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Seismic behaviour of walls with irregular openings

F.V. Yanez, R. Park & T. Paulay
University of Canterbury, Christchurch, New Zealand

ABSTRACT: Six reinforced concrete walls with openings of different size and arrangement were tested under reversed cyclic lateral loading. The vertical reinforcement ratio was 0.5% in all the walls tested. The results indicate that appropriately designed walls with staggered openings can demonstrate the same behaviour and ductility as walls with regular openings. Methods based on strut and tie models are shown to be viable for the design of reinforced concrete walls with irregular openings under reversed cyclic lateral forces.

1 INTRODUCTION

Reinforced concrete structural walls have generally performed well during strong earthquakes. Walls with irregular openings have been used in some countries in spite of the lack of an accepted code procedure for their seismic design, Wood et al (1987).

The objectives of this present research are to study the seismic behaviour of walls with openings irregularly distributed and reinforced with small or moderate amounts of steel, and to formulate a rational procedure for their seismic design using strut and tie models. Strut and tie models were developed by past investigators, for example Marti (1985) and Schlaich et al (1987), to fulfil the need for improved detailing of reinforcement in disturbed regions of elements of reinforced concrete. Since strut and tie models were formulated for nonreversed load, an attempt to validate this approach for reversed loads is made in the present study.

The results from six three storey reinforced concrete model walls, scaled to one third, tested under reversed cyclic lateral load, are presented. Three of the walls had irregular openings, two had regular openings, and one had no openings. The main variables were the size and arrangement of the openings. The amount of vertical and horizontal reinforcement was the same for the six walls tested. The cross section of all walls was rectangular. No axial load was applied to the walls, since it is usually small in structural walls and generally has a favourable effect on their behaviour, Park and Paulay (1975).

2 DETAILS OF WALLS TESTED

The walls tested had a height to length ratio, h_w/l_w, of 1.25 being 2 m wide, 2.3 m high and 120 mm thick. The wall specimens are shown in Fig.1. Specimen S1 was without openings. Specimens S2, S3 and S5 had 600 x 600 mm openings and specimens S4 and S6 had 400 x 400 mm openings, corresponding to 23.5% and 10.4% of the wall area, respectively. The section of the columns of specimens S2, S3 and S4 were 120 x 200 mm, 120 x 300 mm and 120 x 500 mm, respectively. The section of the beams of specimen S5 was 120 x 250 mm and that of specimen S5 was 120 x 350 mm.

The concrete used had 13 mm maximum size aggregate and its mean strength at 28 days, f'_c, is shown in Table 1. The vertical and horizontal reinforcement in the walls was of deformed steel bars, identified as HD bars, with a measured yield strength of 475 MPa and ultimate strength of 690 MPa. A typical measured stress-strain curve for that steel is shown in Fig.2. As the steel did not have a well defined yield plateau the yield strength was defined as the stress at the 0.2% off-set strain. The hoops of columns and beams were of plain round steel bars, identified as R bars, with a measured yield strength of 350 MPa and ultimate strength of 470 MPa.

The walls contained vertical and horizontal reinforcement as shown in Fig.1. The concrete cover to the horizontal bars, which were placed outside the vertical bars, was 14 mm to the wall faces and 18 mm to the wall ends.

