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TESTS ON CONNECTIONS TYPICAL OF LOW RISE PRECAST REINFORCED CONCRETE BUILDINGS IN NEW ZEALAND

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ABSTRACT

A large percentage of low-rise buildings in New Zealand are built using tilt-up construction. These buildings are commonly designed for earthquake resistance as elastic responding structures or as structures of limited ductility. The thickness of the reinforced concrete walls in these type of buildings ranges between 120 and 160 mm. Suspended floors are built using proprietary precast floor systems, particularly hollowcore units over which 50 to 70 mm thick of concrete topping is cast-in-place. Wall-to-slab connections are designed to transfer the inertia forces generated by the mass of the slab by a shear friction mechanism. The connections need to withstand imposed rotations due to the out-of-plane lateral displacement of the walls and therefore should be designed to avoid a brittle concrete pull-out failure. This paper summarizes a test programme on the seismic resistance of thin wall-to-hollowcore slab connections of details commonly used in New Zealand in low rise buildings.

KEYWORDS

Anchorage; precast concrete; tests; tilt-up construction; wall-slab connections.

INTRODUCTION

Tilt-up construction has been used in New Zealand for about 35 years. Today, this method of construction is widely used in low rise buildings up to two storeys and occasionally in three storey buildings. Residential buildings can be described as "box" shaped structures since they possess significant number of walls spread throughout the built area. In contrast, commercial and industrial buildings using tilt-up construction usually have a limited number of walls, chiefly located on their perimeter. A feature of these low rise precast concrete buildings is their rather short fundamental period of vibration of no more than 0.4 secs.

Tilt-up buildings are designed for earthquake resistance using lateral load and design requirements stipulated by design standards (NZS 4203, 1992 and NZS 3101, 1995) for "elastic" responding structures and for "structures of limited ductility".

Structural engineers have devised a range of connection methods with little standardization of details. Often, these methods go beyond the recommendations of the standard for the design of concrete structures (NZS 3101, 1995) which sets minimum requirements for the construction with precast concrete elements. For example, the standard for the design of concrete structures (NZS 3101, 1995) requires that precast floor slabs be seated at least 50 mm or L/180, whichever is greater, where L is the length of the span. Shorter seating lengths can only be used if demonstrated to be adequate by analytical or experimental means. The standard also requires that reinforcing bars that are used for connecting the walls and the slabs be properly anchored in the wall to develop a mode failure by yielding of the steel. For bars anchored with a standard hook and protruding from the face of the walls, NZS 3101 requires the anchorage length to be at least 150 mm or 8 times the diameter of the bar, whichever is greater. A number of buildings built prior to 1995 have seating details or anchorage lengths of the hooked bars that do not satisfy the standard requirements.

The University of Canterbury has been engaged in a research programme to assess the seismic resistance of tilt-up structures and their connections. This paper summarizes the experimental programme conducted on wall-to-slab connections.

TEST PROGRAMME ON WALL-TO-SLAB CONNECTIONS

The objective of test programme was to study the seismic behaviour of connections between thin reinforced concrete walls and proprietary precast concrete floors. The connections studied are commonly used in local construction in New Zealand and do not satisfy the requirements of the current design standard.

This research focused on tilt-up industrial or commercial buildings because they have seismic forces per unit length of walls much larger than the walls of residential buildings. A typical building was designed as a benchmark structure for the tests. The plan view and elevation of the building are shown in Fig. 1. The test assemblies were extracted from the 4.5 m long walls located in the centre of the building in the longitudinal direction. For seismic forces acting in the long direction of the building, the wall-to-slab connection in this part of the building transfers the inertia forces from the floor to be carried by the wall by a shear friction mechanism. For seismic forces in the orthogonal direction, the inertia forces are transferred to the north and south side walls. The wall-to-slab connections in the studied region of the building act as secondary connections and are subjected to an imposed rotation due to deformation compatibility, plus the additional displacement caused by the flexibility of the diaphragm.

