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THE BEHAVIOUR OF MASONRY IN HORIZONTAL FLEXURE

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ABSTRACT

The strength of masonry in out-of-plane horizontal flexure, with stresses parallel to the bed joints, is an important parameter in the design of walls for lateral loading. Stress distributions in this type of behaviour are complex and little understood, leading to simplified assumptions or empirical relationships in design codes. The paper summarises experimental and analytical work on horizontal flexure for clay brick masonry.

Experimental results for a large number of masonry beams are presented and empirical relationships defined. A detailed description of the load-deflection behaviour of a masonry beam and the development of cracks prior to failure is given. Simplified moment analysis is used to generate an approximate understanding of behaviour. A detailed analysis of curvatures in bricks and mortar joints is then used to estimate flexural stiffness which is compared with test results. The effect on flexural properties of unfilled perpend joints is also discussed. The results provide a better understanding of flexural behaviour in masonry and can be used to formulate more rational design rules.

INTRODUCTION

The properties most important to the behaviour of laterally loaded masonry walls are the flexural strength and stiffness in the two 'natural' directions of the brickwork. Because masonry contains continuous bed joints which constitute planes of weakness, its resistance is best expressed in terms of orthotropic flexural properties about axes parallel to and normal to these planes. Some evidence has been presented (Satti and Hendry, 1973 and Sahlin, 1971) that the flexural strength about an axis inclined to the bed joints can be predicted from the strengths about orthogonal axes aligned with the joints, and it has been suggested (Baker, 1982) that an elliptical interaction diagram may be appropriate. Flexural behaviour of masonry about an axis parallel to the bed joints

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(called 'vertical bending') is simple to understand and to model but behaviour about the axis at right angles, or normal to the bed joints (called horizontal bending) is relatively complex and difficult to model. A simple empirical relationship between horizontal and vertical bending has been demonstrated (Lawrence, 1975) and analysis was attempted (Baker, 1979) but in the absence of a full understanding of horizontal flexural behaviour and the magnitude of the torsional moments a complete analysis was not possible.

This paper considers the behaviour of masonry in horizontal flexure, both experimentally and theoretically, and examines how that behaviour may be expressed in terms of the brick properties and the basic bond strength of the masonry. By this approach the need to test masonry beams or wallettes in flexure across the perpends is obviated and the design of laterally loaded walls requires fewer parameters.

There are no universally accepted methods for testing the flexural properties of masonry. The methods stipulated in the British masonry code (British Standards Institution, 1992) are considered to be unsuitable for two reasons. Firstly, the vertical orientation of the specimens produces a superimposed compressive stress due to self-weight, which differs from bed joint to bed joint, making the result different from the true flexural strength. Secondly, also because of the vertical orientation, there is the possibility of friction at the base of the horizontal bending specimen, which could affect the failure load and even the pattern of cracking. It is desirable for the measure of flexural properties in each of the two orthogonal directions to be free of the effects of self-weight and independent of the method of test. Self-weight effects can then be included as a separate factor in the design of laterally loaded walls.

In conjunction with an extensive program of testing laterally loaded walls a large number of beam specimens has been tested in both vertical and horizontal bending. These results provide a valuable body of data for the study of behaviour in horizontal bending and particularly how this behaviour relates to other properties. The tests used a total of seven separate batches of the same clay brick laid with 1:1:6 mortar (cement:lime:sand by volume) and have been fully reported (Lawrence, 1983). The total number of specimens tested is 311 vertical beams and 310 horizontal beams, plus some additional specimens to investigate the effects of unfilled perpend joints.

TEST SPECIMENS AND PROCEDURES

For flexure across the perpend joints a section of stretcher-bonded brickwork, supported as a simple beam and loaded with two line loads near the third points, is an obvious configuration. An earlier investigation (Lawrence & Morgan, 1975) examined various possible configurations for the flexural test across perpends. Conclusions were drawn on the basis of span-to-depth ratios, the positioning of loads relative to perpend joints, and the observed performance of each specimen. The recommendation from those tests, to use a specimen four bricks in length and four courses high with two line loads, was followed for the investigations reported in this paper. This specimen is referred to throughout this paper as a horizontal beam because it represents a horizontal strip of masonry in a wall.

