A STUDY OF THE BEHAVIOUR OF COUPLED SHEAR WALL STRUCTUES

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SYNOPSIS

This paper presents "direct model" construction and testing techniques used in the study of the behaviour of two-storey coupled shear wall-slab structures.

Shear walls studied behaved as individual deep beams and failure occurred in a shear-compression mode with a ductility of four and above. The wall size had a significant effect on the ultimate strength of the assembly.

With improved reinforcing details, the shear-compression mode of failure can be either delayed or eliminated to obtain a more ductile flexural failure. However, the value of K assigned by SEAOC for ductility values above four remains constant at 0.67.

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Introduction

Shear walls have been used extensively in tall buildings to resist lateral forces due to wind and earthquakes. In the past, many building codes have treated shear walls as stiff and brittle elements not suitable for structures in earthquake prone regions. The Structural Engineers Association of California[1,2] has suggested a K value of 1.33 for box systems in which the lateral forces are resisted by shear walls. The comparitive value of K allowed for ductile moment resisting space frame resisting lateral loads is 0.67, suggesting that shear walls are non-ductile. The Commentary to the SEAOC Code suggests special provisions and limitations to minimize the brittle behaviour of shear walls.

Allen, Jaeger, and Fenton[3] divide shear walls into three classes: The shear-shear wall, the moment shear-wall, and the ductile-moment shear wall. The ductile-moment shear-wall is essentially an axially loaded flexural member having a minimum ductility of 3. In an analytical study, they noted that the ductility of a ductile moment shear-wall increases with a decrease in axial load or with an increase in flange area gross wall cross sectional area ratio, decreases with a decrease in concrete strength, and remains relatively constant with changes in main steel area. Suitable design techniques have been suggested to achieve ductile behaviour, without unloading or sudden collapse, when subjected to moderate or very heavy seismic loadings.

According to Jaeger et al[4], an elastic system consisting of two or three shear walls connected by a large number of horizontal beams, subjected to transverse (horizontal) loading in a vertical plane can be analysed with sufficient accuracy using techniques available in literature[5]. They developed a sufficiently accurate three dimensional analysis of multiple shear wall buildings which can be extended to study the dynamic behaviour of the structure.

Barnard and Schwaighofer[6] suggest that the theories available for analysis of coupled shear walls are time consuming and cannot be used in the design of shear wall-slab structures unless the width of the slab strip connecting the shear walls is known. They constructed 1/64-scale models of a 22 storeyed coupled shear wall-slab system with the walls cut from 1/4 in. thick epoxy sheets and the floor slabs were slotted to fit around the walls and then glued to the walls using an epoxy glue. The model was rigidly fixed at the base and the tests were performed with the model in a horizontal position with the uniformly distributed loading (total load = 73.5 lb.) applied at every alternate floor. They also noted from the experimental results on three models, each with a different slab width, that in using Rosman's theory, the entire bay width must be used as the width of the slab strip. In each test the stresses calculated by the Rosman theory agreed very well with the test results. They also outlined a simplification of Rosman's theory which reduces the amount of calculations required to analyse a system of shear walls and yet yields results of sufficient accuracy for design.

Qadeer and Stafford Smith[7] discuss the interaction between shear walls and slabs in shear wall-slab structures. They examined the bending of the slab along the line of the walls along with its resistance to the wall rotation. The slab was analysed as an elastic plate by using the finite difference method in the form of a generalized computer program. Results are presented graphically and give the non-dimensional slab stiffness for varying slab proportions, wall spacings, and slab to wall lengths. An alternative value for the slab stiffness in terms of an EI or an "effective width" may also be derived from the curves.

Qadeer and Stafford Smith(7) constructed a working model consisting of two walls, pivoted at their bases and rigidly coupled to a floor slab. Asbestos cement sheet was used for the slab and the walls were made of heavy steel plate to insure that their in-plane strains were relatively small and had a negligible effect on the measured slab stiffnesses. The slab was clamped to the walls by bolts passing through the slab and holding a steel stiffener against its upper face. Two types of connections were examined: The first held the slab along the whole section of the wall and was intended to represent the actual conditions in a shear wall building; the second pinned the slab to the walls only at the ends and middle of the length of the wall to represent the nodal deformations assumed for theoretical analysis. The difference in the measured stiffness was insignificant and the subsequent experiments were performed using the full length connection. Load increments were applied horizontally at the slab level and the rotation of the walls was measured about their pivots, thereby generating data for flexural stiffness for various slabs and widths of walls. The experimental results provide strong evidence to support the accuracy of the theory and the computer program.

The proposed ACI Code(8) includes an appendix giving special provision for the design of reinforced concrete structures located in a seismic zone where major damage to construction has a high probability of occurrence. The major philosophy of this appendix is to minimize seismic forces by producing a ductile energy absorbing structural system containing elements whose ultimate strength tends to develop through the formation of plastic hinges rather than through less ductile flexural, shear, or compression failures.

A study of available literature(5) on shear walls, shear walls coupled with frames, and shear walls coupled with slabs indicates that most of the methods of analysis are based on the assumption of linear elastic behaviour. Suitable design and analysis tools are available for the ultimate strength design of multi-storey, multi-bay reinforced concrete frames. A computer program is being developed at McGill University to predict complete behaviour of a reinforced concrete frame from zero load up to failure. An estimate of the ductility of a framed structure with or without shear walls can be obtained using this finite element program.

Limited work(6,7) on shear wall-slab interaction is basically an extension of analyses of shear walls connected with beams suggested by Rosman(9), Beck(10), Coull and Choudhry(11), and Khan and Sbarounis(12), etc. These methods treat the connecting slabs as beams and work out an equivalent flexural stiffness for slabs which are assumed to behave linear elastically. The experimental work using indirect models is also aimed at investigating behaviour in the linear elastic range. Mamet(13) has extended Mehrotra and Mufti's finite element program(14) for complete three-dimensional analysis of tall buildings by considering floors as substructures.

