JOINT REINFORCEMENT IN SINGLE WYTHE CERAMIC MASONRY WALLS

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0.0 ABSTRACT

0.1 New Zealand Standard NZS 4230P:1984 Code of Practice for Masonry Design requires that, with the exception of unreinforced veneers, all masonry constructions be reinforced, both horizontally and vertically.

0.2 In single wythe masonry constructions of limited thickness, (90mm), horizontal reinforcement in the form of a thin steel struss configuration laid in the bed joints of the wall, combined with purpose-made knock-out end bricks which accept vertical steel reinforcement, has been tested and is proposed as an acceptable method of providing basketing reinforcement for specific types of masonry walls.

0.3 This paper discusses the performance of bed-joint reinforcement combined with vertical mild steel reinforcement built into single wythe walls when subjected to static and dynamic face load moments and to cyclic in-plane loads.

1.0 INTRODUCTION

1.1 New Zealand, in common with other countries subject to seismic activity, has adopted the principle of the basketing of reinforcement for masonry, based on the practice normal to reinforced concrete.

1.2 Reinforcing experiments in New Zealand (Scrivener, 1969) on ceramic brickwork followed the line of using twin-port 'hollow' bricks to take vertical bar reinforcement in conjunction with bond-beam units encasing horizontal bar reinforcement. Panels built of these components have been tested and were shown to be capable of satisfactorily resisting face loads to the theoretical ultimate flexural capacity of joint reinforced ceramic masonry without loss of integrity between the masonry units.

1.3 More recently, the use of a lattice truss configuration in 4 mm thick joint reinforcement was seen to be a logical solution to the problem of manufacturing (and grouting in) bond-beam bricks. However, the paucity of data on this type of reinforcement prevented its consideration on an acceptable method of construction.

1.4 An 'in-house' test programme at AB Bricks Limited, Auckland (1979-80) showed that under static cyclic loadings, this style of joint reinforcement would indeed enhance the flexural capacity of 90 mm wide ceramic brickwork panels. A parallel development was the use of purpose-made K.O.E. (knock-out end) bricks to encase the vertical steel bars, using bricklayer's mortar as the grout.

1.5 The results of the 'in-house' tests were sufficiently promising to warrant commissioning a laboratory test series. This paper describes the initial tests and those conducted under the auspices of P.A.C.R.A., the Pottery and Ceramics Research Association, Lower Hutt, New Zealand.

2.0 MATERIALS

2.1 Bricks:- AB Bricks Limited metric module 90 mm wide extruded perforated 10 hole and K.O.E. wirecut units;
Golden Buff - face size 230 mm x 76 mm Panels A and B
M4 Reds - face size 230 mm x 90 mm Panels C, D, E, F and G.

2.2 Mortar:- strong mix 1:1:2.5 cement:lime:sand.
- standard mix 1:1:4 cement:lime:sand.

2.3 Joint Reinforcement:- Bricklok - two 4 mm diameter mild steel rods, held 60 mm apart by a welded lattice configuration of 2 mm diameter mild steel wires; galvanized after fabrication (see Fig. 1(a)).

2.4 Vertical Reinforcement:- 10 mm deformed grade 275 steel bars.

3.0 MATERIAL PROPERTIES

3.1 Bricks:- Compressive Strength - Golden Buff 35 MPa
- M4 Reds 23 MPa.

3.2 Mortar:- Compressive strength 28 day cure - strong mortar 29 MPa
- standard mortar 9.5 MPa
- Masonry to mortar bond strengths -

		Mean	Range	c_v
High strength mortar	kPa	745	403	18%
Standard mortar	kPa	794	496	26%

3.3 Joint Reinforcement:- Nominal cross-sectional area = 12.57 mm²
Fig 1(a)
- Yield load 6,031N
- f_y 479.8 MPa
- Ultimate tensile load 6,600N
- Tensile strength (ultimate) 525 MPa

3.4 Vertical Reinforcement:- Nominal cross-sectional area = 78.54 mm²
- Yield load 24711N
- f_y 314.63 MPa
- Ultimate tensile load 33834N
- Tensile strength (ultimate) 430.79 MPa

3.5 Mild steel confining plates - ex 3 mm thick mild steel plate Fig. 1(b)

Fig. 1(a)

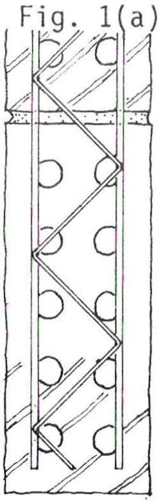
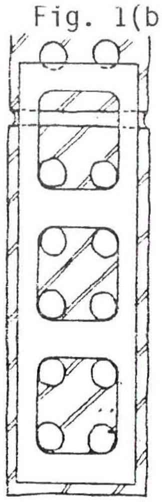


Fig. 1(b)



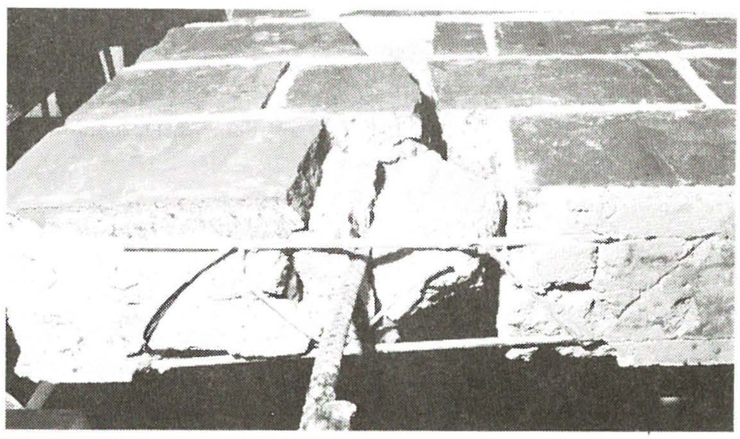


Fig. 2. Panel F viewed from the underside after the racking test, brickwork removed.

4.0 PANEL CONSTRUCTIONS

4.1 All panels tested in this programme were tradesman constructed, using accepted bedding and laying practices for brickwork.

4.2 The points of difference are the use of Bricklok joint reinforcement laid directly on the top of the relevant brick course, the use of bricklayer's mortar to slush and fill around the vertical steel in the K.O.E. ports and the introduction of confining plates in the toes of those panels subjected to racking loads.