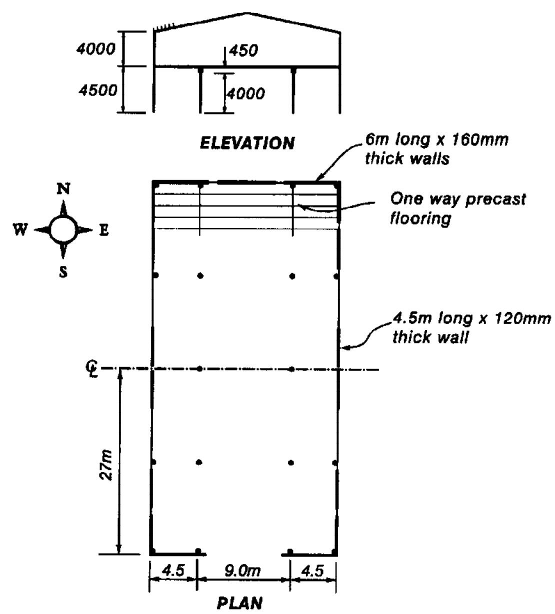


Fig. 1. Plan view of the prototype building.

Fig. 2 depicts the test arrangement. The connection region between the wall and the slab was subjected to simultaneous rotation in the direction of the slab and to shear loading along the direction of the wall. The rotation was imposed by vertically displacing the end of the slab with a double acting hydraulic actuator. The shear loading was applied by hydraulic jacks located as close as possible to the face of the wall, thus, simulating the boundary conditions of the extracted assembly within the whole building. The vertical position of the jacks applying the shear loading coincided with the centroid of dead load. The slab was loaded with lead ingots to represent the superimposed dead load plus 40 percent of the service live load of the building.

Fig. 3 shows the load regime applied to the test units. The imposed rotation (see Fig. 3 (a)) was defined as the vertical displacement of the hollowcore unit measured at 2.51 m from the centreline of the wall divided by this distance. The shear loading is shown in Fig. 3 (b). The magnitude of the shear loading and rotation applied to the units was derived from time-history inelastic dynamic analyses of the building. The dynamic analyses, accounted for the diaphragm flexibility and used earthquake records matching the site elastic response spectra recommended by NZS 4203. The analysis indicated that the rotation at the connection region at the face of the wall in these type of buildings may be of the order of 0.01 rads. In the longitudinal direction (see Fig. 3 (a)), the cycles were applied by controlling the rotation, whereas in the transverse direction (see Fig. 3 (b)) the cycles were load controlled. A complete load cycle in the transverse direction

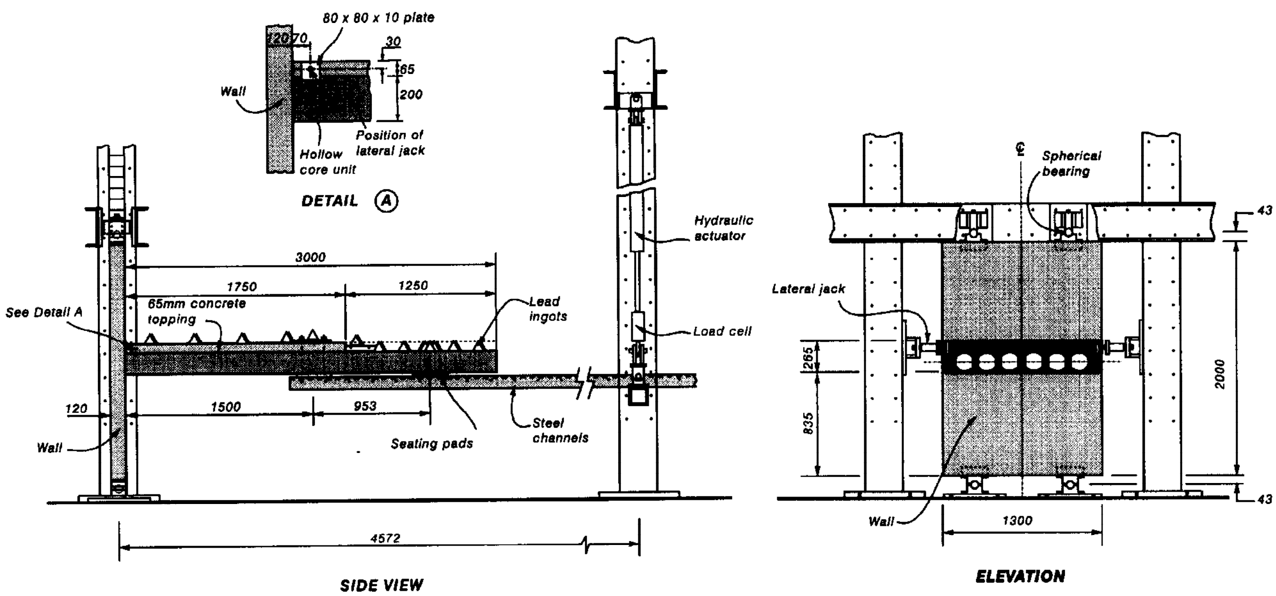


Fig. 2. Loading arrangement.