He accounts for the flexural and in-plane deformations of the elements which are assumed to be linear elastic in their behaviour. This represents a significant improvement on the existing analysis methods for shear wall structures, however no tools are available to assess the ductility of reinforced concrete shear wall-slab structures which are being used extensively in tall apartment and office buildings.

The behaviour of shear wall-slab structures depends on material properties (steel and concrete), geometry and reinforcement details of slabs and shear walls, and their layout in plan. The objectives of this experimental study are as follows:

1) To develop techniques for building and testing suitable direct models of shear wall-slab structures until failure, and

2) To study the behaviour of shear walls for the following effects:

a) The behaviour of two identical shear walls lying in one plane connected by slabs (this study was limited to two-storey structures).

b) the effect of clearance between the shear walls (distance between the inside faces of the shear walls in the model)

c) the effect of variation of connecting slab width and

d) the ductility and the energy absorption characteristics of shear wall-slab structures.

Construction of Models

A shear wall-slab structure is basically a three-dimensional problem which requires an examination of the rotations and deformations in the shear walls and the connecting slabs comprising the structure. However, for shear-wall slab structures which consist of symmetrically laid simple shear walls and are subjected to symmetrical loadings, it is adequate to consider the behaviour of an interior bay with the shear walls and the connecting slabs. The test specimen selected is a one-tenth scale model of an interior bay of such a structure (Fig. 1).

Casting a monolithic shear wall-slab system would have presented problems in the construction of the formwork, placing the reinforcement, concreting, and the subsequent stripping of forms. Therefore it was decided to use precast wall and slab elements which were cast in flat plywood forms. One set of forms was used for the 3/4 inch thick shear wall elements and could be adapted for any number of storeys. Two wooden studs were screwed to the base of the slab form to provide openings for the shear walls. The wall and the slab elements were reinforced with soft steel deformed wires and were concreted using a micro-concrete mix prepared from a mixture of high early strength cement and crushed quartz sand, (Fig. 2)[15]. The finished slabs were set at 15 inch centres in a storage rack, (Fig. 2,4). The walls were threaded through the slab openings which were carefully aligned and glued with an epoxy glue. (A suitable test was devised to determine the shear strength of glued joints in concrete blocks). The reinforcement at the lower end of this shear wall-slab assembly was cast in a concrete foundation block (36 x 6 x 6 in.) to provide a rigid base for the structure. Suitable lifting handles were provided in the base. The assembly had to be handled extremely carefully because of lack of sufficient strength and stiffness transverse to the shear walls.

Eight assemblies were constructed along the following lines:

1. A series of three similar models for the purpose of developing a technique for assembling the walls and slabs using epoxy glue and a suitable method to test the model to failure. The three models had identical dimensions (Table 1). The wall section used was 9 x 3/4 in. The results of Model #2 are reported in this paper.

2. A series of five models with identical shear walls 6 x 3/4 in. in section and the clearance C between the walls ranging from 3 in. to 15 in. in increments of 3 in. Other details of the assemblies are given in Table 1. Results of tests on Models #1 to 4 with clearances of 6, 9, 12 and 15 in. are reported in this paper.

Test Procedure

The following steps were involved in setting up the assembly for the test, (Fig. 5):

1) The assembly was handled extremely carefully and placed on an 18 WF 50 beam as shown in Fig. 5. The assembly base was clamped to the steel beam with three pairs of steel bars and six one-inch diameter bolts as shown. A steel angle was clamped at the front end of the base (away from the applied loads) to prevent any horizontal movement during loading. This arrangement provided the shear wall-slab assembly with a rigid unyielding foundation.

2) Lateral loads simulating wind pressures were applied at floor levels using two hydraulic jacks attached to a rigid vertical member as shown in Fig. 5. Applied loads were monitored by strain indicators through load cells stationed between the hydraulic jack and the rigid support (Fig. 6). A 3/4-in. diameter roller was set within horizontal guides on the other end of the jack to distribute the applied loads uniformly to the slab end. The load applied to the upper slab was maintained at half the value of the load on the lower slab throughout the test.

3) Fourteen dial gauges were used for measuring deflections. Eight of these gauges were used to measure lateral displacements of the slabs and the wall extremities just above and below the slab (Fig. 6). Two dial gauges were set at the lower wall extremities to check the lateral movement of the base. The remaining four gauges measured the vertical movements of the top corners of the two shear walls. These gauges were modified by attaching a 3 x 1 x 1/8 in. aluminum plate at right angles to the moveable shaft. Each plate was set on the vertex of a right-angled plexiglass triangle glued at a shear wall top corner, (Fig. 7).

4) (a) One day before the test, each load cell was calibrated using a strain indicator. (Two strain indicators were used for the two load cells). A table of strain readings was prepared to give the desired loads P/2 at the upper slab and P at the lower slab.

(b) To achieve the required load levels, each strain indicator was set to a specified strain reading (obtained in step (a) above) and the hydraulic pumps were manipulated until the bridges on the two strain indicators were balanced.

(c) The dial gauge readings were taken approximately five minutes after each loading increment had stabilized. The pumps had to be adjusted slightly to maintain constant loads. The walls and the slabs were examined for the appearance and propagation of cracks which were traced with a marking pen and the ends marked with the load value.

(d) Just before failure, all dial gauges were removed.