5.0 TEST INSTALLATIONS, SCHEMATIC ELEVATIONS.

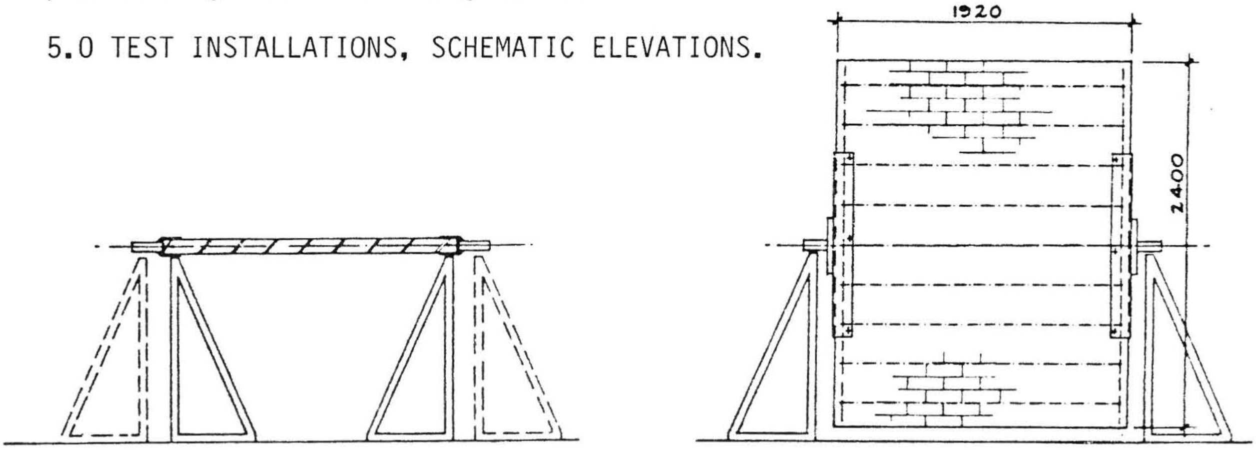


Fig. 3. Static face-loaded test arrangement, Panels A and B.

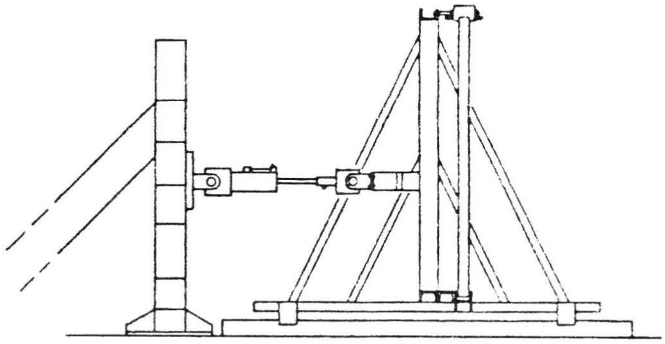


Fig.4. Dynamic face-loaded tests, Panels C and D

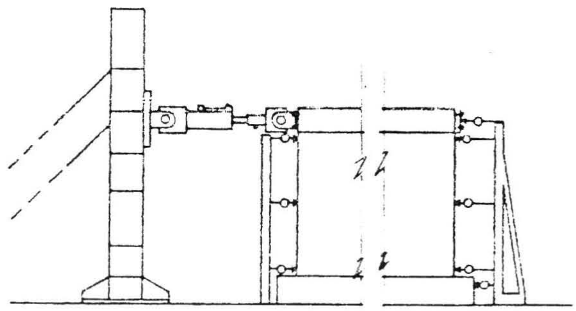


Fig.8. Cyclic in-plane racking tests, Panels E, F and G.

6.0 TEST PROCEDURES

6.1.0 FACE LOADED TEST PANELS, A AND B - STATIC TESTS

6.1.1 Two 'in-house' tests at AB Bricks Limited, (Auckland), Fig. 3, were conducted by building Panels A and B, 2.4 m high x 1.92 m wide. These panels contained Bricklok joint reinforcement at every third course (258 mm centres) with a deformed 16 mm trimmer bar on the long side of each panel. No other reinforcement was used, so that the face loading capabilities of the panels depended entirely on the bond between mortar and masonry and the performance of the joint reinforcement.

6.1.2 One panel was mortared using a 1:1:4 cement:lime:sand mix, while the second, identical in format, used a proprietary admixture in place of lime.

6.1.3 The panels were "pinned" at mid-point, enabling them to be rotated to the horizontal, uniformly supported and loaded with layers of bricks, the deflections recorded, then unloaded and rotated 180°, and the procedure repeated alternately for each side during the test sequence.

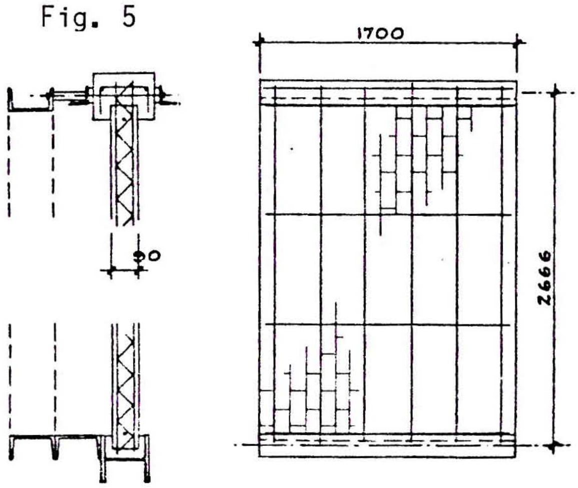
6.1.4 During these tests, cracking occurred initially along the brick mortar interfaces. Additional cracking occurred through the brick panel section after the ultimate theoretical limit had been exceeded. The perceived flexural capacity of the panels was deemed to be sufficiently encouraging to warrant determining the performance of joint reinforcement and K.O.E. bricks in panels subjected to cyclic dynamic loadings.

6.2.0 FACE LOADED TEST PANELS, C AND D - DYNAMIC TESTS.

6.2.1 The dynamic tests were conducted at the P.A.C.R.A. Laboratory, Lower Hutt, Wellington, Fig. 4. Panels C and D, 1.7 m high x 2.6 m wide, were constructed using Bricklok joint reinforcement, spaced every third course (300 mm centres), together with deformed 10 mm bars at every third brick (720 mm centres) vertically; this volume of reinforcement complying with the minimum requirements of NZS 4230P.