Specimen S1 was designed to represent a wall of

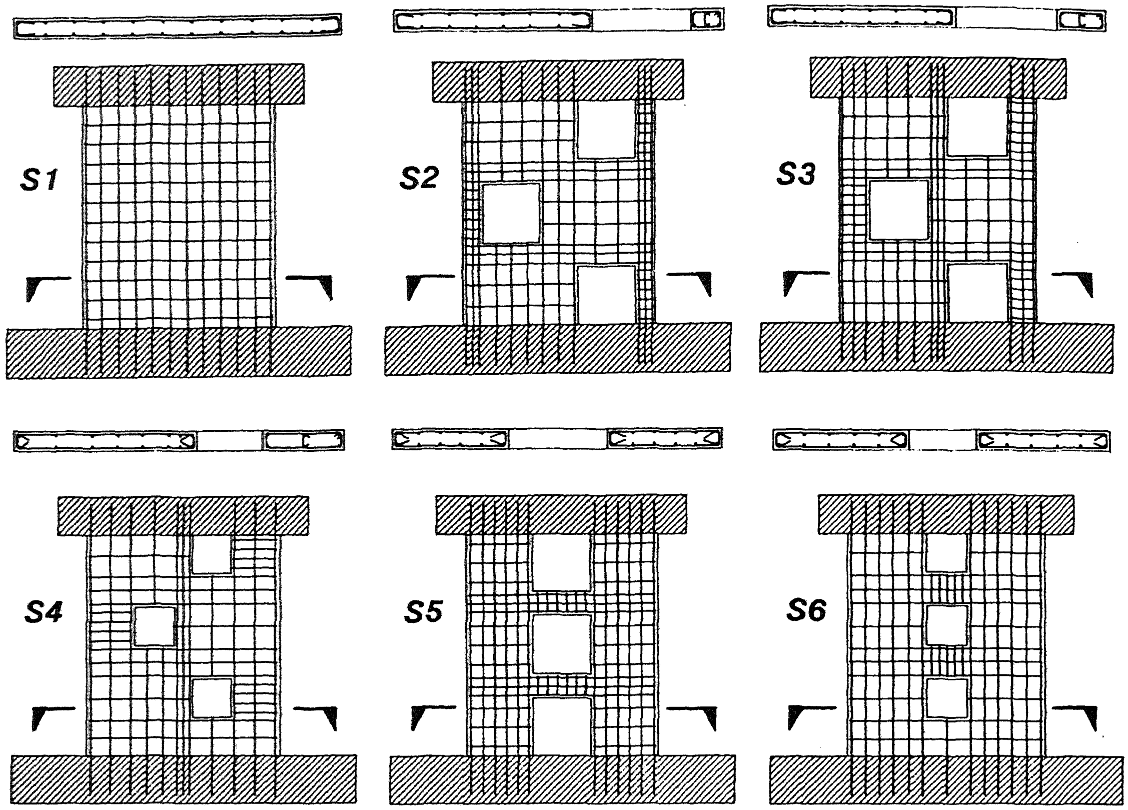


Figure 1. Wall specimens S1 to S6 (cross sections are shown to a larger scale).

limited ductility with a reinforcement ratio in the vertical and horizontal directions of 0.5% and 0.4%, respectively. Specimen S1 had HD8 vertical bars at 180 mm centres and HD8 horizontal bars at 200 mm centres in both faces. All specimens were designed so that the total weight of the reinforcement in each wall was approximately the same, as shown in Table 1. This weight included anchorage lengths at bar ends and hoops in columns and beams, but not the longitudinal bars and hoops in the foundation and loading beams. All the specimens had 24 HD8 vertical bars at the foundation level, except specimen S6 which had 26 bars because two extra bars were added under the bottom opening. The hoops in the columns of specimens S2, S3 and S4 were R6 at 100 mm centres, whereas in the beams of specimens S5 and S6 were R6 at 100 mm and 75 mm centres, respectively.

The theoretical ultimate horizontal load applied at the top of the walls, based on measured steel yield strength, is shown in Table 2. The ultimate load of specimens S1 was determined based on current procedures for cantilever walls.

Specimens S2, S3 and S4 with irregular openings were designed using strut and tie models. Fig.3 shows the models corresponding to the positive and negative directions of loading. The reinforcement was allocated following the tensile load paths in those models. The magnitude of theoretical ultimate horizontal loads, Q, of specimens S2, S3 and S4 was determined from the truss models.

Estimates of the theoretical ultimate loads of specimens S5 and S6 were based on an elastic frame analysis without moment redistribution. The moments of inertia used in the analysis were as follows: $0.9I_g$ for the wall in compression, $0.45I_g$ for the wall in tension, and $0.5I_g$ for the beams, where I_g = moment of inertia of the gross section, Paulay (1986). In this approach it was assumed that the ultimate load was reached when the critical section of the wall (the tension wall) first reached its flexural strength. A limit analysis based on a completely developed plastic mechanism indicated however that the actual ultimate load could be approximately 50% higher than the above estimated values for specimens S5 and S6.