General Behaviour

If the two shear walls are assumed to act separately, they will bend as simple cantilevers each resisting half the applied load, (Fig. 8(a)). However, when coupled with connecting slabs, the interaction with slabs (frame action) can impose reversed curvatures at the slab-wall junction and the assembly behaves as shown in Fig. 8(b). The free body diagrams of shear wall and slab elements of the assembly are shown in Fig. 8(c), along with the forces and moments acting on them. The magnitudes of moments generated by the connecting slabs at D_1, E_1, A_2, and B_2 is dependent on their relative stiffnesses. For relatively stiffer connecting slab elements, it is possible to have compression at A_1 and A_2 and tension at D_1 and D_2 as shown in Fig. 8(c) and thus have a gradual transition from cantilever action to frame action depending on the slab stiffness. This phenomenon is more pronounced when the number of storeys considered is large. This had been demonstrated experimentally by Barnard and Schwaighofer(6) with the 1/64-scale 22-storey models which were instrumented to measure strains and deflections at selected stations.

Vertical movements of points A_1, D_1, A_2, and D_2 were noted for all eight shear wall-slab assemblies. However, no strains were measured in this study. Typical deflections for the upper ends of Models #2 and #5 of Series 2 are shown in Fig. 9 and Fig. 10 respectively. It was noted that the upward movement of point A_1 was always greater than that of A_2 while the downward movement of point D_1 was always smaller than that of D_2. This suggests a gradual transition from cantilever action to frame behaviour as the "clearance" C between the walls was decreased from 15 in. to 6 in. Future investigations will include measurements of strains on concrete and steel to confirm these trends.

Cracking Patterns

1. Shear Walls: Cracks were traced as they appeared and typical crack patterns are shown for Model #2 in Series 1 in Fig. 11

and 12. Details of cracks for Models #1 and #4 in Series 2 are indicated in Fig. 13 and 14 respectively.

a) Flexural cracks were first observed on the tension face of the lower storey shear walls at loads (at level 1) between 1.2 and 1.3 kips for Series 1. Similar cracks were noted in Series 2 for loads between 0.7 and 0.8 kip. However, an examination of the load-deflection curves for all assemblies indicates an earlier cracking at approximately 1.0 kip for Model #2 in Series 1 and approximately 0.4 kip for all assemblies in Series 2.

b) These flexural cracks propagated and widened with further increase of load and became inclined indicating the prominence of shear deformations. The newer cracks that formed at later stages of loading were generally inclined.

c) As the applied loads were increased, the cracks progressed towards the corners F_1 and F_2 and finally failure occurred by crushing of concrete at the lower end of the inclined crack in a "shear-compression" mode. (This is a reasonably ductile mode of shear failure). It must be noted that the reinforcement for the shear walls in Series 2 consisted of a single layer of three D-2.5 wires parallel to the axis of the wall at 2 2/3 in. centres and the transverse reinforcement consisted of D 2.5 wires at 4 in. centres. The reinforcement details can be improved in the prototype walls to prevent or delay the shear-compression failure and to achieve a flexural mode of failure. Also, the concrete at points F_1 and F_2 in the prototype can be suitably confined to increase its strain capacity, thereby adding to the ductility of the assembly.

d) Excepting for one assembly, no tension cracks appeared anywhere in the upper storey shear walls as the applied loads were increased from zero until failure.

2. <u>Connecting Slabs</u>: At higher loads (between 0.5 Pult and 0.7 Pult), radial tension cracks appeared in the slabs originating from the points D_1, E_1, A_2 and B_2 (Fig. 15). The slabs however, continued to transmit the loads to the shear walls as the loads were increased and the cracks continued to propagate and increase in number.

Slab reinforcement will be strain gauged in future experimental work to assess the tensile forces in the plane of the slabs which cause cracking.

Load-Deflection Characteristics and Ultimate Strength of Assemblies

Typical load-deflection curves for Model #2 in Series 1 are indicated in Fig. 16 and 17 respectively. The load-deflection curves indicate a significant increase in the load-carrying capacity and deflections at level 1 after the appearance of diagonal cracks. These curves indicate that based on deflections, these walls have a ductility of 4 or more. With improved reinforcement detailing, a flexural failure can be achieved in these shear walls. This will cause the load-deflection curves to flatten out showing much higher deflections at ultimate load. Loading beyond the ultimate load will result in further deformations accompanied by a drop in strength indicating very ductile behaviour.

It may, however, be noted that the lower-storey shear walls model a 12.5 ft. high shear wall, (60 x 7.5 in. in section) and based on experimental evidence these walls can be expected to exhibit a deflection of approximately 2 inches at a loading stage before failure (when the dial gauges were removed). These walls have significant ductility (more than 4) even when they have failed in a shear-compression mode. With proper reinforcement detailing, much higher ductility can be expected.

The loads at failure at levels 1 and 2 are detailed in Table 2.

TABLE 2

Series	Model Number	Load At Level 2 (P/2) Kips	Load At Level 1 (P) Kips	Total Ultimate Load (P + P/2) Kips
1	#2	1.650	3.300	4.95
2	#1	0.425	0.8500	1.275
2	#2	0.550	1.100	1.650
2	#3	0.475	0.950	1.425
2	#4	0.575	1.150	1.725

It is noted that the size of the shear wall has a significant effect on the assembly ultimate strength. The Model #2 in Series 1 exhibited a strength approximately three times that of the average assembly strength in Series 2. The assemblies in Series 2 show that as the clearance between the shear walls is decreased from 15 in. to 6 in. the total horizontal load (3P/2) resisted falls from 1.725 kips to 1.275 kips. However, more experimental work is needed before any generalizations of the strength and ductility of shear walls can be made with confidence.