6.2.2 Both panels used a cement:lime mortar, with one mix formulated to produce a strength higher than normal.

Fig. 5

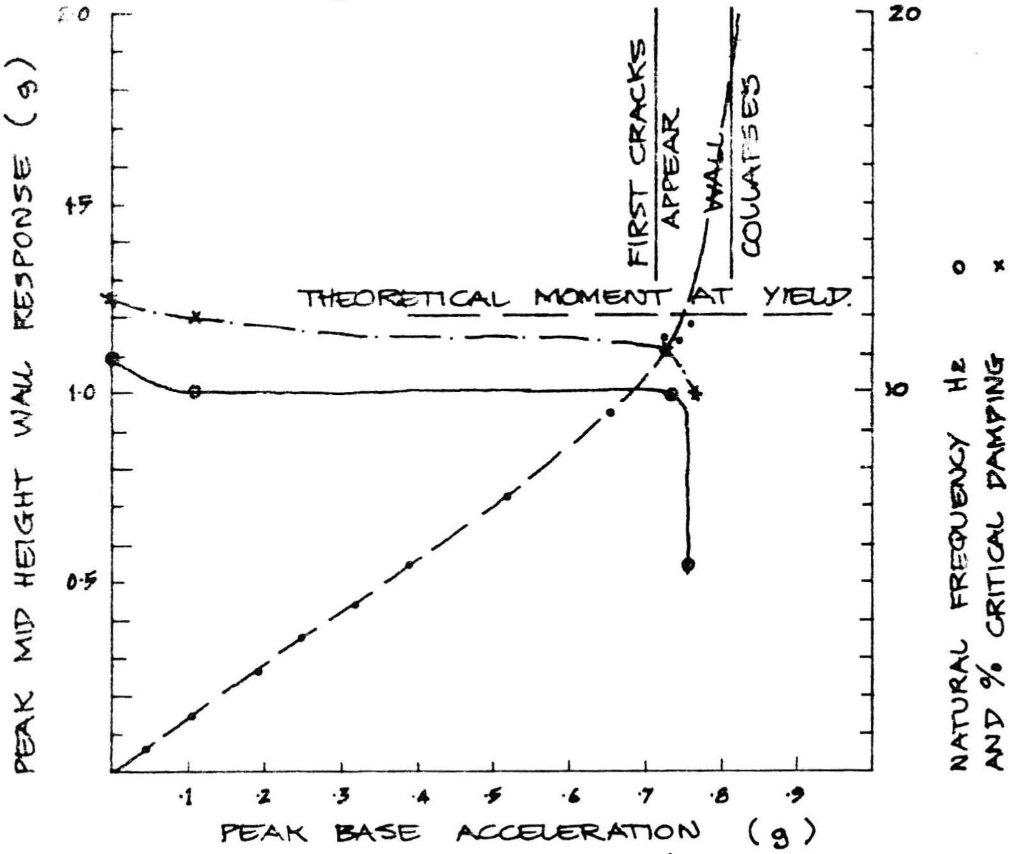


6.2.3 The panels were constructed on their long axes, then subsequently lifted, rotated and aligned into the shaking frame, vertically. The panels were simply supported at top and bottom, Fig. 5. The shaking frame was actuated by a servo-hydraulic mechanism positioned at the mid-height of the test panels, Fig. 4. Strain gauge displacement transducers were located so as to measure the horizontal movement of the panel with regard to the frame, and the base of the shaking frame with respect to the strong floor.

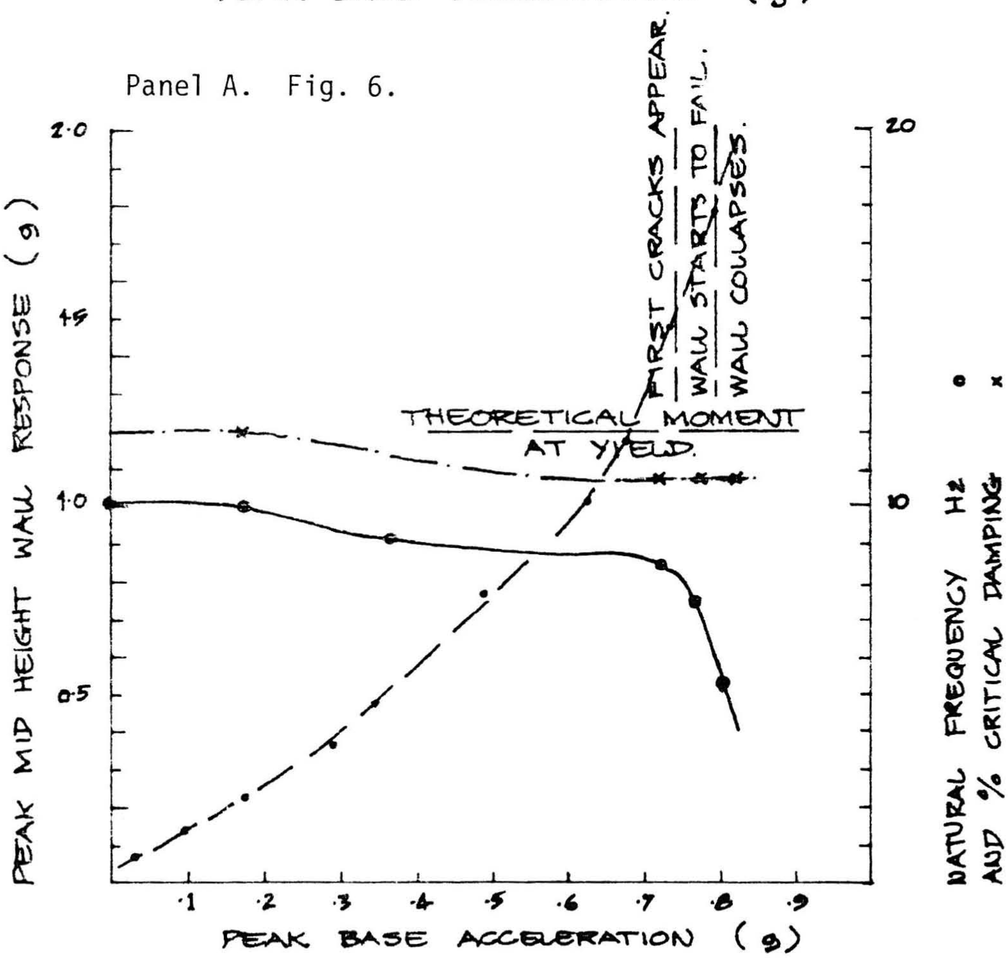
6.2.4 Each panel was shaken using about 20 bursts of sinusoidal accelerations at a frequency of 5 hertz. The initial bursts were of 10-second durations and then reduced to 5 seconds towards the end of the test. The amplitude was progressively increased in magnitude until failure. The natural frequency and damping characteristics of the panel were obtained, at different levels of base acceleration throughout the test, by securely clamping the shaking frame to the strong floor, striking the panel at mid-height and recording the resultant decay response. Further details of the testing procedure are available in ref.7.

