THE USE OF STEEL REINFORCEMENT SYSTEMS TO IMPROVE THE STRENGTH AND STIFFNESS OF LATERALLY LOADED CAVITY BRICK WALLS

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ABSTRACT A vertical shear connector suitable for manufacture by automated sheet metal processes from steel coil is developed in an experimental program involving 90 small area bearing tests, 18 shear tests and 6 tension tests. A comparison of three full scale wall panel tests, (1) standard cavity wall, (2) cavity wall with an unbonded intermediate steel mullion (76x51x3.2 R.H.S.), (3) cavity wall with a vertical shear connector, demonstrate that the connector significantly increases both the post and pre-cracking strength and stiffness under out-of-plane loads.

1. INTRODUCTION

The strength of a cavity brick wall subjected to out-of-plane loads such as wind and earthquake loads is limited.

The ability of the wall to span horizontally between integral return walls is restricted by (a) the wall capacity, (b) the inclusion of windows and doors, (c) the trend for open planning with large distances between return walls and (d) the use of vertical joints for crack control.

The capacity of an unreinforced wall to span vertically between floors or floor and roof structure is shown in Figure 1. Australian codes of practice (Ref. 3) specify that a low rise building on a non-exposed site in a non-cyclonic area such as Brisbane be designed for a wind velocity of 33 m/s (W33). Depending upon building geometry and permeability this results in a design wind pressure ranging between 0.6 kPa inwards to 0.9 kPa outwards. A vertical span of 2.4 m is permissible therefore in normal cavity brick walls only in the most sheltered situations.

The objectives of the experimental program presented in this paper were to:- (a) investigate a cavity brick wall with standard dual limb ties, (b) investigate a cavity wall reinforced with an unbonded intermediate steel mullion (R.H.S.)*[1] in the wall cavity (Figure 2a), (c) develop and investigate a vertical shear connector*[2] (Figure 2b) to achieve shear transfer between the two leaves of the wall prior to cracking and function as bonded reinforcement for increased post-cracking strength and stiffness.

Previous work has been done in Australia on an intermediate steel mullion by Lawrence (Ref. 2) who concluded that a 150 U.C.*[3] built into the leaf of a cavity brick wall has (a) little effect on the behaviour of the wall before cracking of the brickwork and (b) a major influence in determining the ultimate load capacity.

NOTES: *[1] Rectangular Hollow Section.
*[2] Patent Application No. PG5960 by Uniquest, University of Queensland.
*[3] Universal Column.

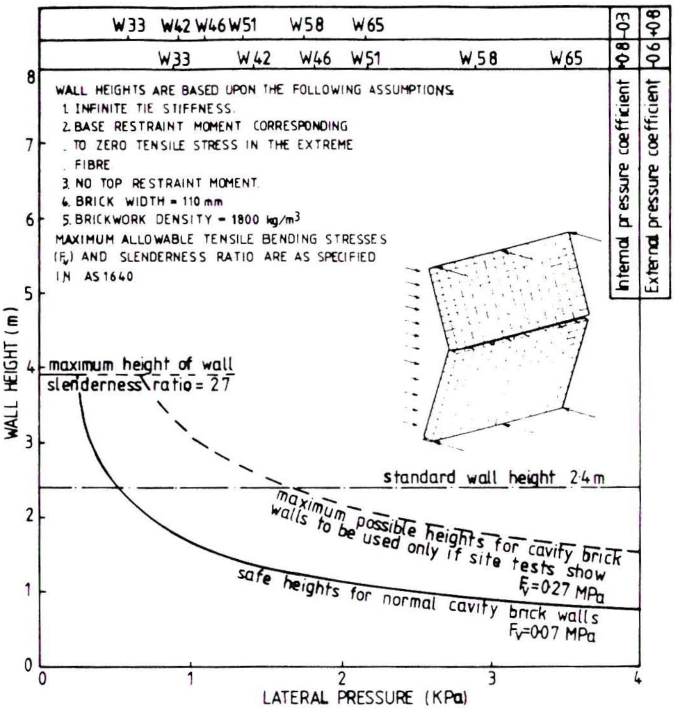
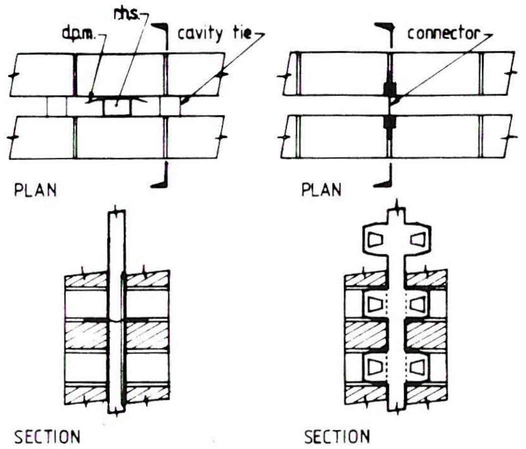


FIGURE 1 – VERTICAL SPANNING CAPACITIES.



(a)-INTERMEDIATE STEEL MULLION (b) VERTICAL SHEAR CONNECTOR

FIGURE 2 – VERTICAL REINFORCEMENT SYSTEMS.

2. EXPERIMENTAL PROGRAM

2.1 Materials

The range of parameters was limited to a single brick and 1:1:6 mortar.