Energy Absorption Characteristics: Rapid and Reversed Loadings

Energy stored in a reinforced concrete shear wall or a reinforced concrete slab can be assessed from a consideration of the area under the moment-curvature curve for flexural deformation in the elastic and inelastic ranges. For shear wall elements, the strain energy due to shear has to be considered while for the slabs the axial strain energy may be significant. Blume, Newmark, and Corning (2) have given equations for evaluating the flexural strain energy. A computer program is being developed at McGill University to assess the energy absorption capacities of concrete elements due to significant deformations, (flexure, axial loads, shear, and torsion) in the elastic as well as the inelastic range.

Several investigations have been carried out to study the behaviour of reinforced concrete under loadings simulating a blast loading or an explosion.[16,17] The resulting strength increase can be attributed to the increase in the steel yield strength with an increase in the rate of loading[18]. The loads induced by earthquakes are applied at a rate slower than shock loadings and consequently the corresponding increase in strength will be smaller. It is therefore advisable to use the static strength of reinforced concrete component members in design for earthquake resistance.

Structures subjected to earthquakes undergo reversals in the direction of loading during an earthquake. As long as the deformations and the resulting stresses are well below the yield level of the member, this reversed loading is similar to a fatigue loading.

However, if some element yields under reversed loadings, it may be necessary to consider the effect this yield will have on the element behaviour under loading reversals. Previous investigations at the University of Illinois have indicated that unless the first damaging load produces deformations in excess of approximately 80 per cent of collapse deformation, the capacity in the reversed direction will only be slightly impaired.

Conclusions

1. Suitable construction and testing techniques have been developed to study the behaviour, strength, and ductility of models of two storey coupled shear wall-slab structures. These techniques can be extended to structures with more than two shear walls and more than two storeys high.

2. A gradual transition is noted in behaviour of coupled shear wall-slab structures from cantilever action to frame action depending on the relative stiffness of the connecting slabs.

3. The shear walls studied in this investigation behaved as individual deep beam elements in a frame and failure occurred in the "shear-compression" mode with a ductility of 4 and above. The value of K allowed by SEAOC for such a structure is 0.67.

4. With proper reinforcing details, the shear-compression can be either delayed or completely eliminated and a more ductile flexural failure can be obtained. However the value of coefficient K for these (more) ductile walls will still be 0.67.

5. Suitable computer programs are being developed to assess the ductility of coupled shear wall-slab structures. These programs will account for the energy stored in the structure elements on account of bending, shear, direct (tensile and compressive) forces and torsion.

6. Shear walls coupled with moment resistant frames and/or slabs provide structures which have adequate ductility and energy absorption capacity and are thus suitable for resisting loads induced by earthquake motions.

Acknowledgements

The experimental work reported in this paper was conducted by Mr. C. K. Wong in the Laboratories of the Department of Civil Engineering and Applied Mechanics, McGill University, Montreal. The assistance and encouragement given by Professor J. O. McCutcheon, Chairman of the Department, is appreciated by the authors. Credit for the idea of using precast shear walls and slabs and the associated formwork and construction techniques is due to Mr. C.J. Zanolin, of Francon Ltee., Montreal. The financial assistance of the National Research Council, Canada Emergency Measures Organization, and the Defence Research Board of Canada is gratefully acknowledged.

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TABLE I

DETAILS OF TEST SPECIMENS

MODEL NUMBER		FLOOR SLAB DETAILS						SHEAR WALL DETAILS					CONCRETE STRENGTHS		
		L	W	T	C	Main Steel	Transverse Steel	L	W	T	Vertical Steel	Horizontal Steel	Compressive	Indirect Tensile	Remarks
SERIES 1	1, 2 & 3	36"	12"	3/4"	6"	4 - D2.5 Wires	D2.5 4 in. Centres	36"	9"	3/4"	4 - D2.5 Wires	D2.5 4 in. Centres	4470 psi	1060 psi	
SERIES 2	1	30"	12"	3/4"	3"	4 - D3.0 Wires	D3.0 4 in. Centres	36"	6"	3/4"	3 - D2.5 Wires	D2.5 4 in. Centres	5310 psi	880 psi	Damaged in handling
	2	30"	12"	3/4"	6"	4 - D3.0 Wires	D3.0 4 in. Centres	36"	6"	3/4"	3 D2.5 Wires	D2.5 4 in. Centres	5870 psi	920 psi	
	3	30"	12"	3/4"	9"	4 - D3.0 Wires	D3.0 4 in. Centres	36"	6"	3/4"	3 D2.5 Wires	D2.5 4 in. Centres	5720 psi	840 psi	
	4	30"	12"	3/4"	12"	4 - D3.0 Wires	D3.0 4 in. Centres	36"	6"	3/4"	3 D2.5 Wires	D2.5 4 in. Centres	5250 psi	940 psi	
	5	30"	12"	3/4"	15"	4 - D 3.0 Wires	D3.0 4 in. Centres	36"	6"	3/4"	3 D2.5 Wires	D2.5 4 in. Centres	5020 psi	820 psi	

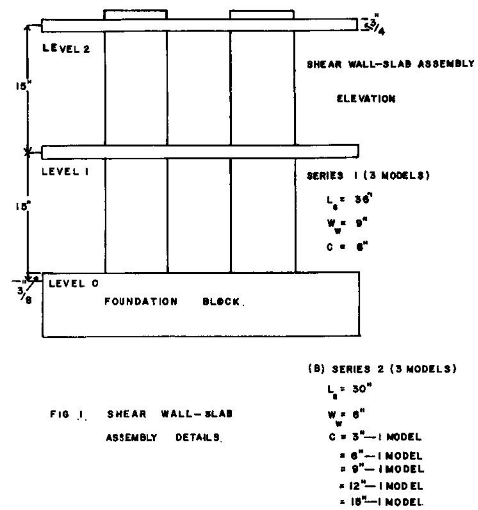


FIG 1. SHEAR WALL-SLAB ASSEMBLY DETAILS.