6.2.5 The performance of Panels C and D was as shown in Figs. 6 and 7. The failure mechanism was by transverse (across the brick courses) cracking at about the vertical mid-span of the panels, after the yielding of the reinforcing steel, Fig. 11.

Panel B. Fig. 7.



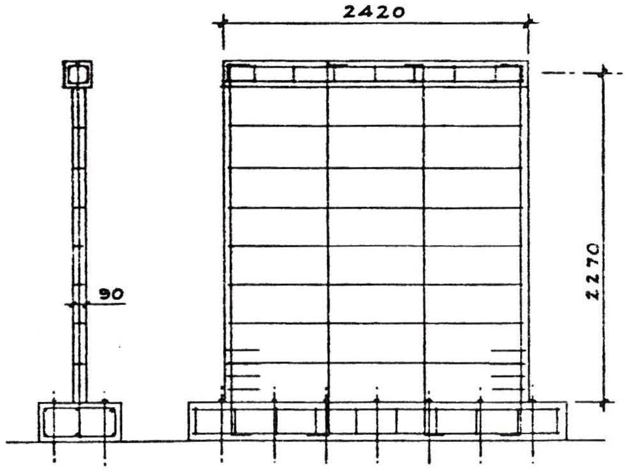
Panel A. Fig. 6.



6.3.0 CYCLIC IN-PLANE RACKING TESTS - PANELS E, F AND G.

6.3.1.0 Fig. 8 shows a schematic view of the test arrangement.

Fig. 9.



6.3.1.1 The configuration of Panel E is shown in Fig. 9. This panel was 2.42 m high x 2.27 m wide, and was constructed using a high-strength mortar testing 29 MPA at 28 days.

6.3.1.2 Panel E was tested under cyclic load control in stages up to the theoretical ultimate load (TUL) capacity, and thereafter by cyclic displacement control to measure different levels of ductility.

6.3.1.3 During this test there were no signs of horizontal cracking in the mortar courses until approximately 75% of TUL had been achieved. The first horizontal crack appeared in the third mortar course from the base, which contained joint reinforcement. Towards the latter stages of the test, a horizontal crack opened up at the base of the panel, together with further cracking of the third mortar bed.

6.3.1.4 Base slip was monitored up to a racking load of ±30 kN at which stage movement was in the order of ±.5 mm. The gauges were then removed to prevent any damage due to unforeseen collapse of the assemblage.

6.3.1.5 In the joint reinforced mortar bed, failure occurred on the lower surface of the mortar, where the joint reinforcement had been laid on the brickwork prior to the mortar being placed. Panel E was subjected to a series of ductility displacement levels and at the last ductility level the panel was sliding extensively on its base.

6.3.1.6 At the end of the test Panel E remained intact, apart from the horizontal cracks at the third mortar course and base. No diagonal cracking was evident.

6.3.2.0 Panel F, Fig. 10, shows the configuration adopted for Panel F. This panel measured 1.95 m high x 3.095 m wide, with a 28-day mortar strength of 29 MPa. Test procedures were similar to those for Panel E.

6.3.2.1 The height to length ratio shown in Panel F was chosen so as to induce a diagonal shear mode failure, hence the inclusion of confining toe plates and the additional starter bars from the base, thereby forcing shear failures into the masonry by reducing sliding of the base, as suggested by Priestley (4,6).

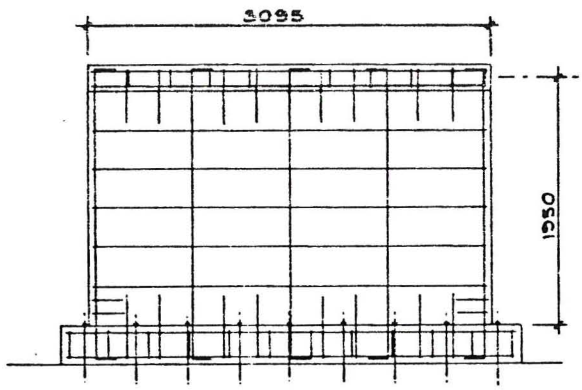


Fig. 10. Panel F details.

6.3.2.2 Cracking appeared in the third mortar course above the base at a load of 30 kN. At a load of 45 kN, a crack appeared in the sixth mortar course above the base, from which diagonal cracks formed to the toe of the panel at both ends at a load of 80 kN.

6.3.2.3 At the theoretical ultimate racking load, vertical cracks appeared in the toe of the panel but were contained by the confining plates. Panel F (Fig. 17) eventually failed by sliding on the sixth mortar course, with lateral movement of the cracked triangular position of the portion above the toe. The confinement plates contained the toe of the panel and allowed high test loads and deflections.

6.3.3.0 Panel G was tested at the Physics and Engineering Laboratory, D.S.I.R., Lower Hutt, Wellington, and was identical to Panel F (Fig. 10) in design, but was constructed using a standard mortar.

6.3.3.1 Panel G was monitored under cyclic displacement control throughout the period of the test. The first horizontal cracks began in the third mortar course from the base at a load level of 30 kN. At a load of 40 kN, a crack occurred in the second mortar course from the base, which propagated the full length of the panel at 60 kN.

6.3.3.2 Failure of the panel occurred along this joint, but unlike Panel F, diagonal cracking did not materialize.

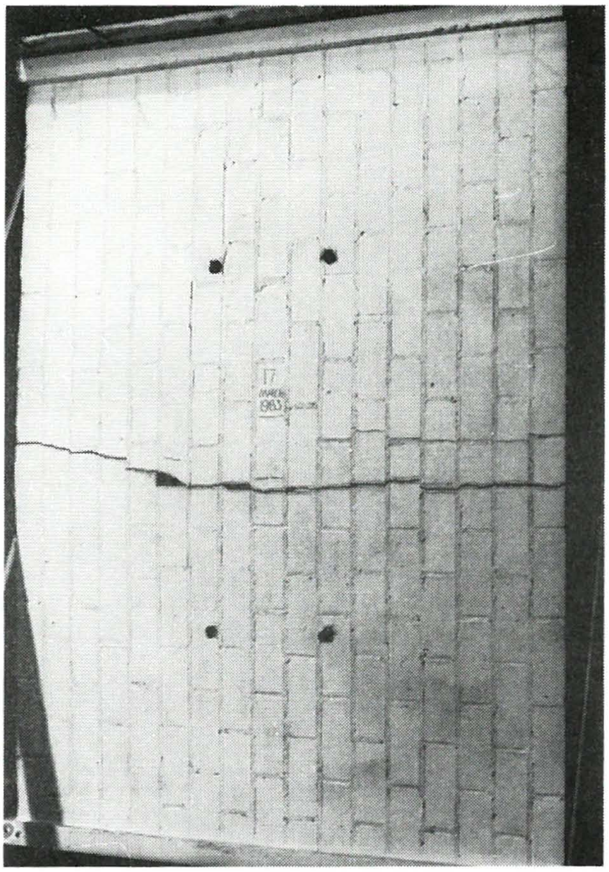


Fig. 11. Panel C, shown during shaking test. Mid-span crack at maximum flexure. Vertical span, 2.6 metres.