2.1.1 Bricks. The extruded clay bricks used in the investigation were nominal 230 mm x 110 mm x 76 mm depth with three 40 mm diameter extrusion holes. Their physical and mechanical properties were determined in accordance with AS1226 (Ref. 4)*1 and were as follows:-

(a) Physical Properties. Average actual dimensions of 24 bricks were 233 mm x 114 mm x 77 mm depth with 41 mm diameter extrusion holes; average absorption coefficient was 10.5% with N*2 = 6, S*3 = 0.28%; slight efflorescence with no spalling; initial rate of absorption was 2.5 $kg/m^2/min$ with N = 6, S = 0.45 $kg/m^2/min$.

(b) Mechanical Properties. Average brick compressive strength was 30.9 MPa with N = 12, S = 2.46 MPa, C*4 = 26.5 MPa; average modulus of rupture was 1.07 MPa with N = 12, S = 0.45 MPa, C = 0.26 MPa.

2.1.2 Mortar. The mortár used was batched in the proportions and by the methods shown in Table 1. Sieve analyses for sand batches A & B are shown in Figure 3. Any mortar remaining after 90 minutes was discarded. Retempering was permitted. Control specimens (70x70x70 mm) were moist cured and tested under axial compression in accordance with AS A123 (Ref. 5) at 28 days and the results are shown in Table 2.

NOTES: *1 AS = Australian Standard
*2 N = Number of Samples
*3 S = Standard Deviation
*4 C = Characteristic Strength based upon a 95 percentile.

Mortar Designation	Cement	Lime	Sand	Water	Sand Batch	Batching Method
S	1	1	6	3.3	A	by Volume
L	1	1	6	2	B	by Weight

Table 1 - Mortar Mix Properties by Volume

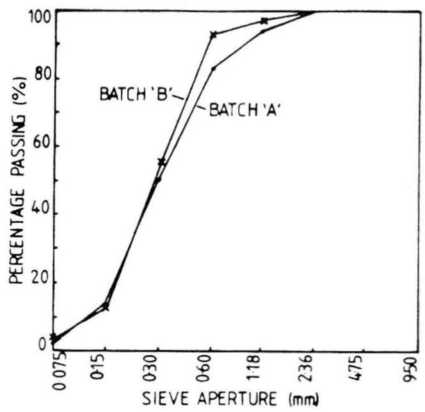


FIGURE 3- SIEVE ANALYSES.

Test	Mortar Designation	Average Ultimate Compressive Stress (MPa)	Number of Specimens	Standard Deviation (MPa)
Brickwork compression, Small area bearing series 1.	S	4.3	6	0.1
Tab shear series 1, Small area bearing series 2.	S	5.7	10	1.3
Tab shear series 2.	S	6.0	9	1.0
Standard cavity wall	L	10.1	31	0.8
Intermediate steel mullion wall	L	8.9	20	0.6
Vertical shear connector wall	L	8.9	18	0.6

Table 2 - Mortar Compressive Strengths

2.1.3 Brickwork

(a) Compressive Strength. Four-high brickwork piers constructed with type S mortar and tested under axial compression in accordance with AS1640 (Ref. 1) at 28 days had an average ultimate compressive strength of 12.9 MPa with N = 6, S = 1.0 MPa.

(b) Bond Strengths. Bond strengths were determined by (1) the bond in bending test on 9-high brick piers similar to AS1640 and (2) the 'bond wrench' method developed by the B.D.R.I.*[1]. Bond strength values are shown in Table 3. Bond strength values were obtained from the standard cavity wall after the wall had failed under out-of-plane loading - by progressively removing the perpend mortar with non-percussion high speed drills and measuring the bond strength of the bed joint with the 'bond wrench'. The bond in bending tests were performed in a compression testing machine with the brick beam supported on self-aligning rollers at 700 mm centres and load applied by a central line load mounted on a spherical seat. The average modulus of elasticity of the beams at 56 days was 4.9E3 MPa with N = 15, S = 1.1E3 MPa

NOTES: *[1] BDRI = Brick Development Research Institute, Australia.

Bond Strength Test Method		Standard Cavity Wall	Intermediate Steel Mullion Wall	Vertical Shear Connector Wall	
AS1640 - Bond in Bending					
(a) 9 high piers	A	56	56	7	56
	N	6	6	2	3
	$\bar{X}$	0.59	0.77	0.97	1.08
	S	0.25	0.16	-	-
	C	0.09	0.45	-	-
B.D.R.I. - Bond Wrench					
(a) Remnants of bond in bending piers	A	56	56	7	56
	N	40	41	21	21
	$\bar{X}$	0.78	0.84	0.94	0.98
	S	0.26	0.17	0.25	0.21
	C	0.36	0.56	0.51	0.62
(b) 4 high piers					56
					12
					1.13
					0.20
					0.77
(c) Remnants of test wall	A	~ 60	A = Age at test (days)		
	N	60	N = Number of tests		
	$\bar{X}$	0.89	$\bar{X}$ = Average ultimate bond strength (MPa)		
	S	0.37	S = Standard deviation (MPa)		
	C	0.29	C = Characteristic strength based on a 95 percentile (MPa)		

Table 3 - Bond Strengths

2.2 Tab Development Tests

The vertical shear connector necessitates a tab which is built into the brickwork and has sufficient capacity to transfer longitudinal (i.e. vertical) shear forces in the plane of the wall and out of plane tension and compression forces. Tests undertaken in the development of a possible tab are described in the following.

2.2.1 Small Area Bearing Tests. A total of 90 small area bearing specimens consisting of a steel bearing plate embedded in mortar on a half brick were tested with the half brick mounted on 3 mm plywood in a compression testing machine. Load was applied through a spherically seated loading head directly onto the steel bearing plate. Two series of tests were performed. In series 1 bearing plates were located at a corner of the brick while in series 2 bearing plates were located at an edge and remote from a corner. Ultimate bearing stress based upon the area of the steel bearing plate is plotted against mortar thickness in Figure 4. Predominantly specimens failed by vertical splitting of the brick.