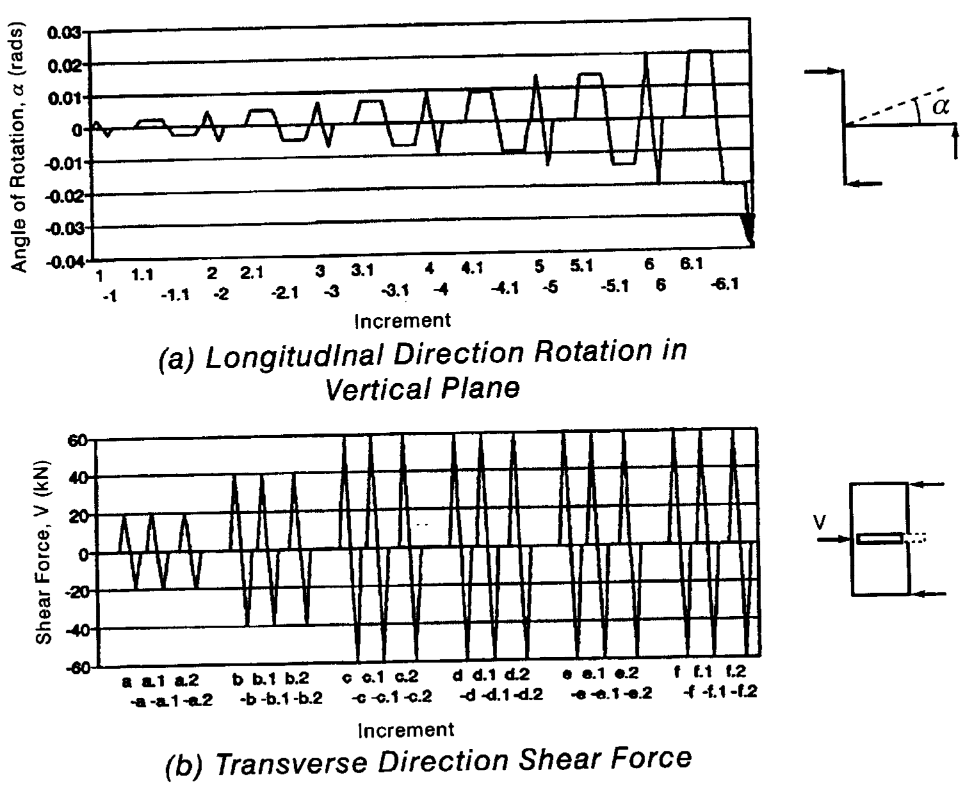
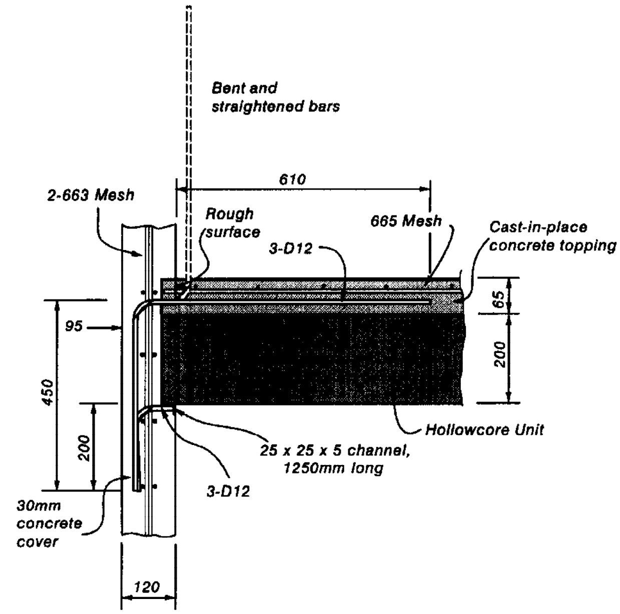
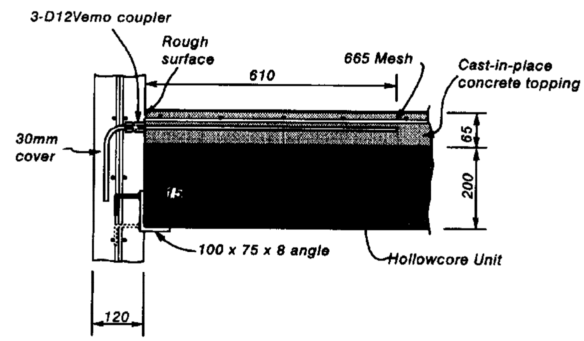