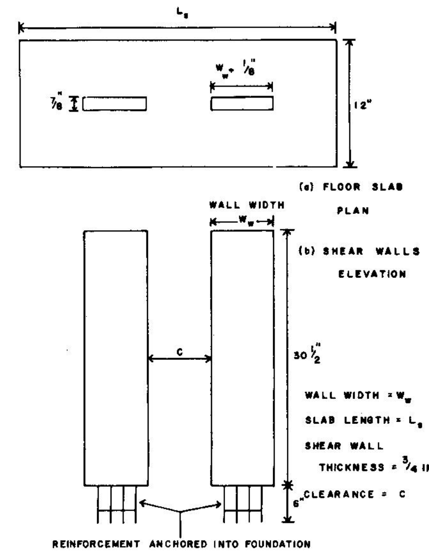


FIG.2. SHEAR WALL & SLAB DETAILS

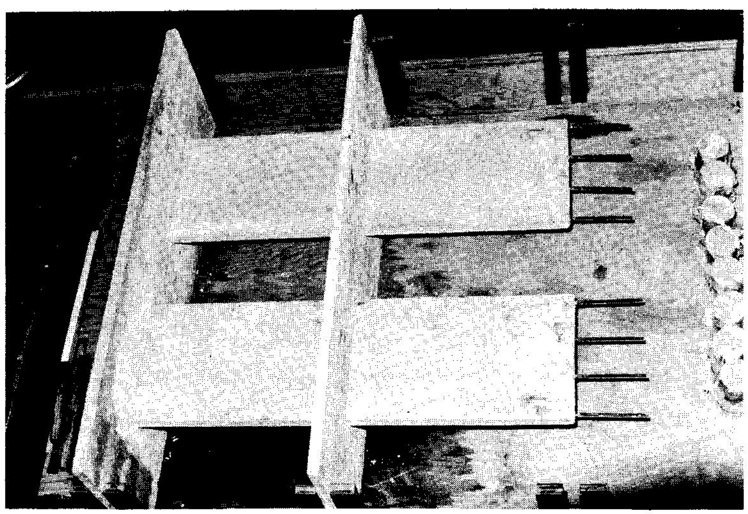


FIG .4. . ASSEMBLED SHEAR WALL - SLAB ASSEMBLY

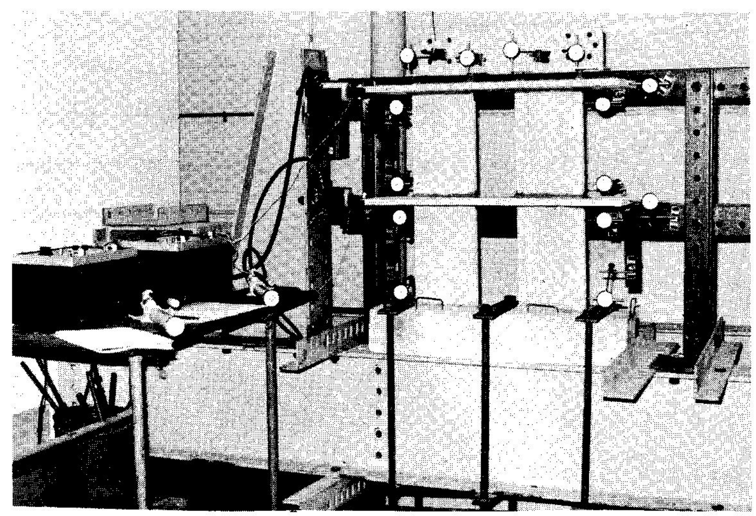


FIG .3. TEST SET—UP

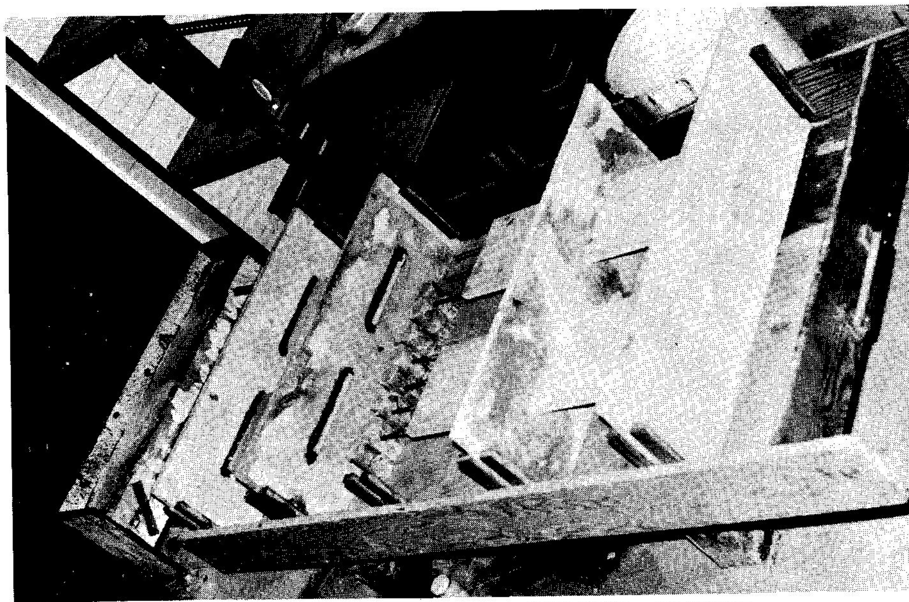


FIG. 5. STORAGE RACK WITH FOUNDATION FORMWORK

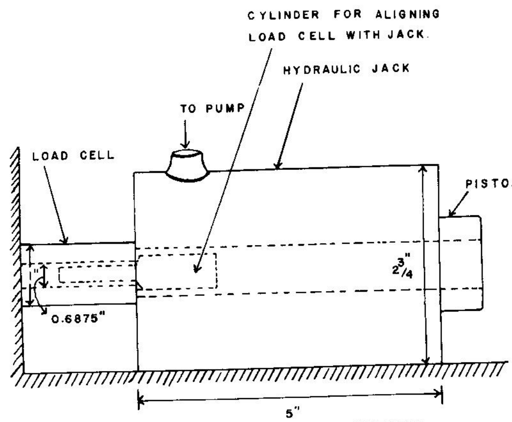


FIG. 6. LOADING ARRANGEMENT

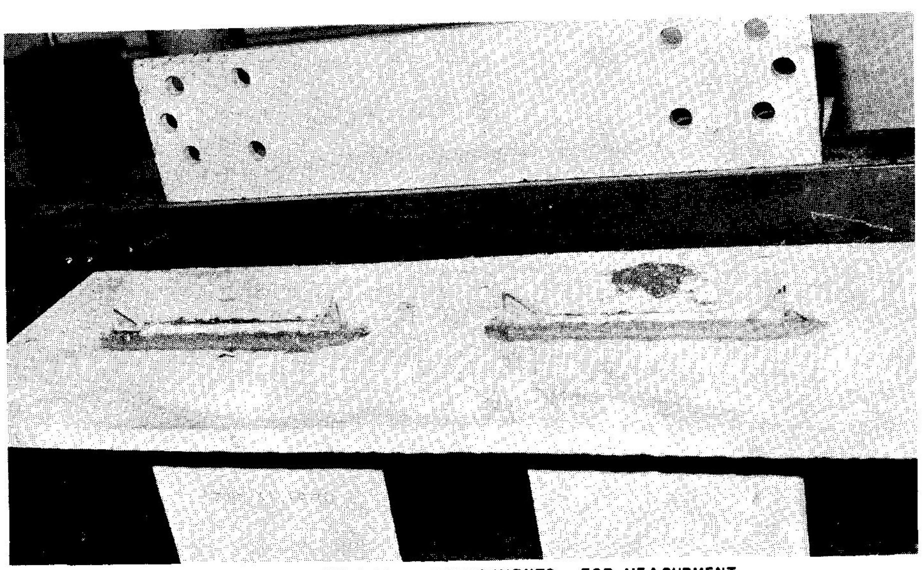
