# COMPARISON OF THE BEHAVIOUR OF LOAD BEARING SHEAR WALL PANELS SUBJECTED TO REPEATED CYCLIC LATERAL LOADING

by

Derek J. DeLisle (I) and A.C. Heidebrecht (II)

## SYNOPSIS

This paper describes an experimental investigation into the behaviour of reinforced concrete shear wall panels subjected to cyclic lateral loading. In particular, the purpose is to determine whether the slitted wall concept, which the Japanese have originally used for in filled wall panels, is applicable to the load bearing situation occurring in shear wall structures. The effects of vertical load and slit length are included in the investigations. Results are presented in terms of stiffness deterioration and energy dissipation. It appears that the slitted wall is not significantly better than the ordinary reinforced concrete wall, although the presence of vertical loads, applied to the slitted wall, does reduce the stiffness deterioration and increases the energy dissipation.

## INTRODUCTION

In recent years, a large amount of design-oriented earthquake engineering research has been conducted in Japan, resulting in the removal of the building height restriction in Tokyo, which is an extremely active seismic area. The first few high rise buildings constructed in Tokyo have been of a composite form consisting of a steel framework and reinforced concrete infilled walls (1) (III). The displacement incompatibility between the flexible steel framework and the relatively rigid infilled wall panels is overcome by use of a new concept, the slitted wall panel (1,2). The slitted wall panel is a reinforced concrete panel containing a series of vertical slits, normally filled with asbestos sheeting, and forming a complete discontinuity in the wall structure. Experimental tests of such walls, subjected to reversible shear forces, concluded that the inadequacies of the normal reinforced concrete wall panel were overcome by the reduced stiffness and improved ductility of the slitted wall panel. Although the high rise buildings in Tokyo which incorporate this concept have been subjected to small earthquake tremors, they have not yet been subjected to any large motions which could test the actual validity of the slitted wall panel concept, particularly with regard to the postulated ductility characteristics.

The purpose of the experimental investigation described in this paper is to determine whether the slitted wall concept is applicable to shear wall structures, in which each panel is subjected to both vertical load and cyclic lateral load during an earthquake.

There currently exists some doubt as to the capability of reinforced

I Graduate Student, Department of Civil Engineering and Engineering Mechanics, McMaster University, Hamilton, Ontario.

II Associate Professor, Department of Civil Engineering and Engineering Mechanics, McMaster University, Hamilton, Ontario.

III Arabic numerals appearing in brackets in the text refer to references listed in the Bibliography.

concrete shear wall structures to resist severe earthquakes. This doubt is expressed in the earthquake load provisions of the 1970 National Building Code of Canada (3) in two ways:

a) the seismic force factor K for shear wall buildings is specified to be 1.33, compared with 0.67 for ductile moment resisting space frames, and

b) shear wall buildings are restricted to a height of 200 ft. in Zones 1, 2, and 3, although can exceed this restriction only in Zone 1 if "the walls are designed with special provisions required for their ductile behaviour".

Similar provisions are specified in the 1966 Code of the Structural Engineers Association of California (SEAOC), although the height restriction is set at 160 ft. (4). The reasons for these doubts are due to the damage suffered by shear wall structures subjected to major earthquakes (5).

The authorities responsible for the code provisions have not yet had sufficient experimental evidence of the behaviour of shear walls in the post-elastic range to permit a more liberal approach to the design of shear wall buildings, whether in terms of a reduced seismic force factor or the elimination of height restrictions. It should be noted that the National Building Code of Canada permits departure from the height restriction if the authority having jurisdiction is provided with evidence that the structure can withstand the appropriate design earthquake with ductility and energy absorptive capacity equivalent to that in a structure with at least 25 per cent of its seismic resistance provided by a ductile moment resisting frame.

Consequently, the particular objective of this investigation was to compare the behaviour of slitted wall panel with that of a normal reinforced concrete panel when subjected to reversed loading resulting in large inelastic deformations.

## EXPERIMENTAL PROGRAMME

Four wall panels were constructed and tested under repeated cyclic lateral loads. Three of the panels contained vertical slits while the fourth panel was an ordinary reinforced concrete wall. In addition to the direct comparison between slitted and unslitted walls, the programme included the effect of applying vertical loads to the wall and the effect of increasing the slit length to the full panel height. Comparisons were made by considering the stiffness and energy properties of each panel. Descriptions of the wall panels, the loading frame and the test procedure follow:

### Wall Panels

A typical panel is shown in Figure 1. The panel as constructed for this investigation is a half-scale version of the lowest storey of an idealized ten storey building with coupled shear walls spaced at twenty foot intervals. For purposes of this investigation, in order to concentrate on the panel behaviour under lateral load, the effects of the coupling beams were neglected. The behaviour of coupling beam has been studied by Paulay and the results of his investigation have recently appeared in the literature (6).