2.2.2 Tab Shear Tests. Five-course high cavity brick walls were constructed with one tab built into the central perpend in the second course of each leaf. The walls were cured under plastic sheeting in the laboratory for 28 days then tested as shown in Figure 5. Tab shear force was assumed to equal half the jack force

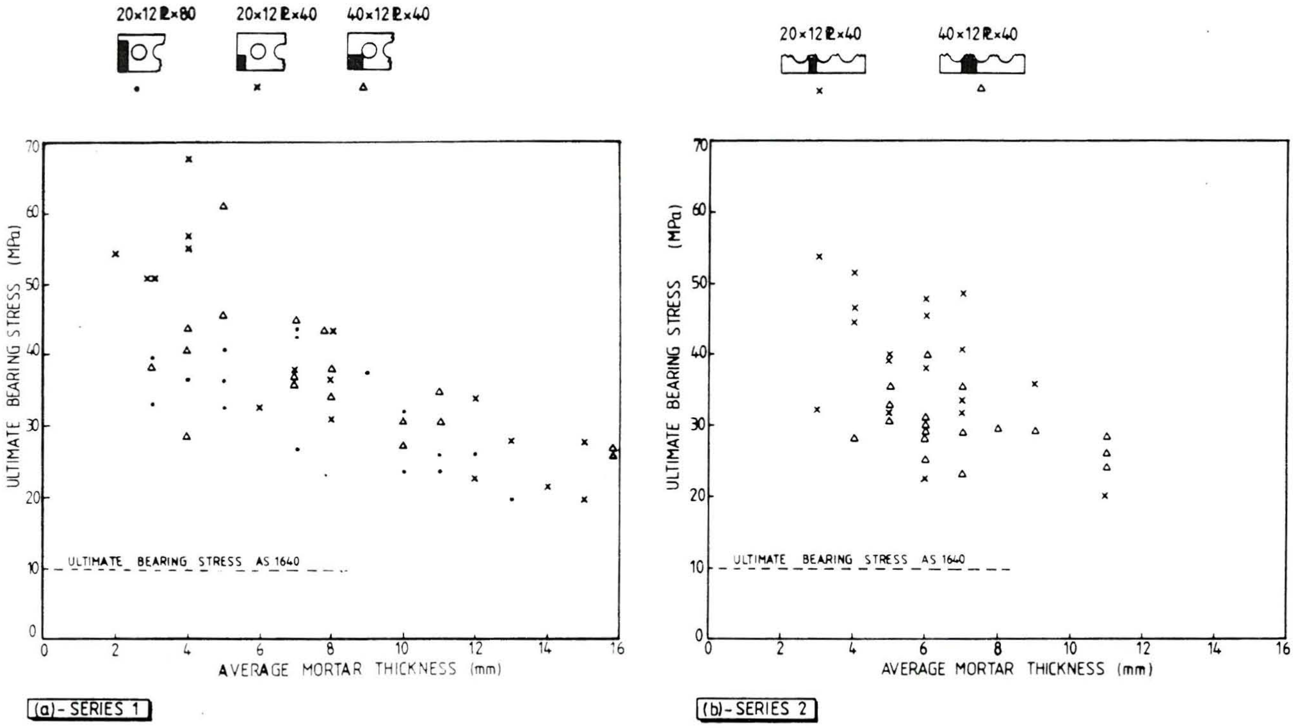


FIGURE 4 - SMALL AREA BEARING TESTS.