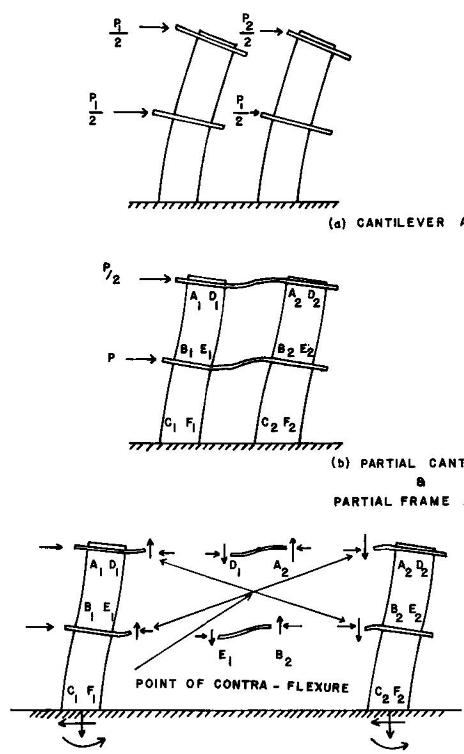


FIG. 7. PLEXIGLASS ATTACHMENTS FOR MEASURMENT OF VERTICAL MOVEMENT OF SHEAR WALLS.



(a) CANTILEVER ACTION.

(b) PARTIAL CANTILEVER & PARTIAL FRAME ACTION.

(c) FREE BODY DIAGRAM OF SHEAR WALL & SLAB ELEMENTS.

FIG. 8. DEFORMED SHAPES OF SHEAR WALLS

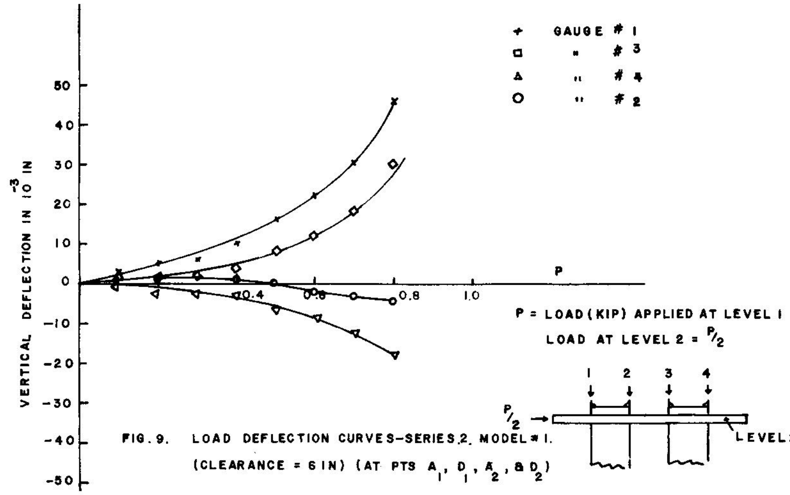


FIG. 9. LOAD DEFLECTION CURVES–SERIES 2, MODEL # 1.
(CLEARANCE = 6 IN) (AT PTS A_1, D_1, A_2, & D_2)