An analysis of the above building subjected to the normal triangular loading pattern showed that only nominal reinforcement was required in the walls. The reinforcing used in the walls was based on the recommended minimum of 0.25 per cent of the gross cross-sectional area of the wall (7,8). The arrangement of the reinforcing was considered in conjunction with the spacing of the vertical slits. A straight forward bar arrangement coincided with a slit spacing of 1.5 ft., as shown in Figure 1 for the standard slitted wall (Panels A and C in the test program). Panel B, the ordinary reinforced concrete wall, had a reinforcing mat with bars spaced at 4 1/2" in both directions. Panel D had vertical slits with length equal to the panel height and had a similar reinforcing arrangement to panels A and C.

The reinforcing bars used in all panels were 1/4" diameter smooth bars having an average yield strength of 44,500 psi. A standard commercial ready mix concrete was used in the manufacture of all panels, reaching an average compressive strength of 3760 psi at the time of the panel tests. The slits in the slitted walls contained 1/2" thick asbestos boards.

## Loading Frame

The special loading frame in which the panels were tested is shown in Figure 2. This frame was designed to incorporate the following requirements:

a) the frame should be capable of testing walls of other materials and dimensions than those used in this particular programme.

b) the frame should permit reversible lateral loads to be applied to the wall panels, and

c) the frame should be able to apply constant vertical loads if desired.

The lateral load capability was supplied by a horizontal actuator operating from a servo-controlled electro-hydraulic loading system, with a thrust capacity of 250 kips in either direction. The vertical loading system, as shown in Figure 3, used a technique for maintaining constant loads which is commonly used in applying such loads in creep testing of reinforced concrete specimens. Such a technique uses four stiff springs at each load point to prevent large variations in load during the deformation of the wall panel. Horizontal movement of the vertical loading system was prevented by a bracing system.

The lateral load was introduced into the wall panel through a loading yoke. The loading yoke was connected to the flange beam at the top of the panel by a series of threaded rods cast into the concrete flange beam. This method of connection was used to simulate the distributed manner in which the inertial forces, produced in a floor mass during dynamic motion, would be introduced into the adjacent wall panel.

## Test Procedure

The servo-controlled loading system permitted the use of displacement control for the lateral load. Displacement control is advantageous in that it enables the determination of behaviour near the point of failure. Failure of a wall panel is considered to be that point after which further displacement results in a reduction of lateral load.

Each of the four panels was subjected to approximately the same sequential pattern of imposed lateral displacements. This pattern consisted of 13 cycles. These cycles consisted of four sets of equi-displacement cycles, with each set having a larger peak displacement than the previous set; these sets were spread throughout the range of lateral displacement of the wall. Each set consisted of three cycles conducted at the same peak displacement, so as to enable the determination of the effect of constant displacement cycling on the wall properties. The first set was used to study the behaviour of the wall before cracking. The final cycle of the pattern was at the maximum displacement the loading frame permitted.

Measurements of load and displacements were made at approximately thirty points during each cycle. Fluctuations in the readings were observed during the data recording period, which necessitated the recording of initial and final readings during each period. The average of the initial and final readings were used in all calculations based on the results

Panels C and D were each subjected to two constant vertical point loads applied throughout the lateral loading sequence. These loads were used to simulate the compressive stress condition in the lowest storey wall panel due to wall and floor loadings (dead load plus one-third live load) in the storeys above. Each point load had a magnitude of 75 kips; theses were monitored throughout the test and the maximum variations were 1 kip. The lateral friction forces on the panel, produced by the roller bars in the vertical loading system, were evaluated by a short series of tests on Panels C and D, the calculated value was 0.3 kips per load point in each case.

## DISCUSSION OF RESULTS

### Stiffness

The load deflection diagrams for panels A, B and C are presented in Figures 4, 5 and 6. Only the first cycle of each set of three is shown since the additional cycles contribute very little to the overall picture. The slopes of the load-deflection hysteresis loops indicate the stiffnesses of the panel at the various stages of the test.

During the last cycles of each test, when the wall panel had deformed extensively into the cracked region, the load-deflection curve is made up of three distinguishable zones:

a) Initial Zone: The stiffness in this zone is low. Lateral load is required to close the cracks developed during the reverse loading part of the cycle. Until the cracks are closed, the compression forces are transferred only by the reinforcing steel across the cracks.

b) Middle Zone: The cracks have now closed and the slope of the load-deflection curve is increasing. The compression forces are being transferred across the crack by the bearing of the two concrete surfaces.

c) Final Zone: Further cracking occurs and the yielding of the reinforcing steel causes a decrease in the slope of the load-deflection curve.

In the tests, the development of the first two zones described above depended upon the width of crack which had formed during the preceding reverse loading part of the cycle. Tests of panels A and B, which were subjected to lateral load only, showed relatively large crack widths so that all three zones are clearly identifiable, as indicated in Figures 4 and 5. The crack widths in panels C and D, which were also subjected to the vertical loads, were smaller and the three zones are more difficult to distinguish, as can be seen in Figure 6.