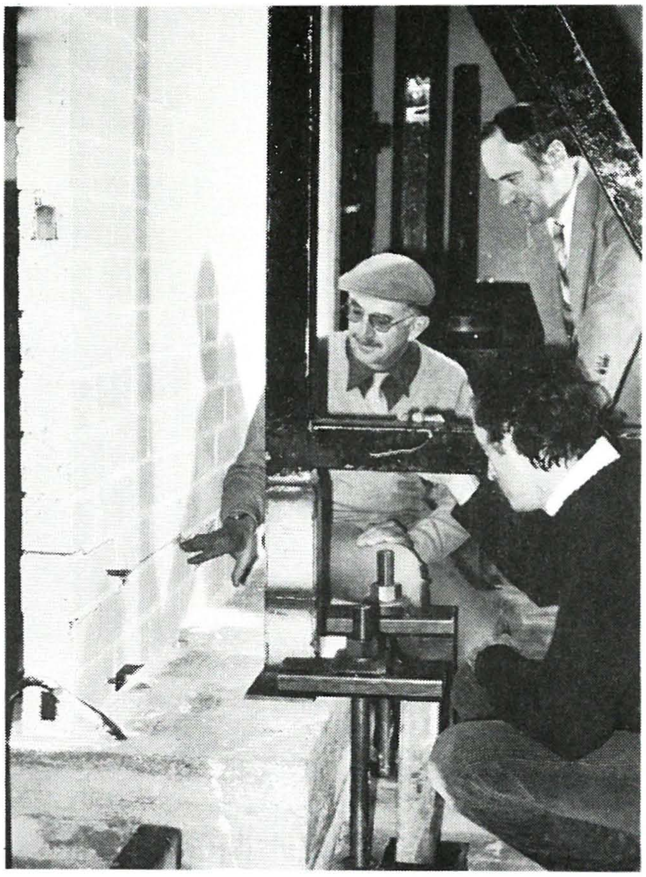


Fig. 12. Panel G being examined at the conclusion of racking test by the authors Messrs Allen & Lapish, together with Mr K. Sullivan, PEL/DSIR.

7.0 DISCUSSION

7.1 New Zealand usage of joint reinforcement and vertical bar reinforcement is governed by the need for compliance with NZS 4230P, which, while not recognising joint reinforcement as an allowable element in resisting shear loads, effectively sets out the parameters for its use.

7.2 The minimum volume area of horizontal reinforcement is .07% of the gross area of the wall, and this determines the joint reinforcement spacing.

7.3 The limitation for vertical steel bars is one-eighth of the gross wall thickness or one-quarter of the least dimension (50 mm) of the flue created by the use of K.O.E. bricks. Deformed 10 mm bars comply in this regard.

7.4 The sum of horizontal and vertical steel (.2143%) in Panels C and D and in Panels, E, F and G, was calculated to exceed the Code minimum of .2%, using a vertical bar spacing of 720 mm centres. This is the maximum spacing permitted under NZS 4230P for structural walls, secondary walls and infill panels in Seismic Zones B and C of New Zealand. It is not anticipated that confinement toe plates will be used in practice in this form of construction.

7.5 Face Loaded Dynamic Tests

7.5.1 The object of the dynamic face loading tests, Panels C and D, was to assess the flexural capacity of joint reinforced assemblages and the integrity of the masonry, Fig. 11.

7.5.2 Both face loaded panels failed in flexure under the dynamically induced loadings. Failure was by cracking at about mid-span of the section, after yielding of the joint reinforcement. The panels failed above their theoretical moment capacity, and remained intact without shedding of masonry.

7.6 Cyclic In-Plane Racking Tests

7.6.1 The principal object of the racking tests was to examine what contribution joint reinforcement might make to the shear capacity of a joint reinforced masonry wall subject to in-plane loading. A second objective was to determine the level of shear stress at ultimate load and check this capacity to the design values permitted by the New Zealand Masonry Design Code.

7.6.2 In Panels E and G subjected to in-plane loadings, cracking occurred mainly along the bedding planes between the joint reinforcement and the brickwork, Fig. 12. Panel F additionally showed typical confined crushing zone failure, Fig. 17. The shear stresses required to create these movements were in excess of that permitted by NZS 4230P for Class B masonry (240 kPa). Apart from a minor contribution the joint reinforcement made in resisting shear in Panel G, joint reinforcement cannot be relied upon to resist in-plane shear loads.

7.6.3 When the panels were tested to a high level of load, the aggregate interlock in the mortar was ineffective and the horizontal in-plane shears were taken by dowel action of the vertical reinforcement.

7.6.4 The degradation of the panels at maximum loads occurred due to slippage over the length of the wall. All three panels maintained their integrity throughout the test and at no stage did collapse or breakdown of the panels eventuate.

8.0 TEST RECORDING

8.1 The laboratory tests were under cyclic load control and the resulting deflections recorded. Loads were increased until the theoretical ultimate capacity of the panels were achieved and the dial gauges measuring the deflections removed. Panels were then loaded under cyclic displacement control where set deflections were maintained under cyclic loadings, to examine wall ductilities. A number of different displacement controlled deflections were examined and cycled for at least four cycles at each deflection, simulating ductilities of between 1.5 and 2.5.

8.2 Test loads were adjusted to ensure that deflection at each peak was maintained. The hysteresis loops recorded the fall off in load at each peak. Examples are shown in Figs. 13 to 15.