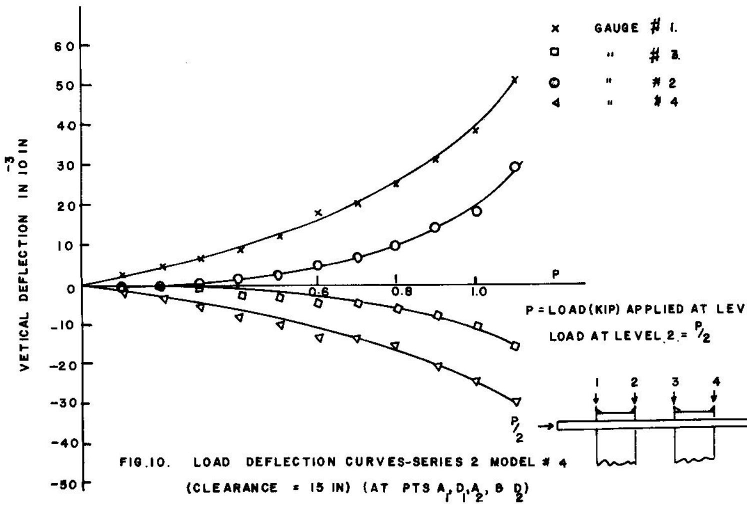


FIG. 10. LOAD DEFLECTION CURVES–SERIES 2 MODEL # 4
(CLEARANCE = 15 IN) (AT PTS A_1, D_1, A_2, & D_2)

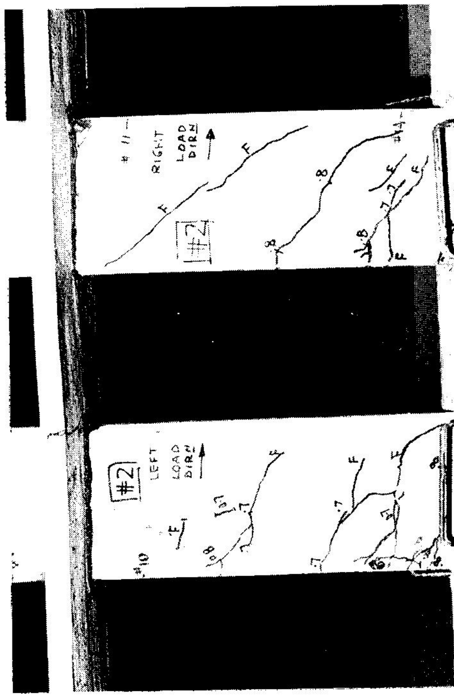


FIG. 13. CRACK PATTERN & FAILURE MECHANISM
SERIES # 2 MODEL # 2 (LOWER STOREY)

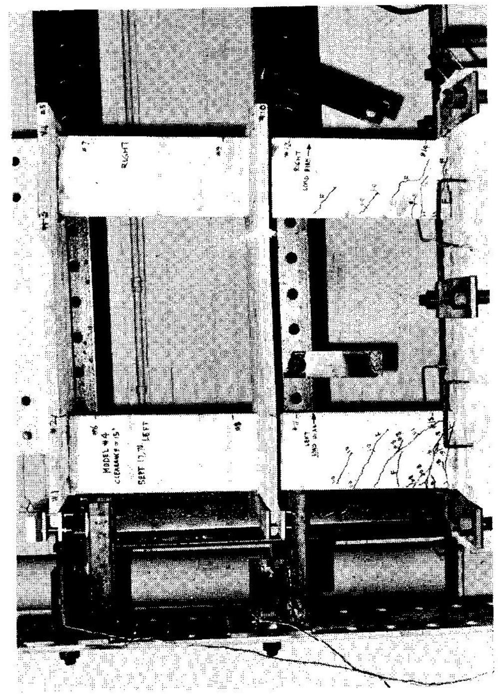


FIG. 14. CRACK PATTERN & FAILURE MECHANISM
SERIES # 2 MODEL # 5 (CLEARANCE = 1.5 IN)

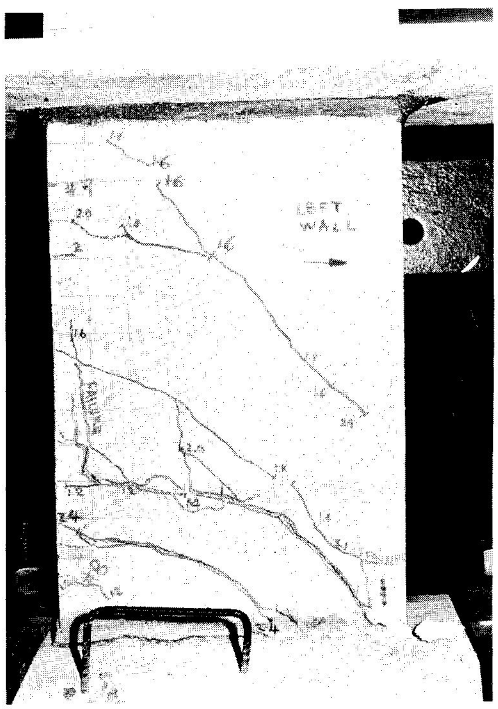


FIG. 11. CRACK PATTERN & FAILURE MECHANISM
SERIES # 1 MODEL # 2 LEFT WALL.