The stiffnesses of all four panels deteriorated significantly as the sequential load cycling progressed. The amount of deterioration was approximately 80 per cent of the initial stiffness in panels A and B and approximately 75 per cent in panels C and D. The stiffness degradation of the four panels is presented in Figure 7. The stiffnesses during the various cycles were calculated considering only the initial stiffness of the forward portion of the load-deflection curve. For those cycles in which a steepening of the slope of the load-deflection curve occurred due to the closing of cracks, an average of the two slopes was used to represent the panel stiffness during that cycle. No modifications were made if cracking caused a change of slope on the load-deflection diagram.

The initial stiffnesses of panel A (slitted wall) and panel B (reinforced concrete wall) were essentially identical. The elastic displacements were extremely small and small errors in the recorded data would have large effects on the calculated stiffness values. After cracking, the stiffness of the slitted wall deteriorated much more rapidly than that of the ordinary reinforced concrete wall; the final stiffness of the slitted wall was lower than that of the reinforced concrete wall.

The differences noted above for the two types of walls are due to a different cracking behaviour in each wall. The initial cracks in both panels A and B occurred at the interface between the wall and the lower flange beam. These cracks were due to flexural stresses. However, after this initial cracking, which initiated the stiffness degradation, the crack formation in the two panels differed considerably.

In panel A diagonal cracks formed between the ends of the slits and the adjacent flange beams. The vertical slits acted as lines of weakness and induced a particular crack pattern in the wall panel. The final crack pattern in panel A is shown in Figure 9. As indicated in this figure, a resistance mechanism formed during the later cycles of this test, consisting of flexural cracks along the wall-flange beam interfaces, diagonal cracks between the interfaces and the ends of the slits, and the slits themselves.

By comparison, the final crack pattern in panel B is shown in Figure 10. After the initial cracking in this panel, further cracking was restricted to a few diagonal cracks at each end of the wall and more cracking along the wall-flange beam interface.

The stiffness deterioration in the slitted wall was more rapid because the vertical slits offered little resistance to cracking. Relative movement along the slits was not restricted by transverse reinforcing or by the irregular surface features which would exist along a normal crack. Consequently the mechanism of panel A offered less resistance to lateral load.

In comparing the test results for panel C (slitted wall with vertical loads) with those of panel A, one can see that the stiffness developed in panel C is larger at all stages of the loading sequence. This increase is due to different cracking pattern which developed due to the presence of the vertical loads. In panel C, diagonal cracks formed due to the slight displacements of the vertical slits immediately below the bearing plates on which the vertical load was applied. These slight displacements were sufficient to induce diagonal cracks between the ends of the slits and the wall-flange beam interfaces. At the upper interface, most of the diagonal cracks propagated towards the points of application of the vertical loads. The final crack pattern for panel C is shown in Figure 11. The resistance mechanism which was so evident in panel A is not nearly so prominent in panel C. The larger stiffness in panel C is due to the more extensive cracking combined with a resulting smaller crack width.

The comparison of stiffness degradation between panels A and C is shown in Figure 8. The initial rate of stiffness deterioration after cracking appears to be unaffected by the presence of the vertical loads. However, the final amount of stiffness degradation is greater in panel A due to the formation of larger crack widths.

The effect of making the slit length the full height of the panel was to decrease both the initial and final stiffness. The vertical slits enabled the wall to undergo greater deformations without cracking since the asbestos material in the slits was far less rigid than the concrete. Consequently, most of the cracks were confined to the wall-flange beam interfaces and to the flange beams. During the final cycles of testing panel D, the lower interface was completely cracked and consequently the lateral load resistance was considerably lower than that of panel C.

Energy

A study of the energy properties of structural components requires the investigation of both the ability to absorb and to dissipate energy. The energy absorption capacity depends upon the yield level and the amount of inelastic deformation which can be sustained after yielding prior to failure (recall previous definition of failure). The ductility factor, the ratio of total displacement to displacement at first cracking or yielding, is a useful parameter for comparisons of energy absorption. The ductility factors for the four panels tested in this investigation are shown in Table 1. These values are considered adequate for most practical cases. Unfortunately, the values shown do not include the failure point due to a limitation imposed by the load frame. A slight modification of the loading frame will allow future wall panel tests to reach failure.

The energy dissipation ability of a structural component is a measure of how the structure will respond to seismic ground motion. If the energy dissipation is large, the vibrational amplitudes of the structural response will be small. Conversely, if the energy dissipation capacity is small, the vibrational amplitudes will build up as the absorbed energy increases, and may result in failure of the component if the "failure" ductility factor is reached.

The energy dissipated in the four wall panel tests, calculated from the area of each cycle's hysteresis loop, is shown in Figure 12. Three significant observations can be made from this figure:

a) the ordinary reinforced concrete wall dissipates more energy than the slitted wall,

b) the addition of vertical loads results in increased energy dissipation, and

c) the lengthening of the slits reduces the energy dissipation.

The explanations of the above observations are found in the energy dissipating processes. Three processes have been identified in this investigation, but the limited number of tests makes it impossible to describe these in other than a qualitative manner.