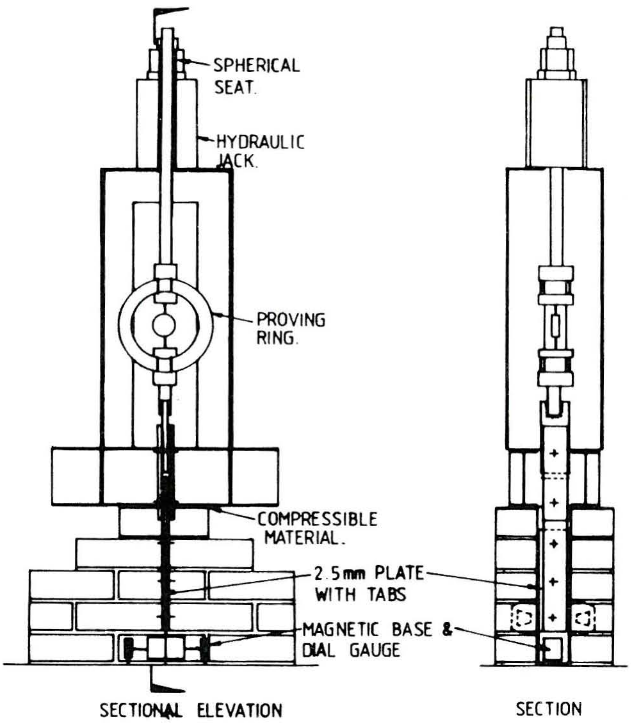
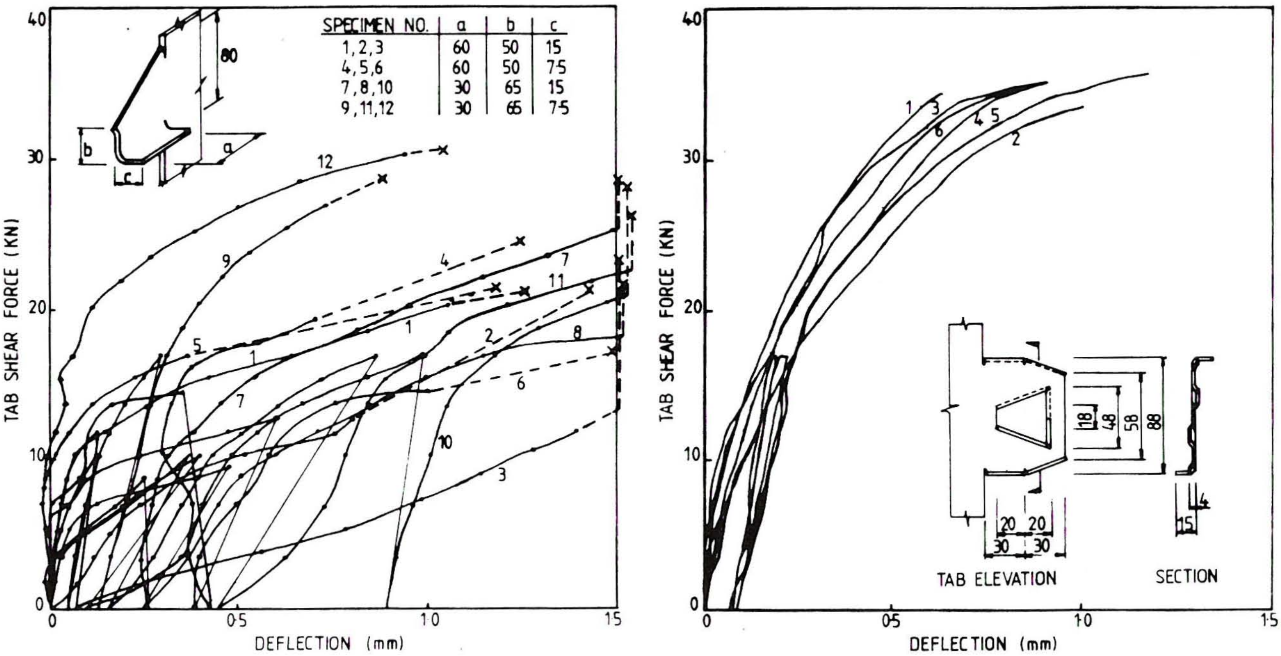


FIGURE 5 - TAB SHEAR TEST ARRANGEMENT

Two series of tests were performed. Series 1 consisted of 12 single bed joint tab specimens with varying perpend and bed joint projections. Tab configuration and test results are shown in Figure 6a. After consideration of the results a double bed joint tab was proposed and 6 specimens were constructed. Series 2 tab configuration and test results are shown in Figure 6b. All 18 specimens failed by splitting the five courses of a leaf vertically at the tab position.

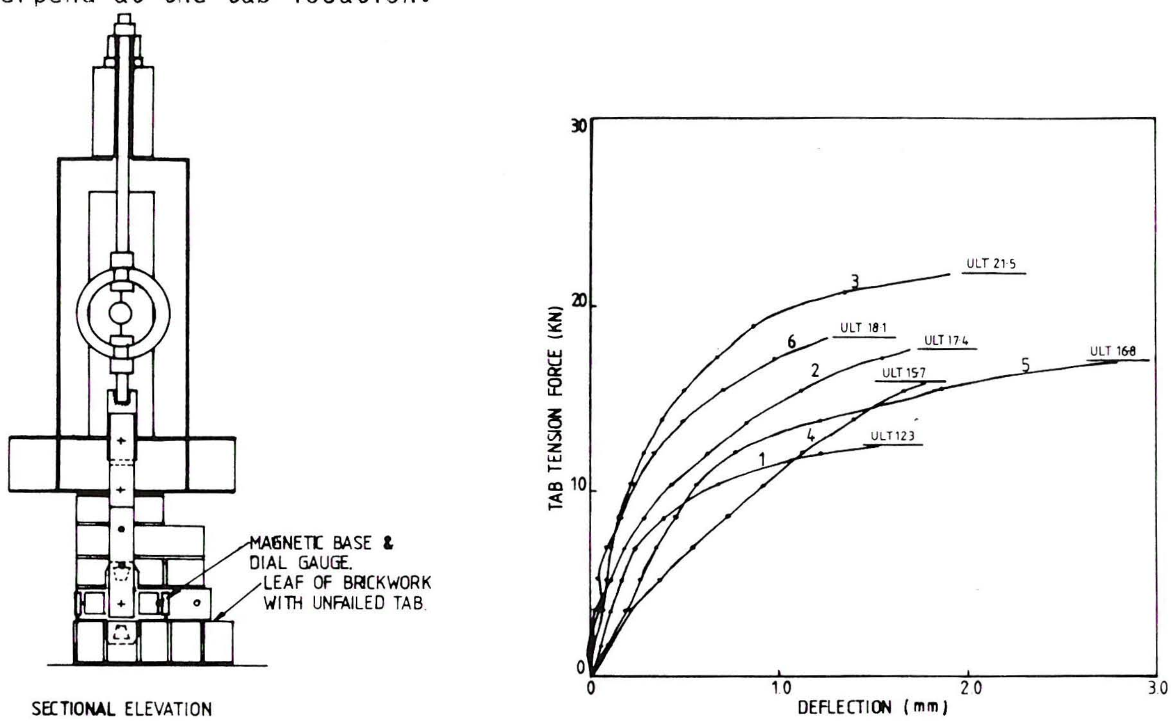


(a)-SERIES 1

(b)-SERIES 2

FIGURE 6 - TAB SHEAR TESTS.