Table 1. Main properties of specimens

Wall	f'_c (MPa)	Weight of reinforcement (kg)	
		Vertical	Horizontal
S1	34	31.0	19.8
S2	23	30.8	20.2
S3	26	30.8	20.5
S4	23	28.8	20.9
S5	44	31.0	22.0
S6	22	31.0	21.3

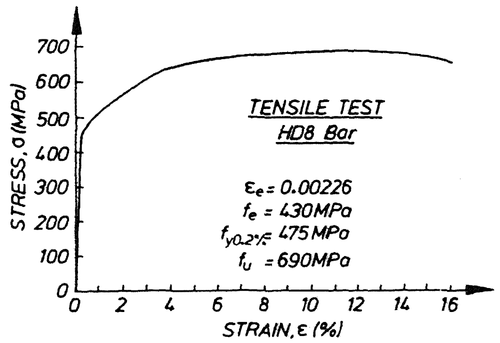
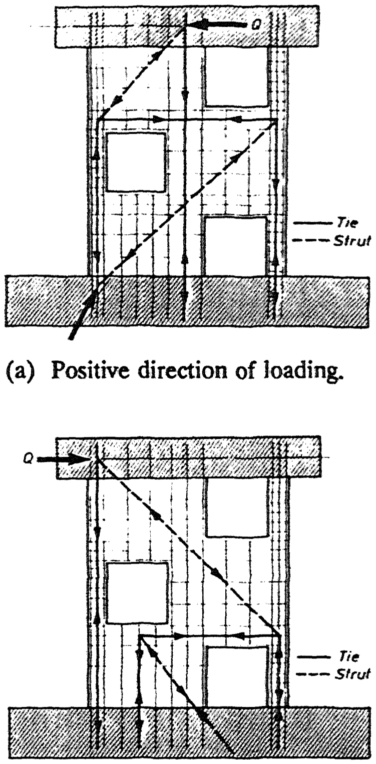


Figure 2. Measured stress-strain curve for HD8 reinforcing steel.



(a) Positive direction of loading.

(b) Negative direction of loading.

Figure 3. Strut and tie models of specimen S2.

3 CONSTRUCTION AND TESTING

The specimens were cast horizontally on the floor and after the concrete had gained strength were lifted and placed in the test rig. Reversed cyclic lateral loads were applied by a hydraulic jack acting at the top of the wall.

The wall specimens were loaded with slow (static) cycles of lateral force, firstly in the elastic range and then with displacement-controlled cycles in the inelastic range. The displacement ductility factor (DF) was defined as the ratio of the lateral displacement at any step to the displacement at yield. The yield displacement, Δ_y, was estimated as 1.33 times the average of the positive and negative displacements corresponding to 75% of the computed maximum ultimate strength of the wall in both directions, respectively, Park (1989). This computed maximum value was evaluated in this particular case using 1.4 times the measured steel yield strength to take into account the strain hardening characteristic of the steel used (see Figure 2). In the displacement controlled cycles the displacements involved in the testing of specimens S2 to S6 were one cycle at displacement ductility factor DF = ± 1, and two cycles at DF = ± 2, ± 3 and so forth until the load carrying capacity of the wall was exhausted. For specimen S1, however, the series was one cycle at DF = ± 1 and two cycles at DF = ± 2.5, ± 3.75 and ± 5 due to an instrument malfunction.

The tests were monitored by a continuous recording of the lateral load versus the lateral deflection at the top of the walls. Deflections and displacements at several points of the specimens and deformations of their inner panels were measured using a set of externally mounted linear potentiometers. Strains in the vertical and horizontal reinforcement were measured at several locations using electrical strain gauges.

4. RESULTS

The measured lateral load versus lateral displacement hysteresis loops for all the specimens are shown in Fig.4 and the main values summarised