The primary energy dissipating process appears to be the actual formation of cracks. In each test, very little energy was dissipated prior to cracking. In panels C and D (which show better energy dissipation properties than panels A and B), the formation of cracks was distributed throughout the thirteen loading cycles. The cracking in panel C was extensive, whereas a large portion of the cracks in panel D formed in the flange beams. By comparison, crack formation essentially ceased after the seventh cycle in panels A and B. Consequently, both the length of cracks and the thickness of the cracked section appear to be important factors in the dissipation of energy by crack formation.

Even though crack formation ceased after the seventh cycle in panels A and B, the amount of energy dissipated increased with further inelastic displacement. Two additional dissipating processes were identified to account for this further dissipation.

One of these additional processes is the widening of cracks. The widening of cracks was evident in the tests of panels A and B, especially in the region of flexural cracking. This crack widening involved a breakdown of bond forces and an extension of the reinforcing steel; on reversal of load further energy was dissipated by the closure of the cracks.

The second additional process is the slippage and relative movement along the crack surface. Energy is dissipated in this process by overcoming the frictional forces between the aggregate particles along the crack surface, and by producing inelastic deformations in the reinforcing steel.

Further evidence of the premise of crack formation as the major energy dissipating process was given by observing the relative energy dissipation during the sets of three cycles at the same peak displacement. Such observations show that the energy dissipation was largest during the first cycle of the three. In that cycle most of the crack formation occurred, and the energy dissipation during the remaining two cycles was essentially limited to crack widening and crack slippage processes.

The ordinary reinforced concrete wall dissipated more energy than the slitted wall. The crack pattern in the slitted wall, panel A, included movement of the vertical slits. Very little energy was dissipated in this slit movement through the breakdown of adhesion forces between the concrete and the asbestos sheet. Also the movement along the slit was not restricted by transverse reinforcing or by irregular surfaces such as would occur along a normal crack face.

The lengthening of the vertical slits reduced the amount of energy dissipated. As indicated previously, cracking was less extensive in panel D than in panel C. Consequently the energy dissipated by cracking, which has been identified as a major process, was reduced by the lengthening of the slits. Also, little movement occurred along the cracks and the crack widths remained small. Consequently, the energy dissipation in panel D was less than in panel C.

## CONCLUSIONS

From this limited investigation, it appears that the slitted wall concept applied to shear wall panels does not produce significant improvements over ordinary reinforced concrete shear wall panels. For the reinforcing ratio of 0.25 per cent, the ordinary reinforced concrete wall had less stiffness deterioration, greater energy dissipation, and showed sufficient capability of sustaining inelastic deformations without failure. The method of applying the vertical loads to the slitted walls affected the formation of cracks. It is not known whether this particular loading process altered the stiffness and energy properties. The tests conducted with this particular vertical load arrangement showed that a significant increase in strength and energy dissipation was produced by the application of vertical loads. The stiffness deterioration was slightly reduced. The lengthening of the vertical slits reduced the stiffness and the amount of energy dissipated.

Further discussion of these tests is given in reference (9).

## ACKNOWLEDGEMENTS

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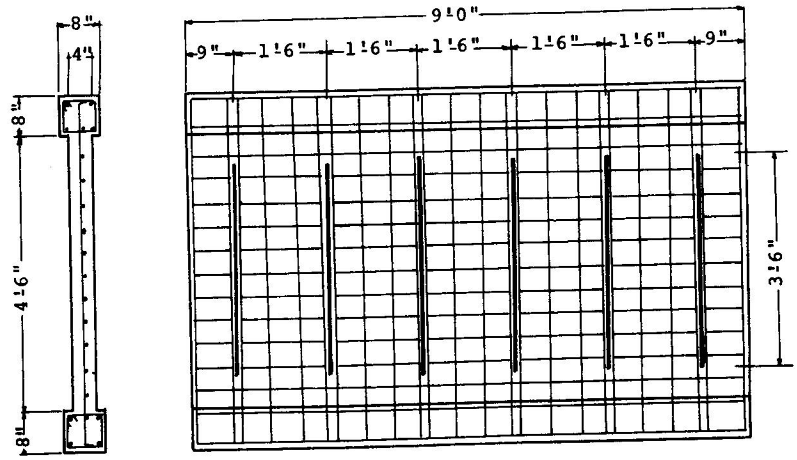


FIG. 1 PRINCIPAL DIMENSIONS OF PANELS
AND REINFORCING ARRANGEMENT FOR PANELS A AND C

| PANEL TYPE | DUCTILITY FACTORS |
|---|---|
| Panel A | 6.2 |
| Panel B | 5.1 |
| Panel C | 16.0 |
| Panel D | 18.3 |