The specimens for flexure across the perpend joints were tested using a support span of 900 mm and a span between load points of 400 mm. For companion tests across the bed joints the specimens were nine-high stack-bonded beams tested on a span of 690 mm with a load bar spacing of 320 mm. Deflections were measured at the centre of the span for both types of specimen. Compressible fibre-board strips or water-filled hoses were used to even out irregularities under the load and support bars. Load and central deflection were continuously traced on an X-Y recorder, producing a load-deflection plot for each specimen. The ultimate load, elastic modulus, and other parameters of interest were subsequently determined from these plots.

Materials of a brittle nature may perform differently when subjected to different rates of loading. While no attempt was made to measure the effect of loading rate on the measured beam properties, it was considered desirable that a steady uniform rate of load increase should be used. A servo-controlled loading system was designed and built for the beam testing apparatus, using the signal from the force transducer in a feedback loop to give a controlled rate of loading.

LOAD-DEFLECTION BEHAVIOUR IN HORIZONTAL FLEXURE

The load-deflection behaviour of horizontal beams almost always follows a pattern characterised by two straight line segments (see Fig. 1). The initial portion of this plot is linear, indicating elastic behaviour. After the sudden change of slope, if the load is removed the unloading characteristic is a straight line which returns approximately to the origin (see Fig. 1). On subsequent reloading the plot retraces this same straight line until the point at which unloading began, and then extends the second line segment at its lower slope. The most likely explanation for this behaviour is that progressive cracking is occurring along the second straight segment of the load-deflection graph, gradually reducing the flexural stiffness of the beam. Failure occurs at the point when the flexural resistance is insufficient to resist the applied load, and further redistribution of moments is not possible.

Four parameters determined from the load-deflection plot have been used to characterise the flexural behaviour of horizontal beams. These parameters are the slope of the initial straight segment, the load at change of slope (cracking load), the ultimate load, and the ultimate deflection. Figure 1 indicates the method used for determination of these parameters. An equivalent ultimate flexural tensile stress was calculated from the ultimate load, taking account of the self-weight of the beam. This stress was calculated at the centre of the beam span using the gross cross-section. Similarly, a flexural tensile stress was calculated to correspond to the change of slope. Each of the load-deflection slopes was used to calculate an equivalent flexural Young's Modulus for an idealised homogeneous beam with the same gross cross-section as the test beam. The ultimate deflection was used in conjunction with the ultimate load to calculate a secant Young's Modulus for the idealised homogeneous beam.

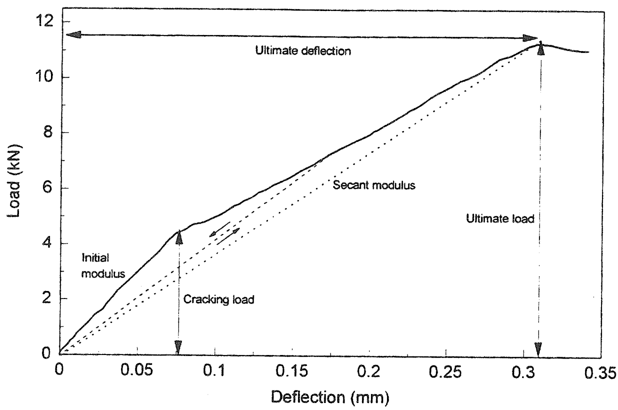

Fig. 1. Load-Deflection Plot for a Horizontal Beam

RESULTS

Table 1 summarises the values for Young's Modulus and stress obtained from the tests on horizontal and vertical beams. Each brick batch is shown separately, with the number of specimens tested and the mean and coefficient of variation of each property. Failures of the horizontal beams were almost invariably along straight lines through the perpend joints and bricks.

The stress at change of slope in horizontal beams is plotted against ultimate strength in Fig. 2 for individual specimens. There is no correlation, indicating that these two properties depend on different factors.