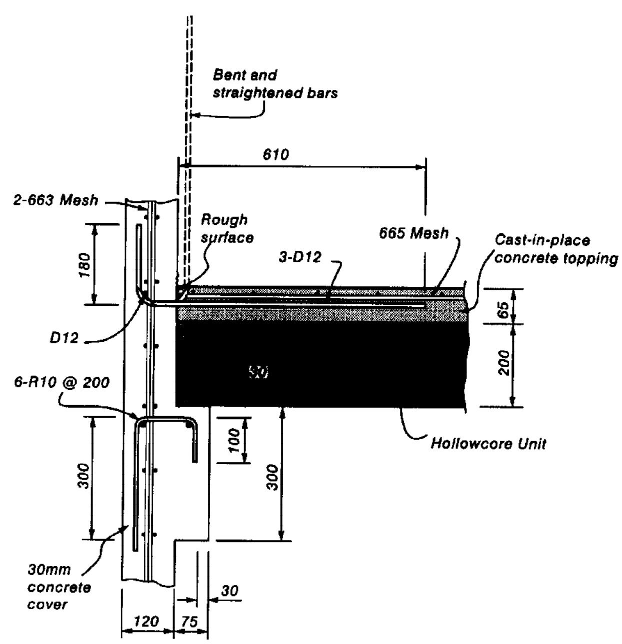
Fig. 3. Loading regime applied to the test units.



(a) Unit WS2



(b) Unit WS3R



(c) Unit WS4

Fig. 4 Details of connections tested.

was imposed to the units when the slab was at the peak of the upward and downward displacements and, also, when it was in the horizontal position.

Fig. 4 illustrates the reinforcing details of the connection region of three units tested in the programme. Of particular interest is to note that in New Zealand the precast floor units are directly seated on the bearing member without a bearing pad. The ledge may or may not be armoured. Another common practice is to cast at least 50 mm of cast-in-place concrete on the top of the proprietary precast concrete floors to achieve the wall-to-slab connection and enable diaphragm action to be developed. No ties are placed between the precast hollowcore unit and the cast-in-place topping. Some cast-in-place concrete, though not very well compacted, usually fills the gap between the wall and the precast floor unit and enables the connection to resist bending moment when subjected to an imposed rotation in the vertical plane.

Fig. 4 (a) shows a connection detail where the wall is cast with a 20 mm recess for seating the hollowcore unit. The bars protruding from the wall towards the slab have limited anchorage in the wall. These bars are cold bent to allow the hollowcore unit to be seated and then straightened. The bars are lapped with reinforcing mesh which is placed prior casting the concrete topping.

According to a proposal for calculating the required embedment length of hooked bars (Restrepo and Park, 1994a), yielding of the starter bars in tension could not be assured with such a short anchorage. The method for calculating the embedment length is based on the ψ-method (Comité, 1994). As a result of tests on the tensile behaviour of small reinforcing bars that are cold bent/straightened in place, Restrepo and Park (1994b) concluded that, under certain conditions and for New Zealand manufactured bars, this practice can be tolerated.

In the connection detail shown in Fig. 4 (b), the hollowcore unit is seated on a steel angle welded to others cast in the wall. The starter bars in the walls for the top bars are spliced at the face of the wall by means of threaded connectors. As in the previous connection detail, a reinforcing mesh is lapped to the starter bars which is placed prior to casting the concrete topping.

Fig. 4 (c) illustrates a connection detail where the hollowcore unit is seated on an unarmored ledge protruding from and cast monolithically with the wall. The bars protruding from the wall towards the slab are cold bent and restraightened as in the connection shown in Fig. 4 (a).

The walls were cast in horizontal position as is typical of tilt-up construction. The region of the wall at the height where the concrete topping on the slab was to be placed was mechanically roughened to an amplitude of less than 5 mm. The end bars protruding from the walls were instrumented a pair of electrical foil strain gauges located at 50 mm from the face of the wall. The walls were to lifted to their vertical position, the hollowcore units were seated on the wall and then the concrete topping was cast. Just before testing, the slab was loaded with lead ingots.