TABLE 1

DUCTILITY FACTORS

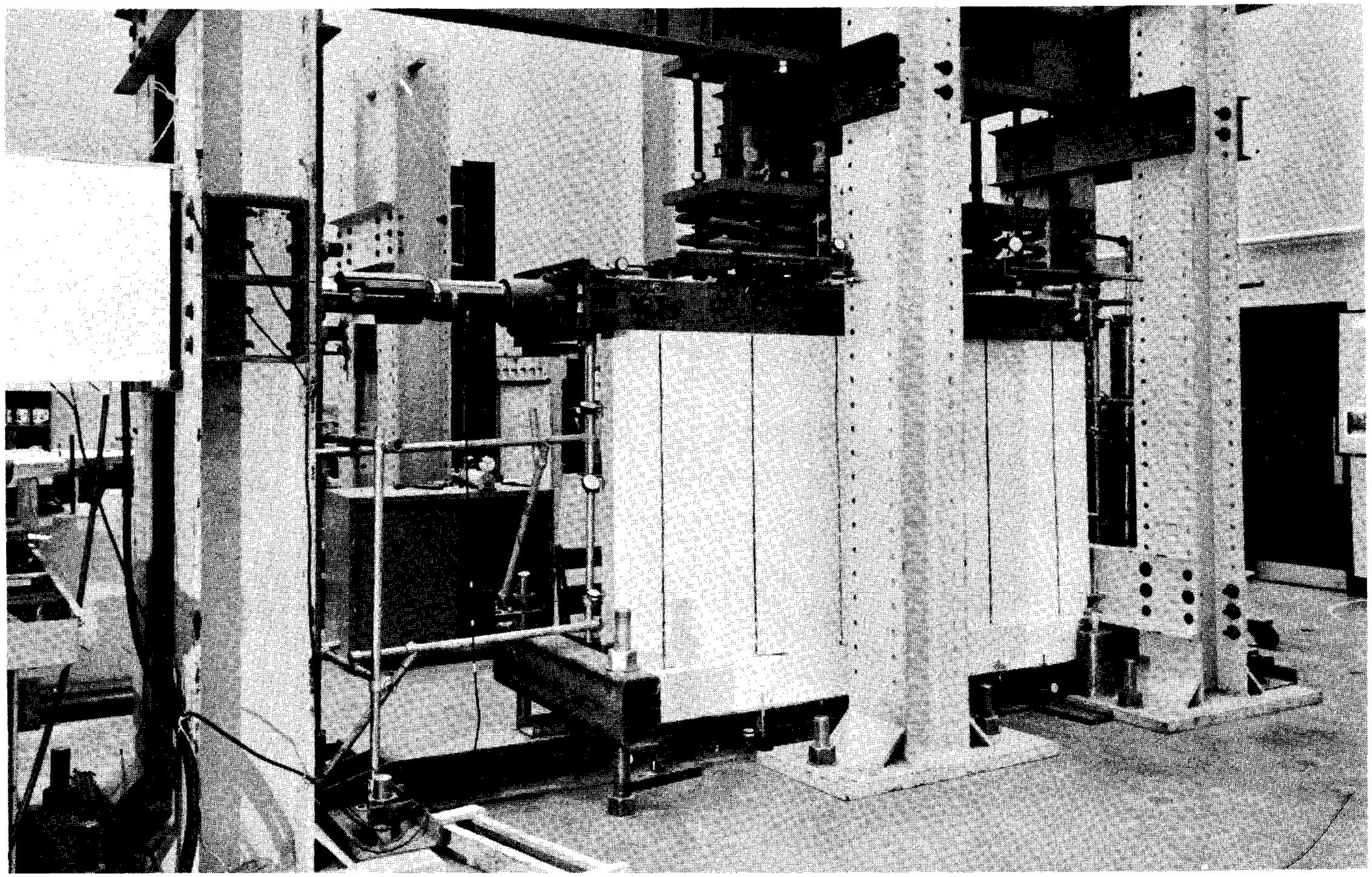

FIG. 2 PANEL D IN THE LOADING FRAME

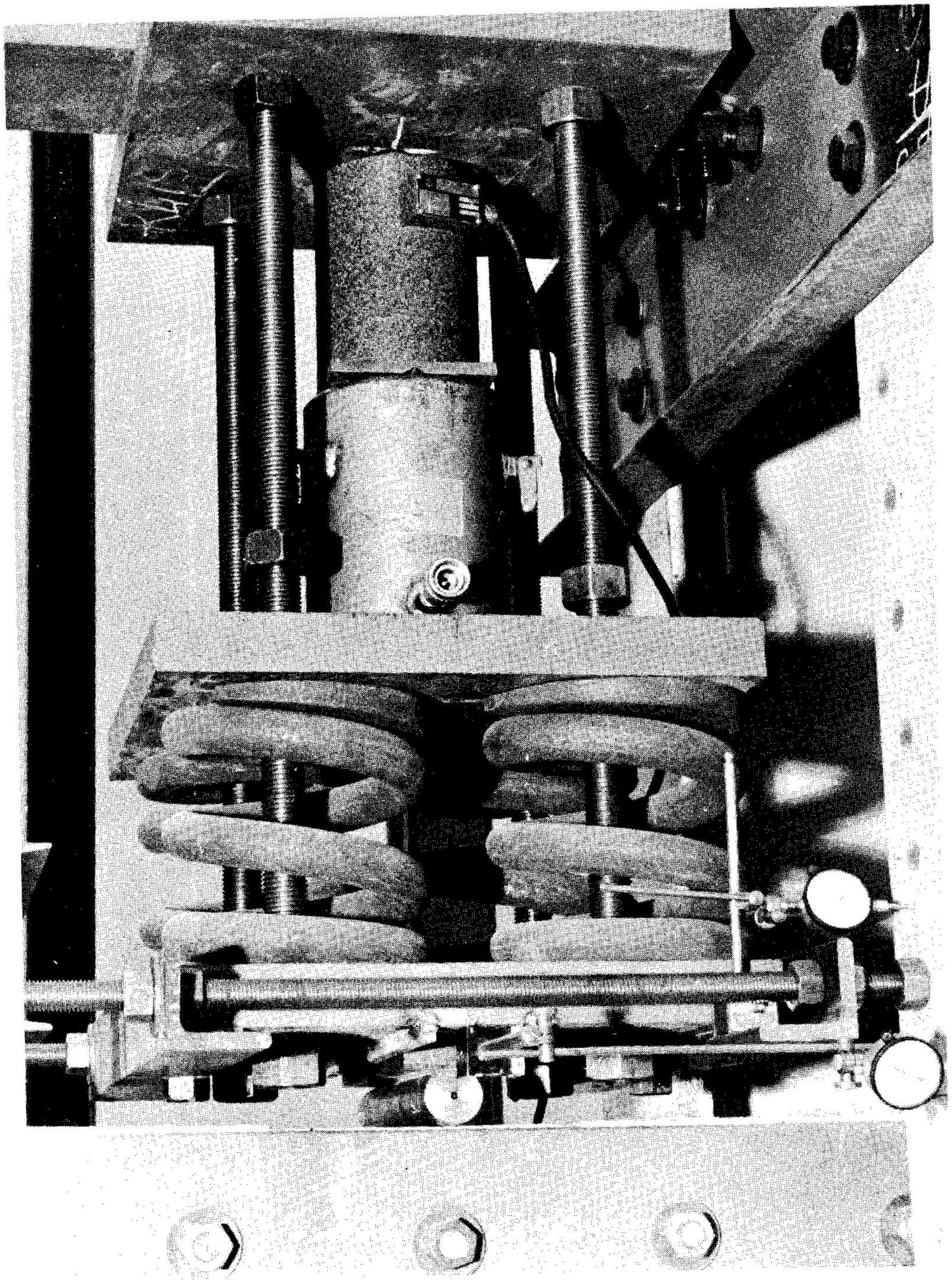

FIG. 3 VERTICAL LOADING SYSTEM

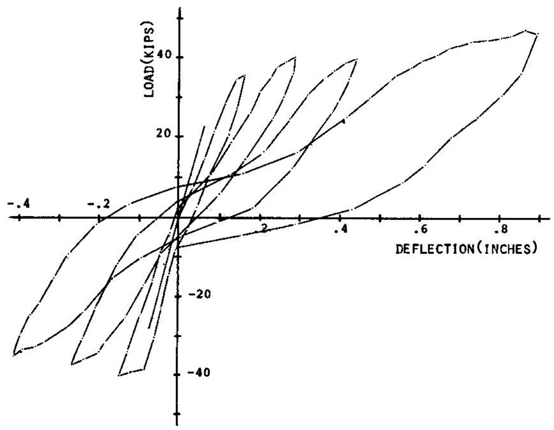


FIG. 4 THE LOAD-DEFLECTION RELATIONSHIP FOR PANEL A

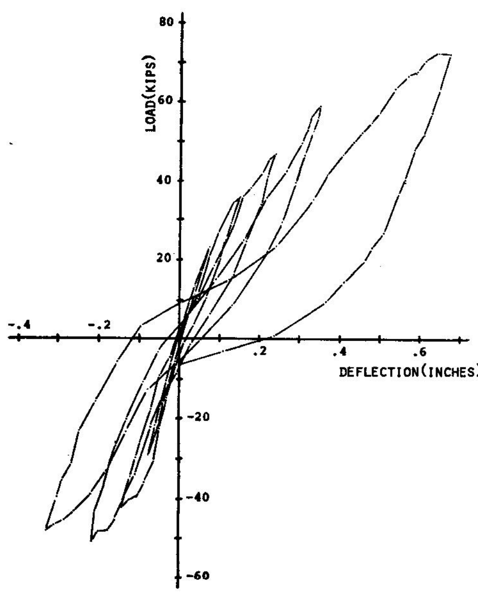


FIG. 5 THE LOAD-DEFLECTION RELATIONSHIP FOR PANEL B

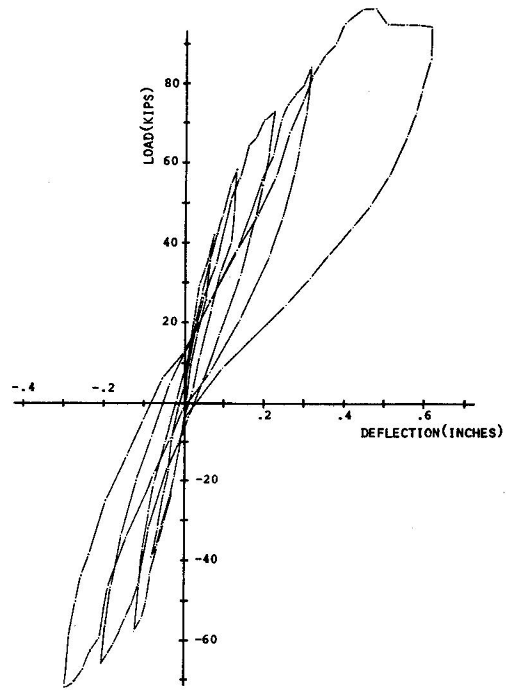


FIG. 6 THE LOAD-DEFLECTION RELATIONSHIP FOR PANEL C

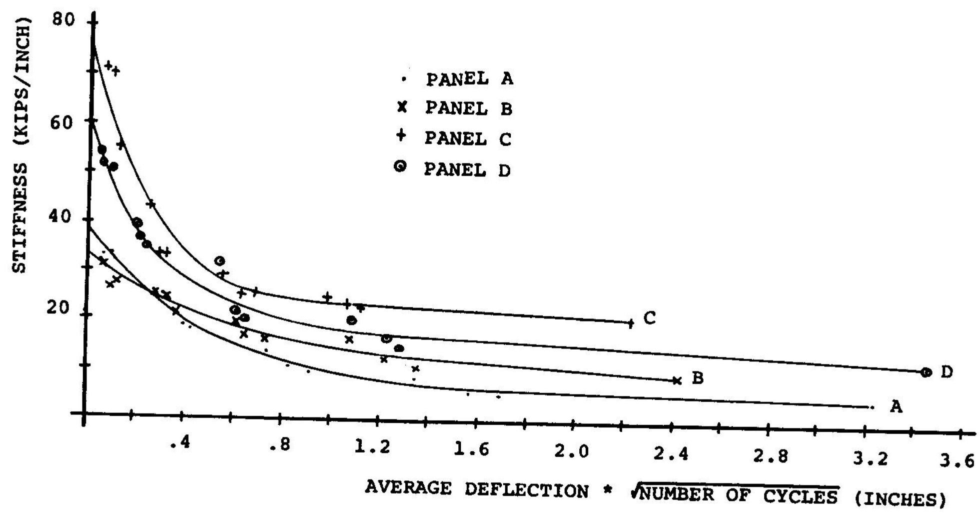


FIG. 7 STIFFNESS DEGRADATION OF WALL PANELS

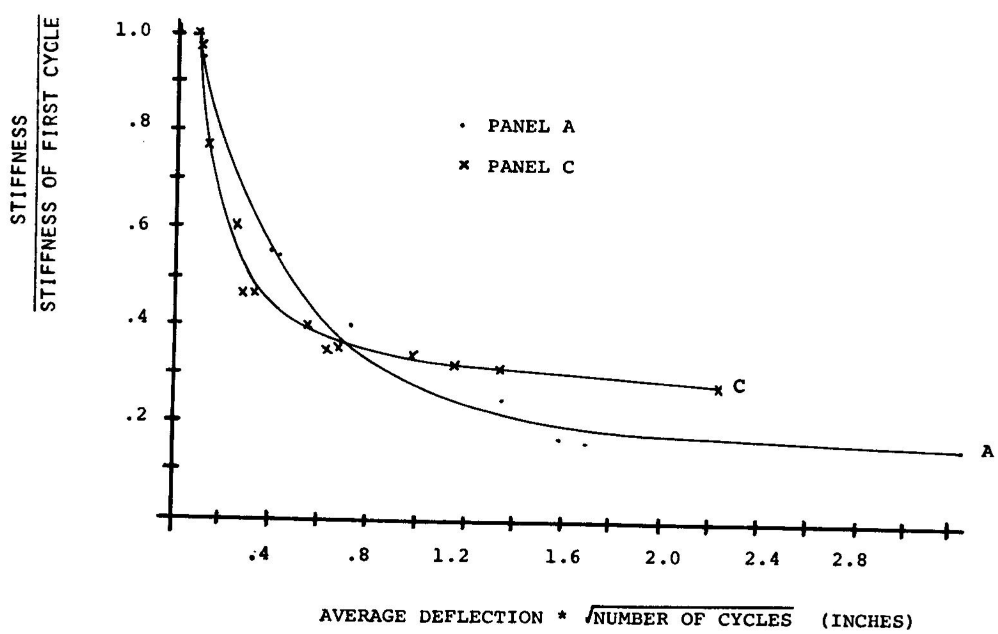


FIG. 8 STIFFNESS DEGRADATION OF WALL PANELS A AND C

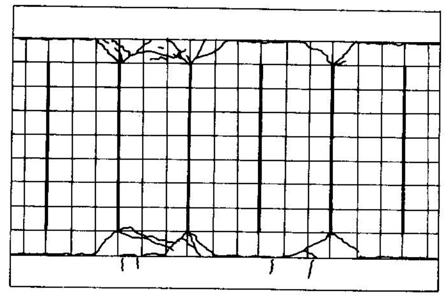

FIG. 9 FINAL CRACK PATTERN OF PANEL A

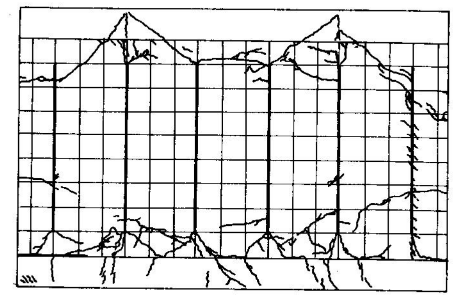

FIG. 11 FINAL CRACK PATTERN OF PANEL C

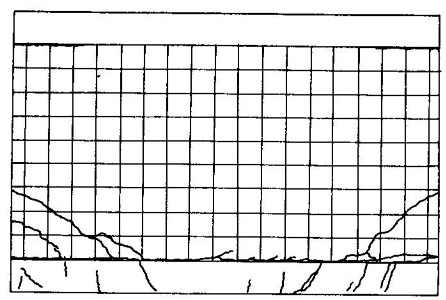

FIG. 10 FINAL CRACK PATTERN OF PANEL B

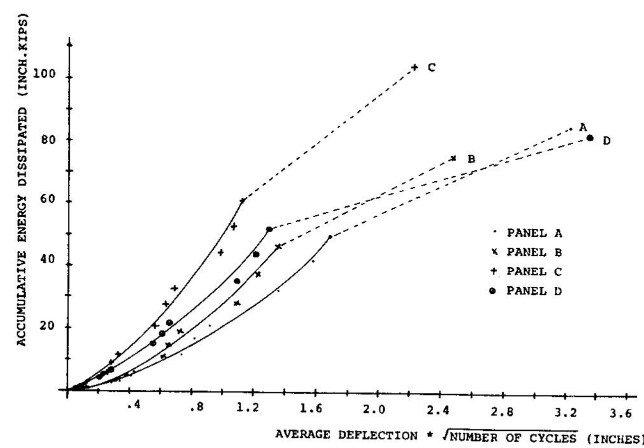