2.2.3 Tab Tension Tests. The 6 unfailed tabs and attached brickwork leaves of the tab shear tests series 2 were tested in tension as shown in Figure 7a. Specimens 1 and 2 were supported on flexible material and tested at 126 days while specimens 3 to 6 were bedded on mortar and tested at 140 days. All specimens were air cured after 28 days. Test results are shown in Figure 7b. Predominantly failure included splitting of the brickwork through the plane of the perpend at the tab location.



(a)-TEST ARRANGMENT

(b)-TEST RESULTS

FIGURE 7 - TAB TENSION TESTS.

2.3 Full Scale Wall Tests

A series of 3 test panels, (a) standard cavity wall, (b) cavity wall with an unbonded intermediate steel mullion (R.H.S.), (c) cavity wall with vertical shear connector, have been constructed and tested.

2.3.1 Test Arrangement. All walls were constructed in a similar manner and consisted of a 50 mm cavity with each leaf 6 bricks long and 35 courses high. Joints were 'struck-off flush'. The walls were air cured in the laboratory and tested at 56 days. Each wall was subjected to a line loading consisting of a distributed out of plane load applied along a central vertical line. The load was applied by two Amsler jacks, mounted horizontally in parallel, to an articulated loading device which applied 8 equal point loads, distributed uniformly between horizontal restraints. The loading device was constructed to allow the vertical shortening necessary at large wall displacements. Figure 8 details the test arrangement and the base restraint. The top horizontal restraint for the standard cavity wall consisted of a 200 UB25 beam with a 20x20 mm continuous bead of non-shrink grout between the beam and the wall extending the length of the wall. For the intermediate steel mullion wall and the vertical shear connector wall the grout bead was replaced by 2 of 20x20x50 mm rubber blocks at 1400 mm centres and the mullion/connector was fixed with an M16 bolt to the reaction frame. A vertically slotted hole was provided in the joint to minimize vertical restraint. The vertical span between horizontal restraints was 2.9 m.

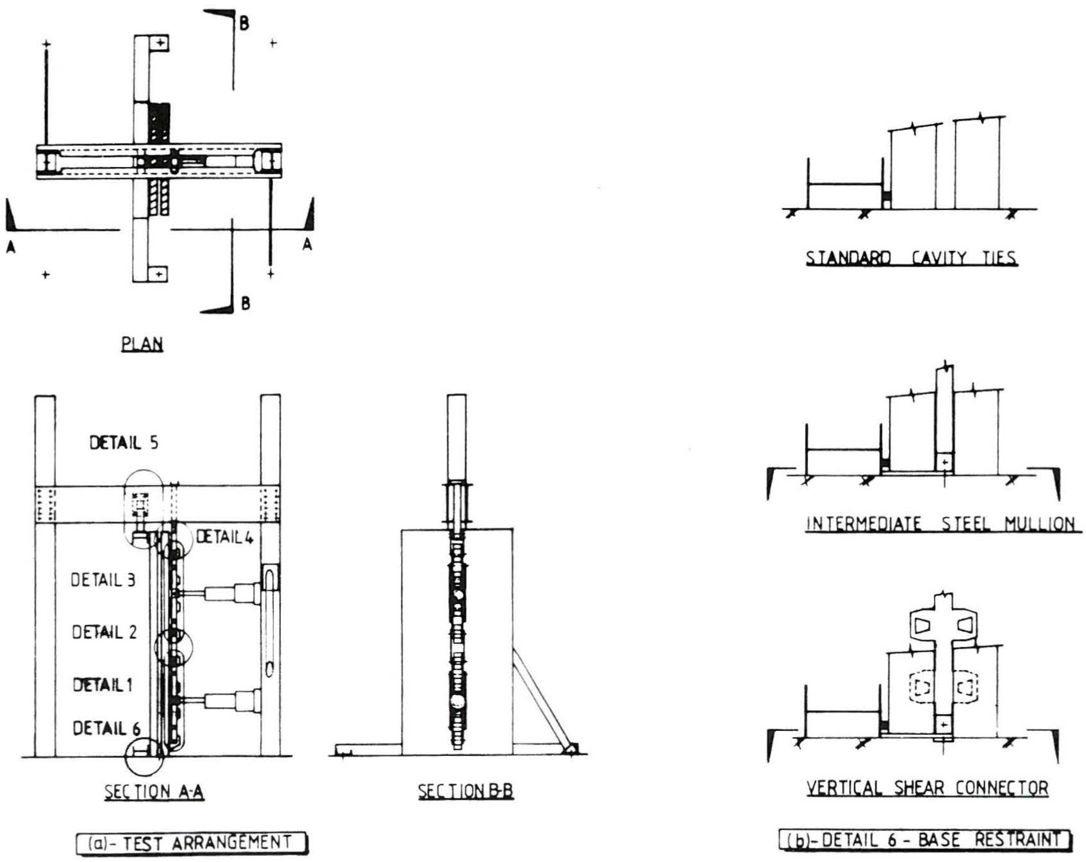


FIGURE 8- FULL SCALE WALL TESTS.

2.3.2 Standard Cavity Wall. Proprietary dual limb cavity ties manufactured from 3.15 mm diameter wire with a 4 mm central kink were installed in the bed joints above the 1st, 7th, 14th, 21st, 28th and 34th courses. Ties were located centrally and at 115 mm from both ends of the course. Five of the ties were strain gauged and calibrated to measure tie force. The ties were installed at locations as shown in Figure 9. Theoretical tie forces were calculated and are compared with measured tie forces in Figure 9.