Table 1. Measured properties of the concrete at the time of testing.

TEST UNIT	REGION	f'_c MPa	f_{sp} MPa
WS2	Wall	33.3	3.6
	Topping	26.3	3.4
WS3R	Wall	36.5	3.8
	Topping	21.6	2.5
WS4	Wall	26.4	2.8
	Topping	29.7	2.9

The concrete compressive and splitting cylinder strengths, f'_c and f_{sp} respectively, at the age of testing the connections for the walls and the concrete topping are shown in Table 1. The hollowcore units were purchased from a precast concrete manufacturer where they had been cast with 40 MPa certified compressive cylinder strength at 28 days. The yield strength of the starter D12 (A_s = 113 mm^2) reinforcing bars was 301 MPa.

TEST RESULTS

Figs. 5 and 6 show the moment versus rotation and shear loading versus the relative wall-to-slab shear displacement for Units WS2, WS3R and WS4, respectively.

Moment-Rotation Behaviour

In Unit WS2, a concrete pull-out failure eventually occurred as a result of the hooked bars being in tension because of the imposed rotation. This failure occurred a an applied rotation of 0.013 rads (see Fig. 5 (a)). At a rotation of 0.02 rads. all the protruding bars had pull-out from the wall. By comparison, Units WS3R and WS4 performed very satisfactorily (see Figs. 5 (b) and (c)). Imposed rotations exceeding 0.05 rads. were applied to the units without inducing a pull-out failure. In these two tests the theoretical flexural capacities (M_n) were nearly attained as if the connection had been monolithically cast.

In Unit WS4 the hooked bars were bent in the wall with the hook upwards and this apparently had no effect on the overall behaviour of connection during the test. This is because the force transfer mechanism relies chiefly on the tensile capacity of the concrete and not on a diagonal compression concrete strut as normally occurs in beam-column joints of reinforced concrete frames.

It is worth noting that in the three tests some positive moment resistance developed in spite of the lack of a positive bottom connection. This resistance was due to friction at the seating of the hollowcore unit.

Another point of interest is the relaxation of load that occurred in the tests when the rotation was held and shear loading was applied in the transverse direction. This relaxation became very pronounced in the final cycle of the test (see the moment decrease in the run -f.2 in Fig. 5).

Shear Loading-Relative Wall-to-Slab Displacement

Fig. 6 shows the shear loading versus relative wall-to-slab shear displacement loops recorded during the applied lateral loading. The relative displacement between the wall and the slab were very small during the test. For example, when the imposed rotation was 0.013 rads. the relative shear displacement was of the less than 0.3 mm in the three tests. The units became softer with increase in the imposed rotation in the longitudinal direction. The magnitude of the relative shear displacement is of secondary importance.

CONCLUSIONS

In New Zealand, connections between thin walls and hollowcore floor slabs are achieved by protruding reinforcing bars that are hooked and anchored in the walls. The protruding reinforcing bars are designed to transfer the inertia forces generated at the floor level by a shear friction mechanism. Tension forces, up to yielding, can develop in these bars as a result of the imposed rotation caused by deformation compatibility.

Hooked bars may be anchored in the walls over a projected length of less than 150 mm or 8 times the diameter of the bar providing that a concrete pull-out failure, when the bars are subjected to tension, is avoided.

Interaction between the orthogonal applied loading regimes caused the negative moment of resistance at the connection region to decrease, particularly after the protruding reinforcing bars had yielded under the imposed rotation.

