 | 85 |
| 5 | 4.5 - 11 | 48 |

FIG. 1 STIFFNESS DEGRADATION CURVES

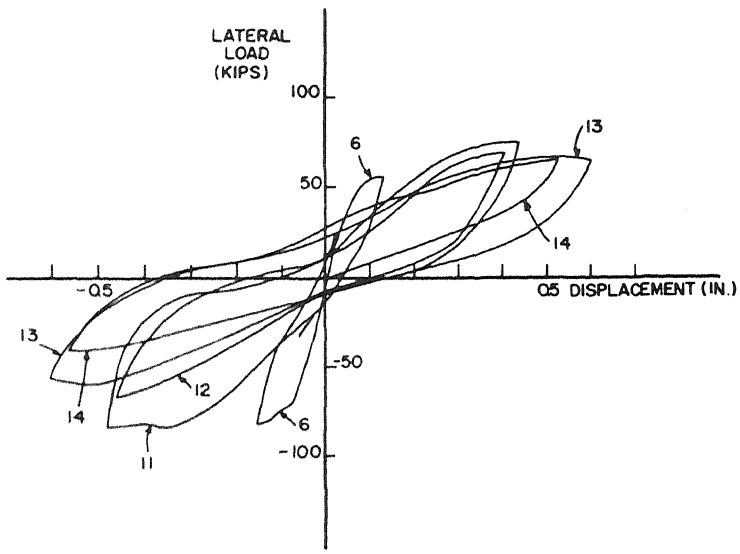



FIG. 2 SELECTED CYCLES OF HYSTERESIS LOOP, PANEL 4

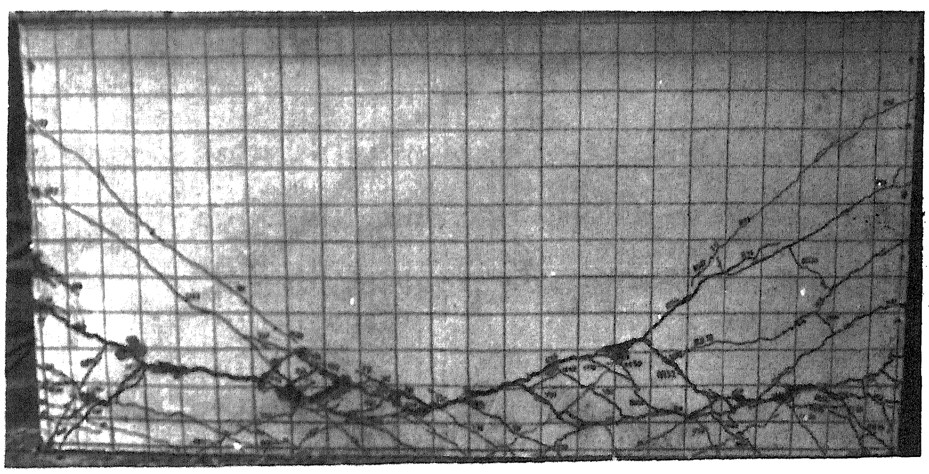


FIG. 3 PANEL 2 AT FAILURE