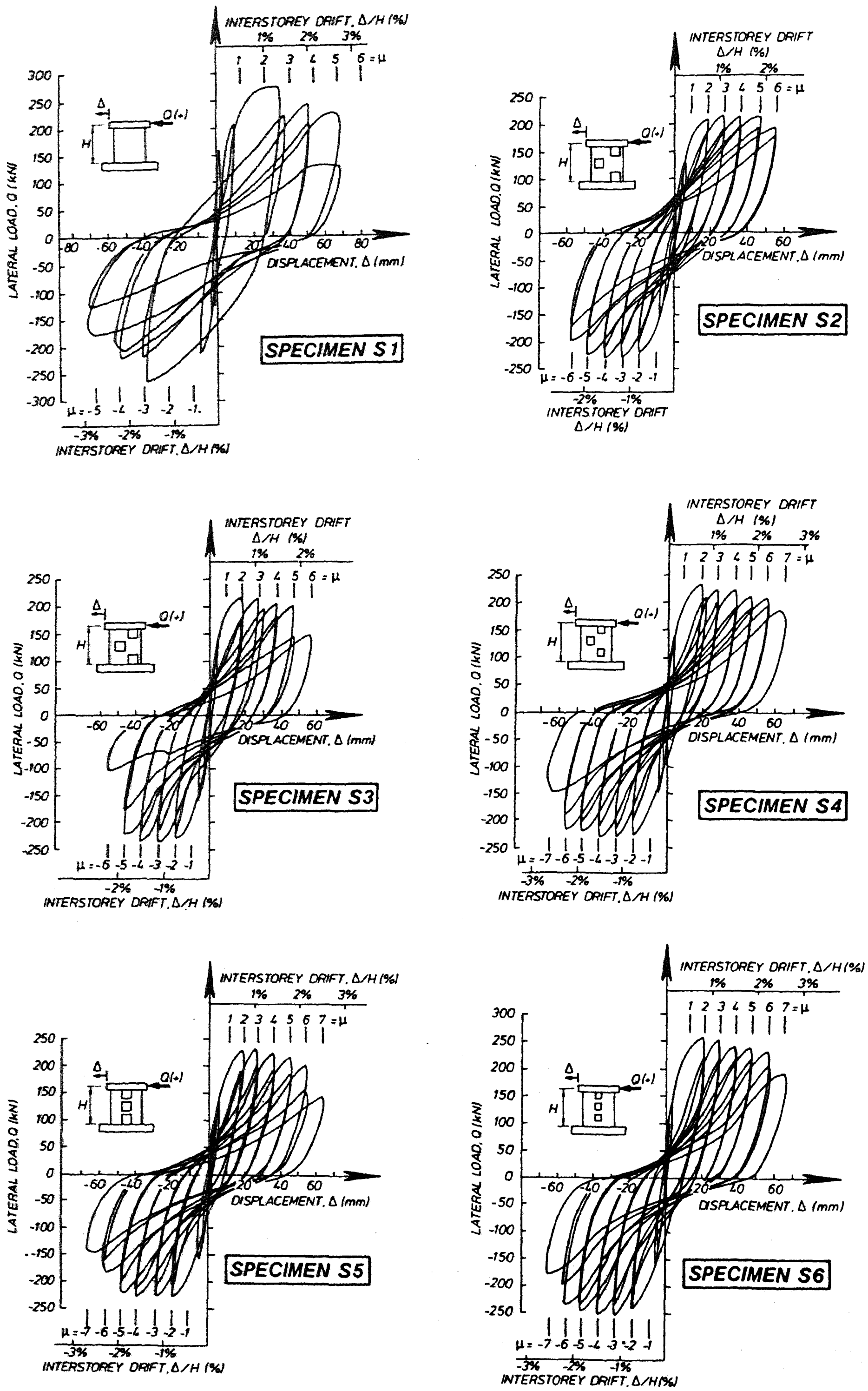


Figure 4. Experimentally observed lateral load versus lateral displacement response.

Table 2. Comparison of experimental and theoretical ultimate horizontal load, ductility and stiffness

Wall	Theoretical ultimate load (kN)		Experimental ultimate load (kN)		Ratio of exp. to theor. ult.load	Experimental maximum displacement ductility	Experimental stiffness kN/mm
	(+) (a)	(-) (b)	(+) (c)	(-) (d)	$\frac{(c)+(d)}{(a)+(b)}$		
1	210	210	287	261	1.30	4.0	37.5
2	139	145	208	228	1.54	6.0	21.3
3	139	140	217	239	1.63	5.0	27.9
4	139	142	235	236	1.68	6.0	36.7
5	141	141	233	227	1.63	6.0	25.9
6	136	136	257	246	1.85	6.0	35.4

in Table 2. All specimens demonstrated ductile behaviour. Eventually failure occurred when the main vertical bars fractured and the compressed concrete crushed at the foundation level. Specimen S2 at DF = -6 is shown in Fig.5.

The behaviour of the columns of specimens S2, S3 and S4 was quite satisfactory. The columns could follow overall displacements without a shear failure. Table 2 shows that specimens S2, S3 and S4 carried a larger ultimate load when the columns in the first storey were under compression. In these cases the columns would have also carried some shear force. When the columns were in tension they would have been incapable of carrying much shear force. The behaviour of the nodes of the struts and ties of specimens S2, S3 and S4 was satisfactory.

Table 2 shows that the stiffness of the specimens S2, S3 and S5 with larger openings, regular or irregular, did not differ appreciably from the average of 25 kN/mm. Stiffnesses of the walls with smaller openings and that of specimen S1 were almost identical with an average of 36.5 kN/mm. Stiffnesses shown in Table 2 were calculated as the ratio of the sum of the positive and negative loads at DF = 1 to the sum of the respective displacements. It appears that stiffnesses were dependent on the size of the openings and not on their arrangement.