ACKNOWLEDGEMENTS

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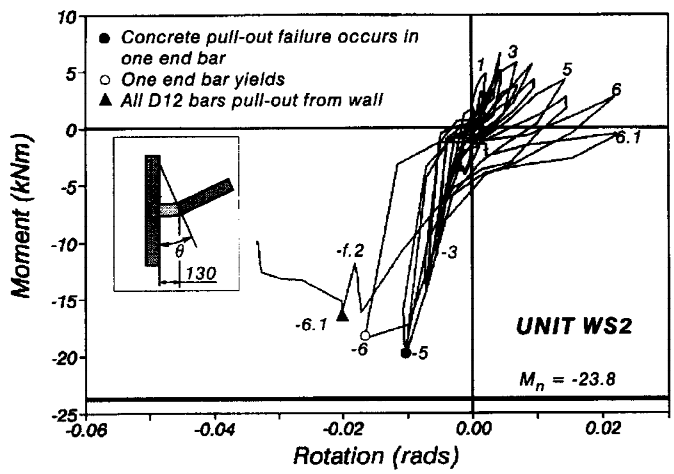
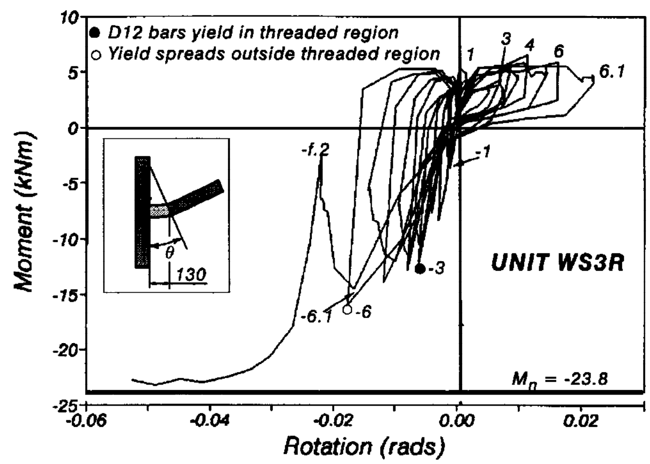


(a)



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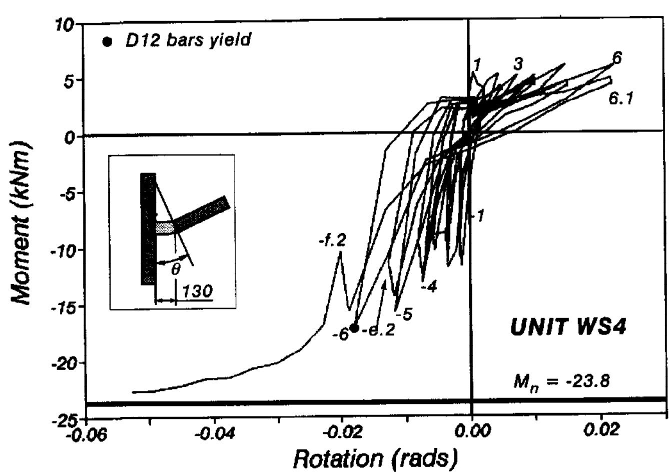
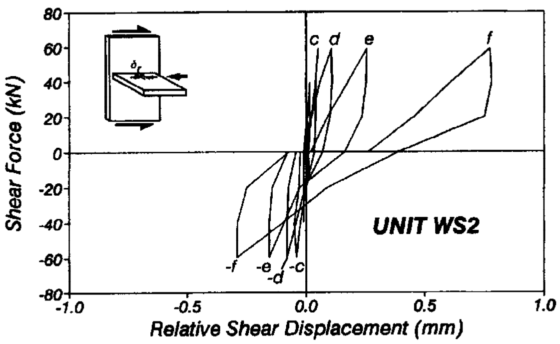


(c)

Fig. 5. Measured moment-rotation relations for the slab at the face of the wall.



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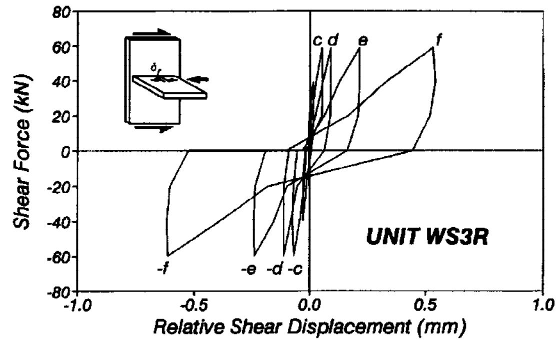

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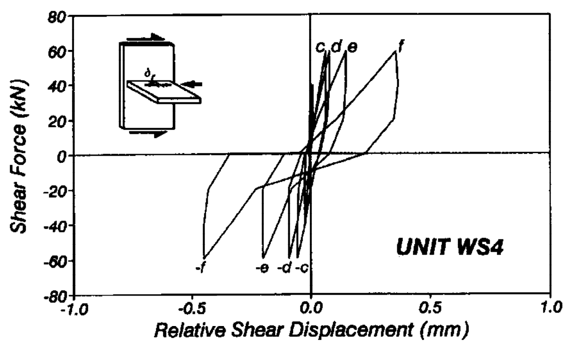

(c)

Fig. 6. Measured wall-to-slab relative shear displacement versus applied shear load when the slab was in horizontal position.