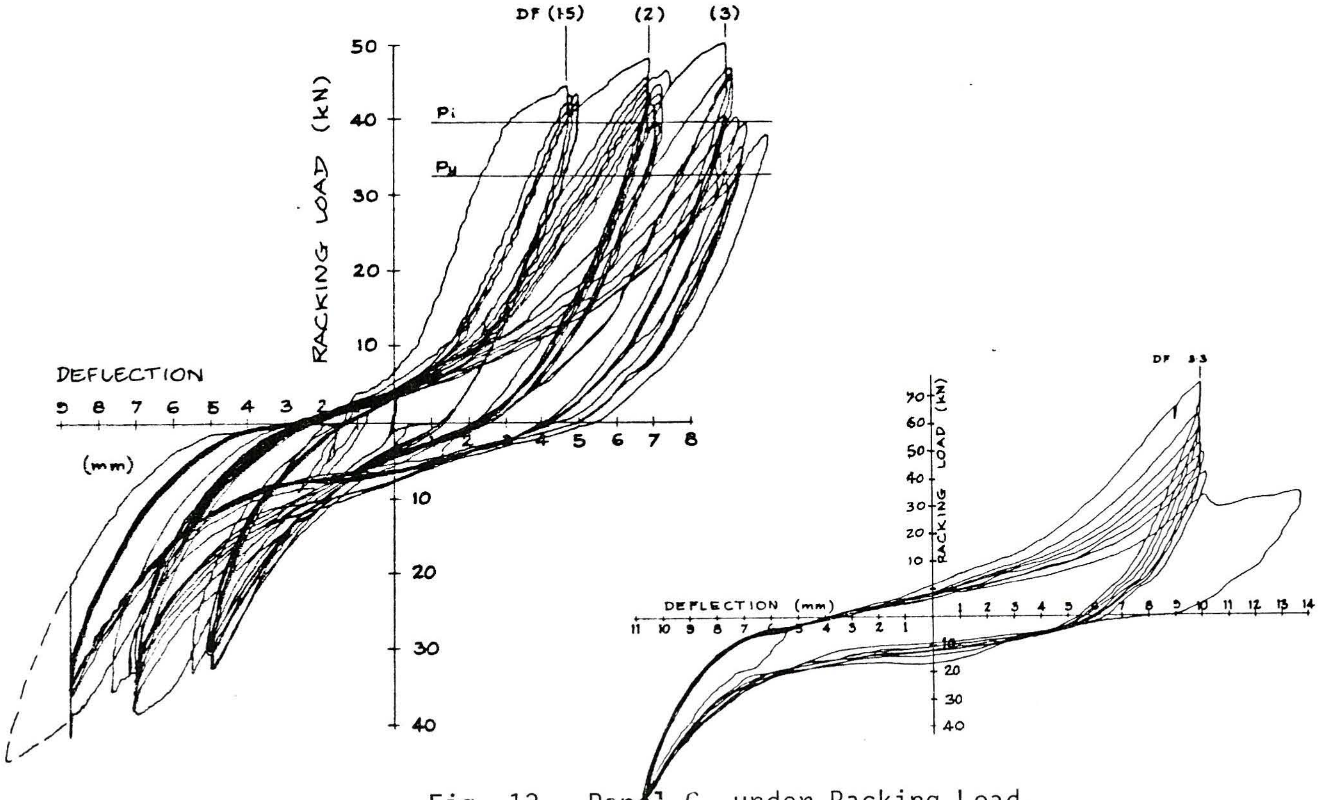


Fig. 13. Panel C under Racking Load.

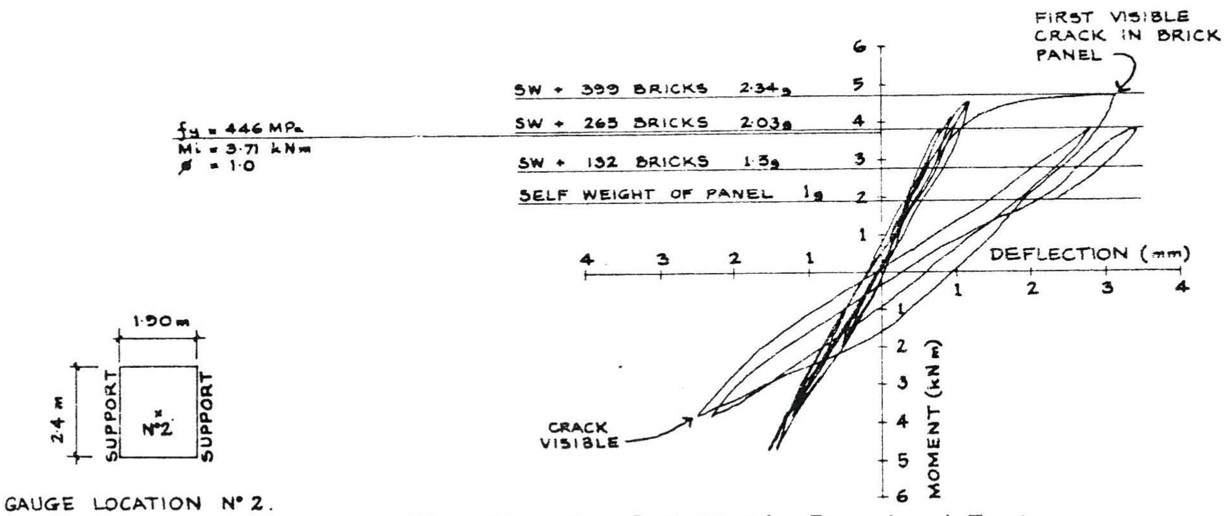


Fig. 14. Panel B Static Face Load Test.