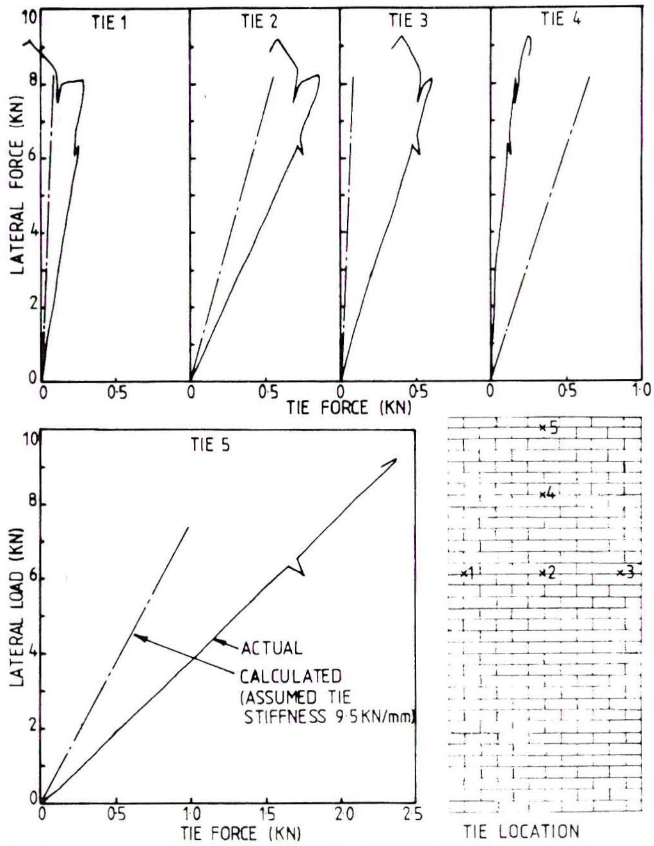


FIGURE 9–DUAL LIMB CAVITY TIE FORCES.

Total lateral load is plotted against central mid height deflection of the rear leaf in Figure 10a. An initial crack was detected in the front leaf at a load of 6 kN but did not propagate until the rear leaf commenced to crack at a load of 8.2 kN. This load corresponds to a bond stress of 0.46 MPa assuming simply supported conditions and the normal design assumption that the capacity is the sum of the capacities of the two leaves. This assumption leads to an upper bound of capacity corresponding to infinite tie stiffness. Cracks were observed in the front leaf in the bed joint above the 19th course and to a lesser degree above the 20th course. The rear leaf cracked in the joint above the 21st course.

2.3.3 Intermediate Steel Mullion Wall. A 76x51x3.2 R.H.S. with a measured yield stress of 430 MPa was installed centrally within the cavity. Each leaf of the wall was built tight against the R.H.S. A vertical damp-proof course was provided between the back leaf and the mullion. Standard cavity ties as described in the standard cavity wall tests were used with the exception that additional ties were provided on both sides of the mullion and at half the vertical spacing.

The wall was unloaded after the first and second cracks occurred in the wall at the joint above the 19th course of the rear and front leaves respectively. Loading then continued with extensive cracking until the central deflection reached an arbitrary 130 mm. The load deflection plot is presented in Figure 10b.