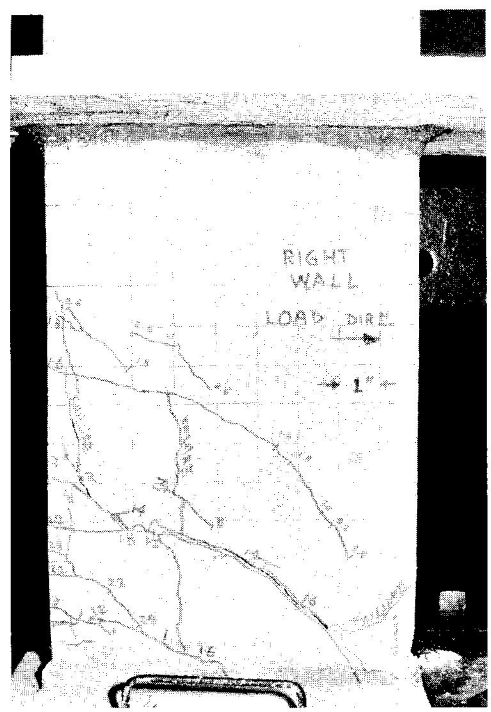
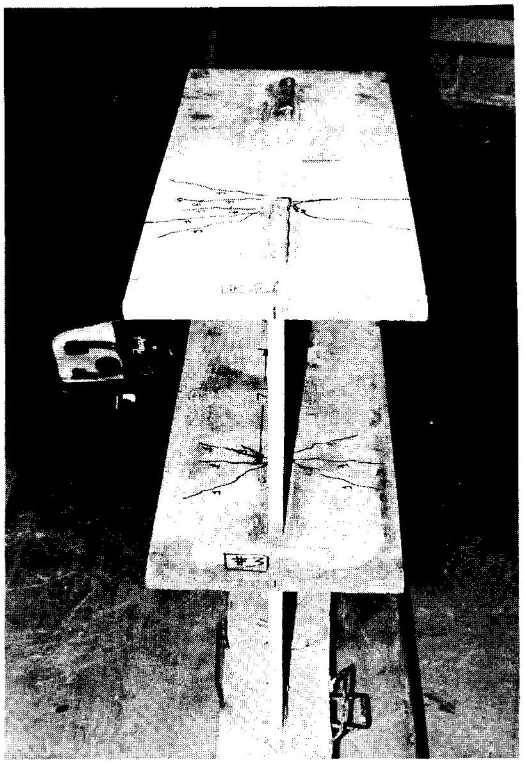


FIG. 12. CRACK PATTERN & FAILURE MECHANISM
SERIES # 1 MODEL # 2 RIGHT WALL.



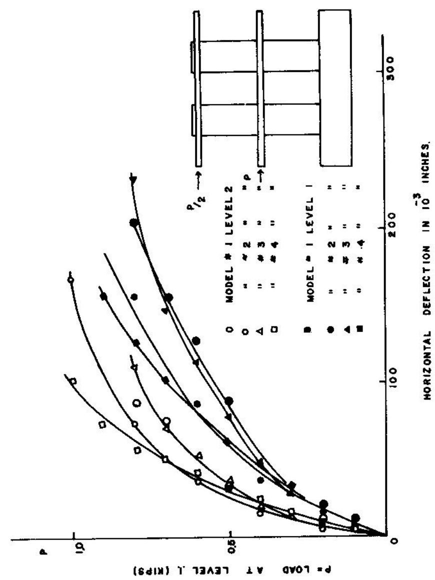


FIG.17. LOAD DEFLECTION CURVES FOR MODELS IN SERIES.2.

FIG .16. TYPICAL TENSION CRACKS IN SLABS AT SLAB WALL JUNCTION (SERIES # 2 MODEL # 4)

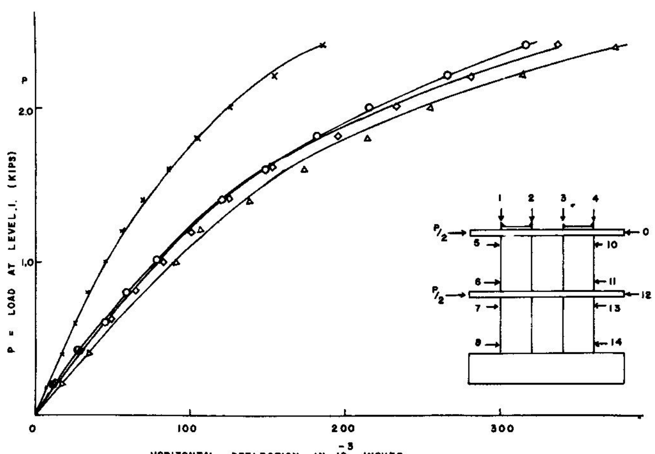


FIG.18. LOAD DEFLECTION CURVES SERIES.1. MODEL.2.

DISCUSSION OF PAPER NO. 16

A STUDY OF THE BEHAVIOUR OF COUPLED SHEAR WALL STRUCTURES

by

M.S. Mirza and L.G. Jaeger

Question by: D. J. DeLisle

What was the reinforcing ratio you used in the walls and why did you use this value?

Reply by: M.S. Mirza

The shear walls in Series 1 (9 in. depth) were reinforced with 4-D2.5 wires while those in Series 2 (6 in. depth) were reinforced with 3-D2.5 wires. Nominal area of cross-section of one D-2.5 wire is 0.025 in^2. Based on the gross cross-sectional area, these areas lead to steel percentages of 1.48 and 1.67 for Series 1 and 2 respectively. These percentages were selected to keep the sections under-reinforced and to use deformed steel wires readily available.

Question by: K.G. Asmis

Would you comment on your experimental program concerning rapid, reversed loading in the non-linear range?

Reply by: M.S. Mirza

The preliminary tests reported in this paper were on coupled shear walls subjected to monotonically increasing static loads until failure. More such tests will be undertaken to understand the behaviour of coupled shear walls and to estimate the influence of different parameters. This phase will be followed by tests under reversed and rapid loadings.