5. DISCUSSION OF THE RESULTS

The test results shown in Fig.4 indicate that the size and arrangement of the openings did not have a significant effect on the hysteretic behaviour of the walls. The tests also revealed that the design of the specimens with irregular openings using the strut and tie method was effective. All specimens demonstrated ductile flexural behaviour up to horizontal drift of at least 2%, in spite of anticipated limited ductile response. The hysteresis loops of specimen S1 differed slightly from the loops of the other specimens, mainly because of the different applied load history.

The comparison of experimental and theoretical ultimate loads shown in Table 2 shows that the strut and tie models and the elastic frame analysis predicted the ultimate loads conservatively. Reasons for the conservative prediction of ultimate loads of specimens S2, S3 and S4 by the strut and tie models would have been, firstly, that the reinforcing steel strain hardened rapidly after yield (note the shape of the stress-strain curve in Fig.2 and that the ratio of ultimate strength to yield strength was 1.45) and, secondly, that the node points did not act as pin-joints. For example, the column of specimen S2, instead of acting as a prop, developed flexural strength and thus transferred some shear force. The reasons for the conservative prediction of the ultimate loads of specimens S5 and S6 by eleastic frame analysis would have been the neglect of moment redistribution resulting in the development of a complete plastic mechanism, as pointed out earlier, and strain hardening of the reinforcement.

The inclined strut mechanism by which shear force is transferred to the foundation in the case of walls with irregular openings is suggested by the crack pattern shown in Fig.5 (see also Fig.3b). Compression of the lowest part of the diagonal strut in the bottom panel was always observed in the specimens S2, S3 and S4. The same mechaism was observed when the load was applied in the positive direction.

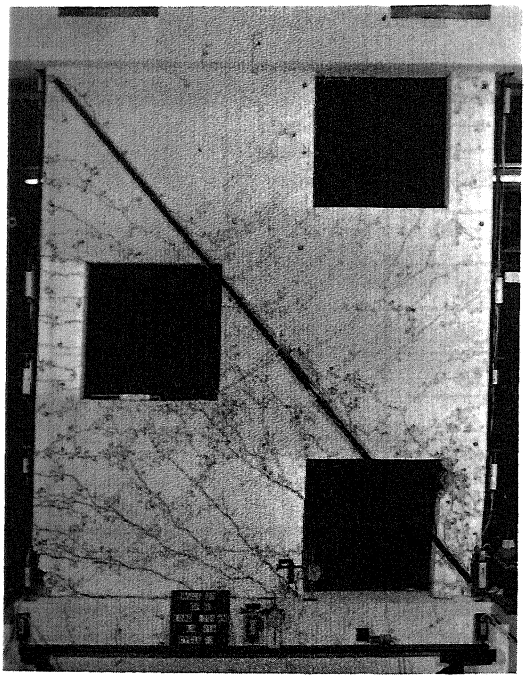


Figure 5. Specimen S2 when loaded in the negative direction at DF = -6.

The behaviour of the nodes of the strut and ties of specimens S2, S3 and S4 was excellent, in spite of the cyclic changes in the directions of stresses (see Fig.3a and b). It is considered that this excellent performance was due to the moderate amount of reinforcement in the walls which induced relatively low concrete compressive stresses in the nodes. These tests indicate that when designing walls with irregular openings, the use of two independent strut and tie mechanisms for lateral loading applied in the positive and negative directions is possible. However, the behaviour of nodes when reinforcement is congested might become critical.

6 CONCLUSIONS

1. The test results from the six wall specimens indicated that the size and arrangement of the openings did not have a significant effect on the behaviour of the walls under cyclic lateral loading. All specimens had a vertical reinforcement ratio of 0.5% and showed ductile behaviour up to a drift of at least 2%.
2. The tests showed that the use of strut and tie models to determine resistance in two directions is valid for the seismic design of reinforced concrete walls with irregular openings.
3. The strut and tie models predicted the ultimate strength of the walls with irregular openings conservatively and with an accuracy equal or better than that of an elastic frame analysis of the walls with regular openings. The strut and tie model appears to be an appropriate method for the seismic design of walls with irregular openings.
4. The stiffness of the walls was dependent on the size of the openings but not on their arrangement. It is suggested that the stiffness of walls without openings can be used for the stiffness of walls with openings smaller than 10% of the wall area.

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