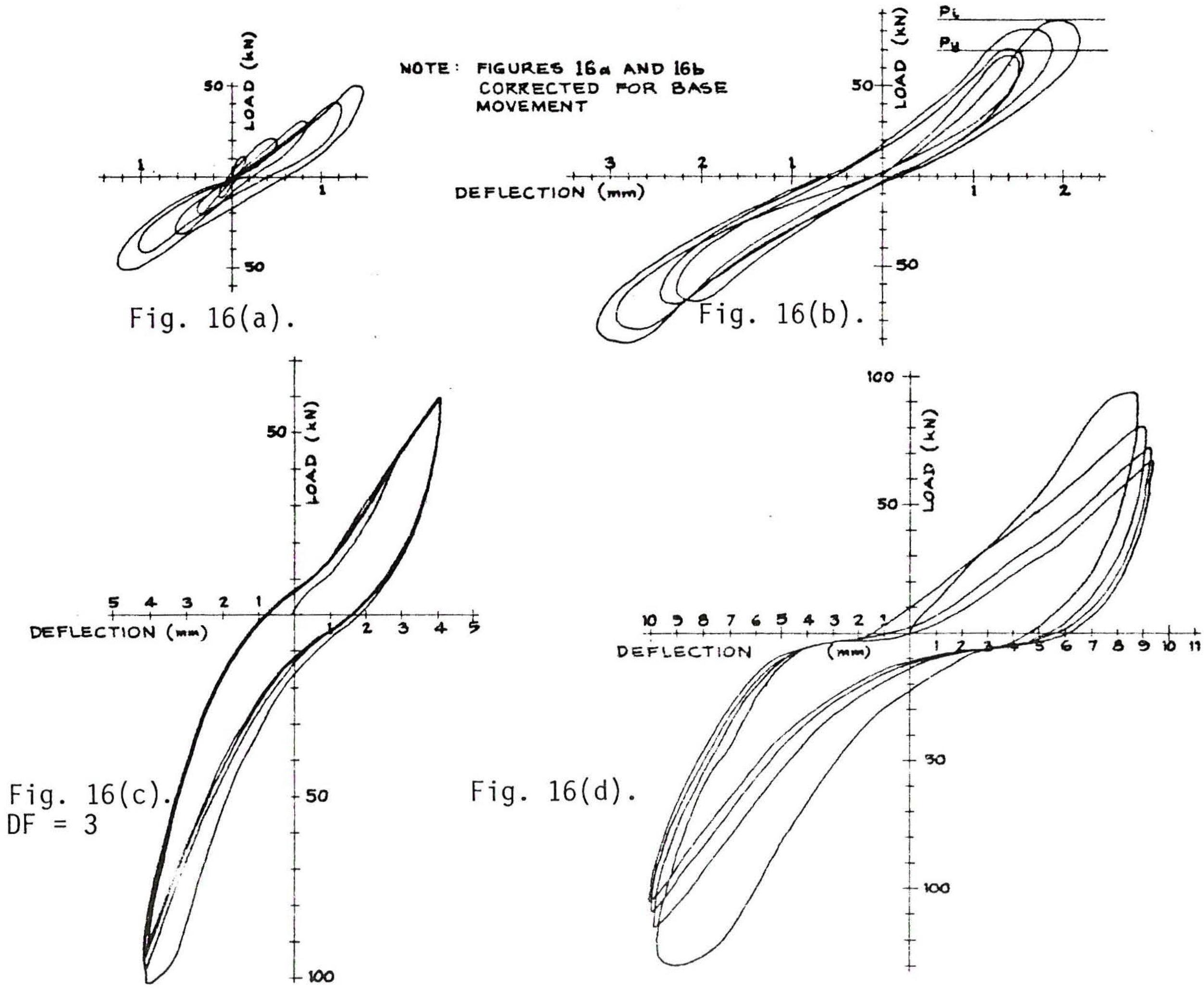


Fig. 16(a).

Fig. 16(b).

Fig. 16(c).
DF = 3

Fig. 16(d).

Fig. 16. Panel G Under Racking Load.

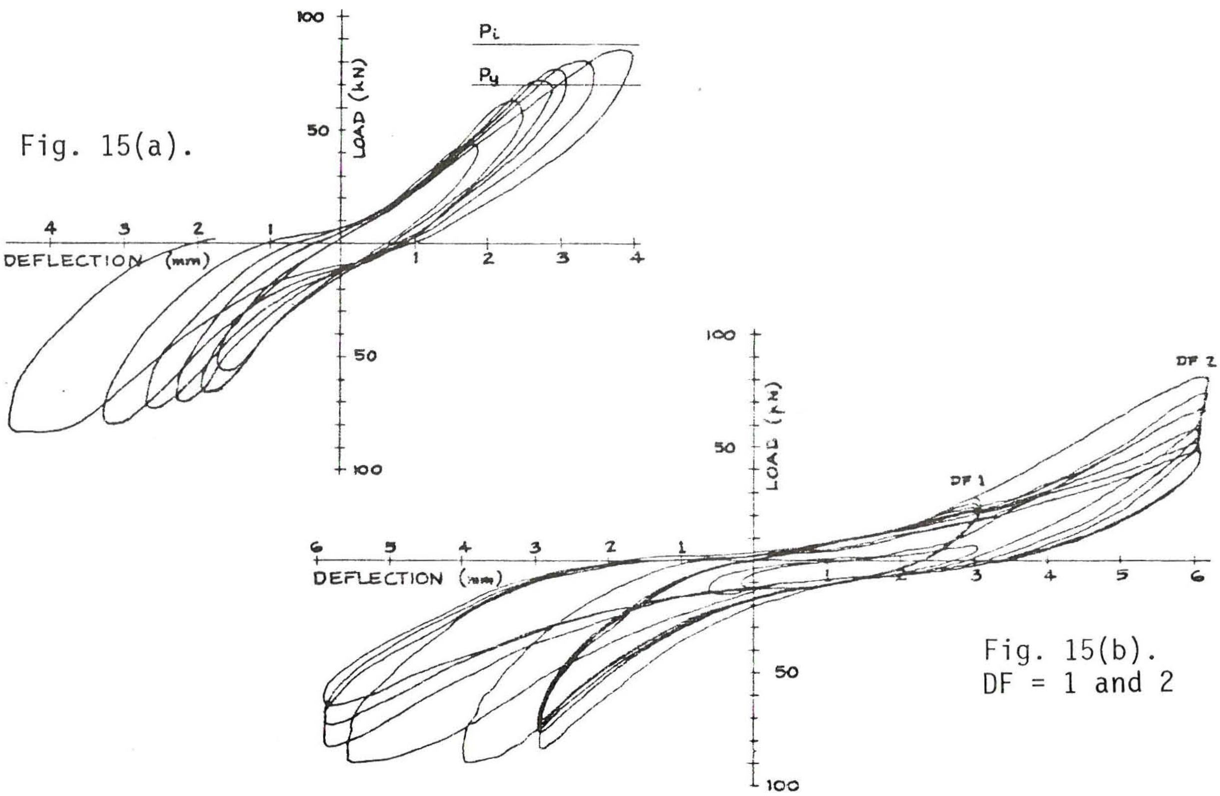


Fig. 15(a).

Fig. 15(b).
DF = 1 and 2

Fig. 15. Panel F Under Racking Load.

9.0 THEORETICAL BEHAVIOUR UNDER RACKING LOADS

9.1 The ultimate flexural load capacity which is required at first yield in the extreme tension bars has been calculated, assuming the following material properties:

Steel modulus of elasticity 207 GPa
Brick prism compressive strength 20.7 MPa
Steel yield stress of vertical steel 275 MPA.