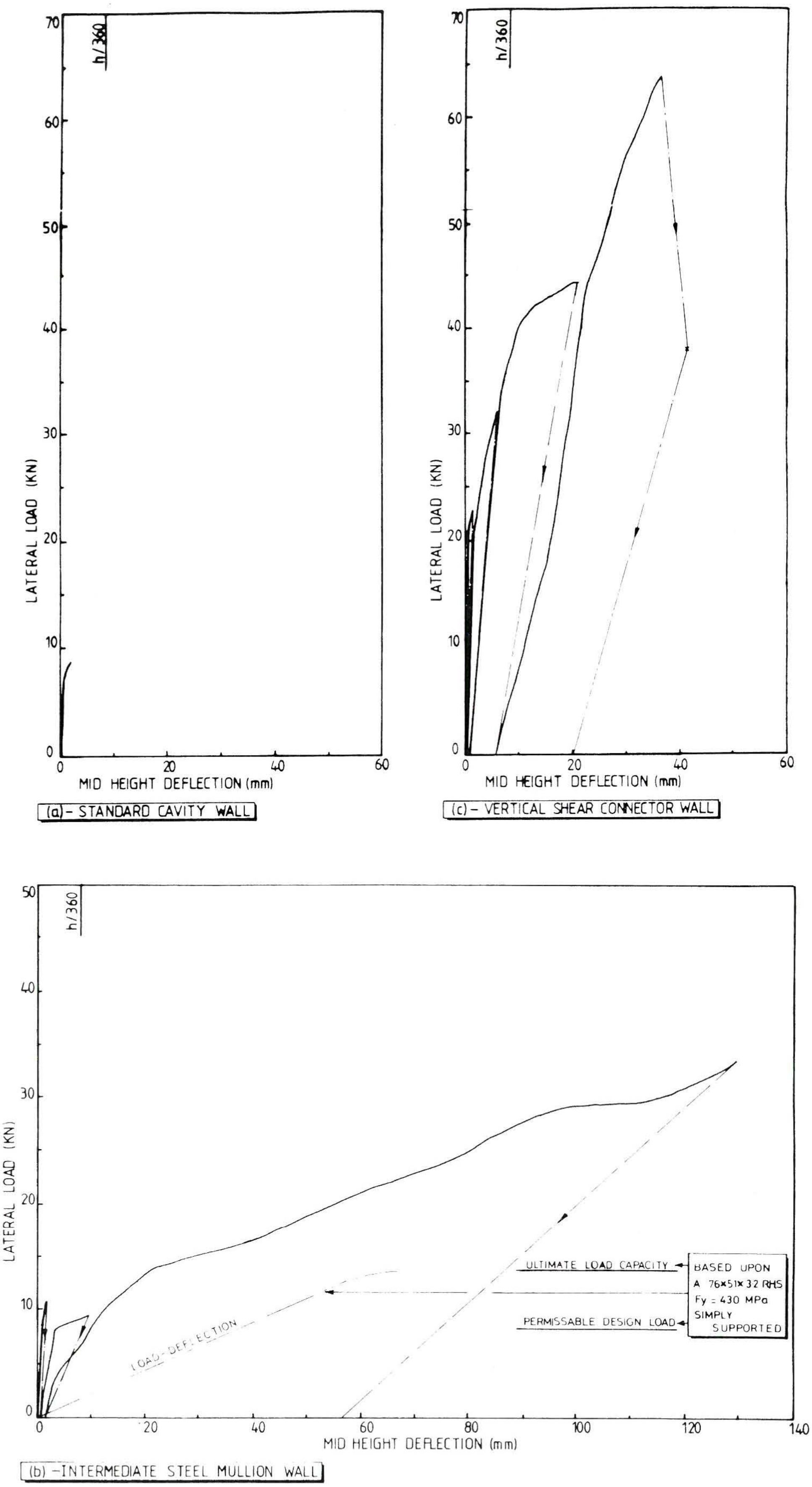
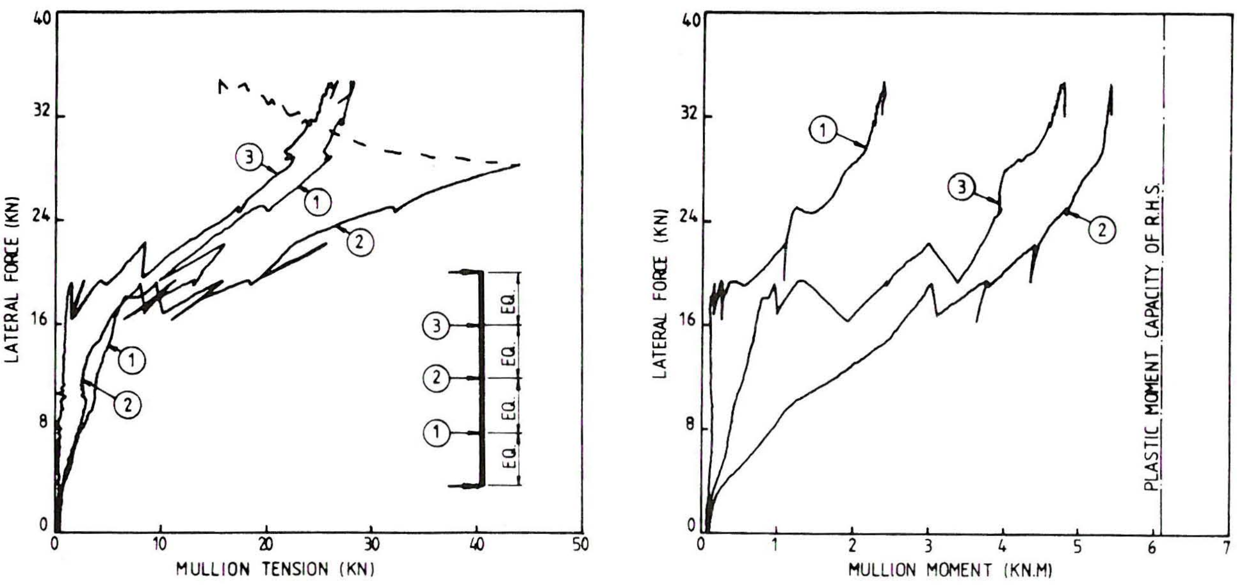


FIGURE 10 – FULL SCALE WALL TESTS.

The mullion was strain gauged at the mid span and quarter points. Mullion tension force and moment are plotted in Figure 11 for the loading after second crack.



(a)-MULLION TENSION

(b)- MULLION MOMENT

FIGURE 11 – MULLION TENSION & MOMENT.

2.3.4 Vertical Shear Connector Wall. A vertical shear connector fabricated from 2.5 mm thick Lysaght Zinc Hi-Ten with a measured yield stress of 480 MPa was installed vertically in the centre of the wall. The connector had a tab configuration as described in Figure 6b. Standard cavity ties were installed at 115 mm from both ends of the bed joints and spaced vertically as defined for the standard cavity wall test.

After the first crack occurred in the back leaf of the wall at the joint above and below the 20th course on the right and left of the connector respectively the wall was unloaded. The wall was also unloaded at an arbitrary load of 32 kN and then loaded until the connector yielded after which the wall was unloaded and then reloaded until the wall failed. At the ultimate load the front leaf of the wall split vertically the full height of the wall through the perpends in which the connector tabs were embedded. No additional horizontal cracks were observed. The load deflection plot is presented in figure 10c.

The connector was strain gauged and the vertical tension force in the connector at the 19th course is plotted against total applied load in Figure 12. The connector tension force is negligible before cracking, but after cracking increases with the applied load. The wall then acts as a composite (brick/steel) beam with tension in the vertical connector and balancing vertical compressive stresses in the front leaf of brickwork.