FIG. 12 ENERGY DISSIPATED BY THE WALL PANELS

DISCUSSION OF PAPER NO. 17

DUCTILITY OF LOAD-BEARING SHEAR WALL PANELS

SUBJECTED TO CYCLIC LATERAL LOADING

by

A.C. Heidebrecht and D. DeLisle

Question by: N.M. Hawkins

I would like to know how the authors decided on the length for the slits in their shear walls. My impression would be that the slits in their specimens were possibly too long for optimum behaviour. As the length of the unslitted portion of the wall is decreased the amount of cracking that will develop also decreases. Further, if the shear-friction concept (ACI Code 318-71) is applicable to slitted shear walls, the ability of the wall to transfer shear across the slit and therefore to dissipate energy along this slit, is dependent on the length of the slit and the amount of reinforcement transverse to the slit in the unslitted portion of the wall on the axis of the slit. Professor Muto has recommended slits with depths between 1/2 and 2/3 rds. of the depth of the wall depending on the amount of reinforcement in the wall. The length of the slit in the speaker's specimens was about three quarters of the depth of the wall.

Reply by: D. DeLisle

The original aim of the Experimental Investigation was to study the effect of including vertical slits, filled with asbestos sheet, in reinforced concrete walls under repeated cycles of lateral load. The initial program consisted of 10 walls with slit lengths ranging from zero length to the full height of the wall panel; because of time limitations, the number of walls to be tested was subsequently reduced to four. The length of slits of these walls were taken as zero length, full panel height and an intermediate panel height. The length of the intermediate height slits was chosen after a detailed study of the earlier Muto paper on the slitted wall concept and the geometry of the walls being tested. The earlier papers on slitted walls by Muto did not give sufficient information in the diagrams to determine the ratio of slit length to panel height. However, the length of the intermediate slits are not considered critical as both extremes (zero length and full panel height) were covered in separate tests and consequently the effect of varying the slits from zero length to the full panel height can be seen in the results presented in the paper.

The author considers that this short experimental program has shown that the inclusion of vertical slits in reinforced concrete walls subjected to repeated cycles of lateral loads is not the most satisfactory method of improving their performance. The most satisfactory method is still to be found.

Question by: J.G. MacGregor

How rapidly were the load cycles applied?

Reply by: D.J. DeLisle

We took about 30 points for each cycle and each was applied as fast as the apparatus would permit. It was not a rapid loading - it was essentially a static test.