9.2 Normal reinforced concrete design equations were used to calculate the loads with a capacity factor of 1.

TABLE ONE - THEORETICAL LOADS

		Panel E	Panel F	Panel G
Load for first yield	kN	32.5	70	70
Shear stress at first yield	kPa	192.2	319	319
Ultimate flexural load	kN	39.5	88	88
Shear stress at ultimate flexural capacity	kPa	283.5	492	492

TABLE TWO - MAXIMUM TEST LOADS

		Panel E	Panel F	Panel G
Maximum flexural loads	kN	50	85	130

10 CONCLUSIONS

10.1 The reinforcement pattern achieved by using joint reinforcement in the bed joints of ceramic brick panels, allied with vertical steel encased in the ports of knock-out end bricks, requires no additional skills on the part of the bricklayer tradesman.

10.2 The assemblages described are susceptible of structural analysis. The face loaded panels maintained their integrity beyond their predicted ultimate flexural capacity. The racked panels resisted shear stresses greater than the 240 kPa permitted for Class B masonry under the New Zealand Masonry Design Code, again without loss of integrity.

10.3 Joint reinforced single wythe ceramic brickwork walls designed in compliance with NZS 4230P must be designed to respond elastically because of N.Z. Codes earthquake criteria, even though test results indicate that little damage resulted, apart from mid-span section cracking under dynamically induced face loads and horizontal sliding of mortar courses at the bottom of racked test panels.

10.4 End use options may therefore include reinforced veneers; infill panels in framed buildings; one-storey high load-bearing buildings and cavity brickwork structures.

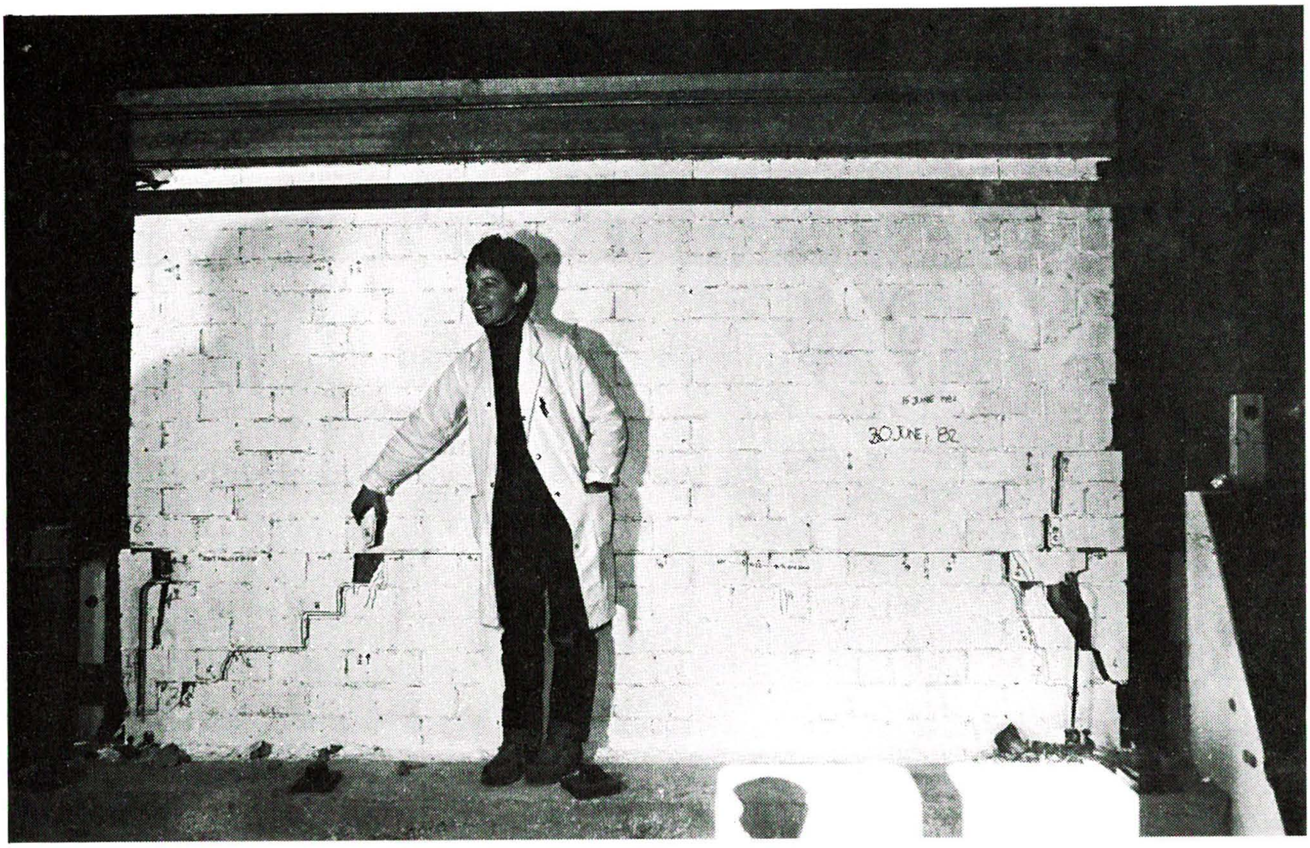


Fig. 17. Panel F at conclusion of racking test, showing confined crushing zone failure at toe of the panel.

REFERENCES

1. NZS 4230P Code of Practice for Masonry Design.

2. New Zealand Pottery and Ceramics Research Association Report A45 - Structural Test Equipment at PACRA.

3. Scrivener, J.C. - Face Load Tests on Reinforced Hollow-brick Non-load-bearing Walls - New Zealand Engineering - 15 July 1969.

4. Priestley, M.J.N. and Elder, D. McG.- Seismic Behaviour of Slender Concrete Masonry Shear Walls - March 1983 - Department of Civil Engineering - University of Canterbury.

5. Priestley, M.J.N. and Bridgeman, D.O.- Seismic Resistance of Brick Masonry Walls - Bulletin of N.Z. National Society for Earthquake Engineering Vol. 7 No. 4 December 1974.

6. Priestley, M.J.N.- Seismic Resistance of Reinforced Concrete Masonry Shear Walls with High Steel Percentages - Bulletin of N.Z. National Society for Earthquake Engineering Vol. 10 No. 1 March 1977.

7. Priestley M.J.N., Thorby P.N., McLarin M.W., Bridgeman D.O. - Dynamic Performance of Brick Masonry Veneer Panels - Bulletin of N.Z. National Society for Earthquake Engineering, Vol. 12 No. 4 December 1979.