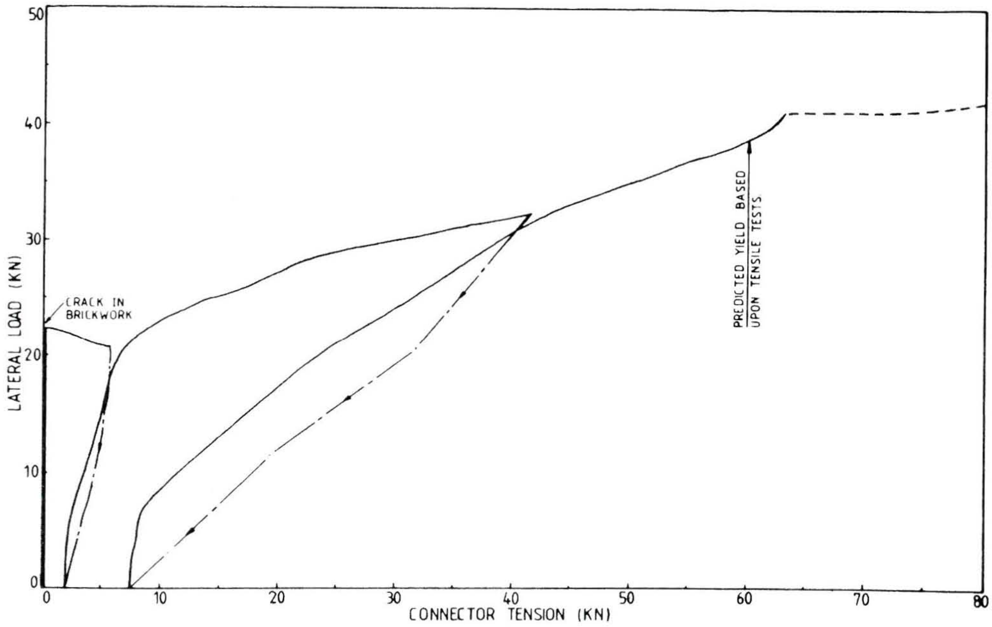


FIGURE 12 – CONNECTOR TENSION

3. REVIEW OF TEST RESULTS

3.1 Tab Development Tests

3.1.1 Small Area Bearing Tests. Examination of Figure 4 reveals a tendency of decreasing ultimate bearing stress with increasing mortar thickness. AS1640 conservatively defines the ultimate bearing capacity of brickwork under small area bearing plates for a mortar thickness of less than 12 mm.

3.1.2 Tab Shear Tests. The load deflection plots presented in Figure 6 for the proposed tab exhibit a high degree of uniformity in both stiffness and ultimate load capacity. The ultimate load capacity is clearly dependent upon the vertical splitting strength of the brickwork.

3.1.3 Tab Tension Tests. The test specimens had been loaded in shear to near failure prior to the tension tests hence the plots in Figure 7b represent a lower bound of tab ultimate tensile strength and stiffness.

3.2 Full Scale Wall Tests

3.2.1 Standard Cavity Wall. Figure 9 shows that the actual force carried by a dual limb cavity tie is highly variable.

3.2.2 Intermediate Steel Mullion Wall. The normal design office assumption that the post cracking strength and stiffness of the wall be based upon the properties of the R.H.S. mullion alone is shown in Figure 10b to be conservative as both the stiffness at permissible capacity and the ultimate capacity of the wall exceed twice that of the R.H.S. mullion. Figure 11a shows that the increases are due to longitudinal shear transfer between the R.H.S. and brickwork resulting in a tension force in the R.H.S. The effect of (a) cyclic loading, (b) a vertical damp-proof membrane and (c) end plates are being investigated.

3.2.3 Vertical Shear Connector Wall. The post cracking stiffness of the vertical shear connector wall is shown in Figure 10c to be of similar magnitude to the pre-cracking stiffness while still exhibiting ductility prior to failure.

3.2.4 Comparison of Wall Behaviour (Figure 10).
(a) The load to cause cracking of the vertical shear connector wall was 2.6 times the load to crack the standard cavity wall and 2.1 times the load to crack the intermediate steel mullion wall.

(b) The maximum load applied to the vertical shear connector wall was 7.4 times the maximum load of the standard cavity wall and 1.9 times the maximum load applied to the intermediate steel mullion wall.

(c) The central deflection of the vertical shear connector wall was within acceptable limits (h/360) up to a load of approximately 40 kN compared with a load of approximately 10 kN for the intermediate steel mullion wall.

(d) The vertical shear connector wall and the intermediate steel mullion wall would have recovered acceptably up to loads of approximately 37 kN and 20 kN respectively; that is, following unloading the walls would almost, if not completely, return to their original locations and cracks could close to within acceptable limits, with fewer cracks in the vertical shear connector wall.

Though the base fixing detail for the intermediate steel mullion wall and the vertical shear connector wall were similar (refer Figure 8b) it appears that the latter developed larger base moments.

The weights of steel required for the fabrication of the vertical shear connector and the intermediate steel mullion used in the tests were 3.3 kg/m and 5.3 kg/m respectively.

4. CONCLUSION

It has been shown that:-
(a) It is possible to provide for effective shear transfer between the leaves of cavity brick walls, and for effective post-cracking composite action (brick/steel) by means of a vertical shear connector of light gauge metal.

(b) This shear transfer and composite action significantly increase the strength and stiffness of cavity brick walls under out-of-plane loads, both before and after cracking.

(c) The estimate of the post-cracking strength and stiffness of a cavity wall reinforced with an intermediate steel mullion (R.H.S.) within the cavity is conservative when based upon the properties of the mullion alone.

5. ACKNOWLEDGEMENT

This research was carried out within the University of Queensland Department of Civil Engineering. The authors wish to thank P.G.H. Clay Bricks and Pavers for providing the 2,000 bricks used in the investigation.

6. REFERENCES

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