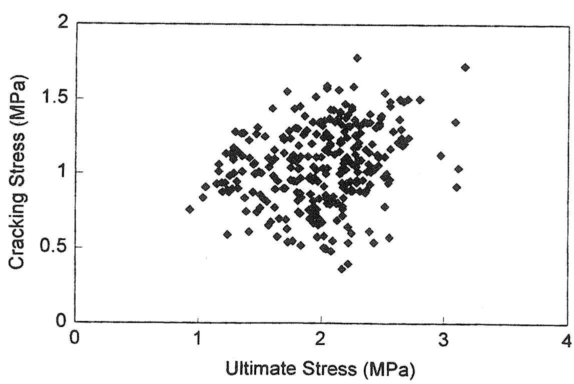

Fig. 2 Cracking vs Ultimate Stress for Horizontal Beams

In Fig. 3 are shown the means of test groups for stress at change of slope in horizontal beams plotted against the strength of vertical beams. Each point represents the average of approximately 9 individual specimen results for each property. The relationship indicates a strong dependence on common factors and supports the view that the change of slope in horizontal beams is due to cracking in the perpends and is therefore related to bond strength.

Table 1. Strength and Elastic Modulus Results From Beam Tests

Property		Batch 1	Batch 2	Batch 3	Batch 4	Batch 5	Batch 6	Batch 7
E_{hl} (GPa)	No.	50	54	53	44	44	36	27
	Mean	18.5	15.1	18.8	18.5	18.0	19.6	19.3
	C.V.	0.12	0.10	0.12	0.14	0.08	0.08	0.08
E_{hs} (GPa)	No.	49	54	54	44	44	35	27
	Mean	15.1	9.7	10.2	11.4	12.3	12.6	11.5
	C.V.	0.17	0.12	0.15	0.14	0.12	0.19	0.28
F_{hc} (MPa)	No.	48	54	52	43	44	36	21
	Mean	1.11	0.88	0.84	1.09	1.28	1.22	1.04
	C.V.	0.27	0.29	0.18	0.17	0.13	0.17	0.13
F_{hu} (MPa)	No.	51	54	54	44	44	36	27
	Mean	2.01	2.10	1.95	2.17	2.13	1.94	1.31
	C.V.	0.11	0.10	0.26	0.18	0.19	0.23	0.20
E_v (GPa)	No.	50	54	54	45	45	36	27
	Mean	20.9	19.8	24.5	23.8	22.8	27.3	27.0
	C.V.	0.14	0.08	0.10	0.05	0.06	0.09	0.05
F_v (MPa)	No.	50	54	54	45	45	36	27
	Mean	1.42	0.95	0.80	1.13	1.60	1.67	1.21
	C.V.	0.12	0.24	0.14	0.22	0.13	0.18	0.15

E_{hl} = Initial tangent modulus (horizontal beams)
E_{hs} = Secant modulus (horizontal beams)
F_{hc} = Cracking stress (horizontal beams)
F_{hu} = Ultimate stress (horizontal beams)
E_v = Elastic modulus (vertical beams)
F_v = Ultimate stress (vertical beams)

SIMPLIFIED MOMENT ANALYSIS

When a section of stretcher bonded brickwork is bent horizontally the bending stresses are not uniform but vary due to the different relative stiffness of bricks and perpend joints. This is illustrated in Fig. 4 where the curved lines on section AA represent the unknown distribution of flexural (tensile or compressive) stresses acting normal to the section. For the purpose of this discussion the segment of brickwork is considered to extend infinitely in two dimensions and is subjected to uniform bending moments about axes normal to the bed joints. Each section through the perpend joints and parallel to

AA would have a similar distribution of stresses acting upon it, but for adjacent sections there is a `phase difference' equal to the height of one course. The transfer of stress between two adjacent sections must produce torsion on the connecting portions of bed joint, allowing the higher stresses in a particular brick to `flow' to the bricks in adjacent courses. This torsional action has been reported previously (Baker, 1979). This paper presents a simplified analysis for determining the magnitude of the torsional moments.

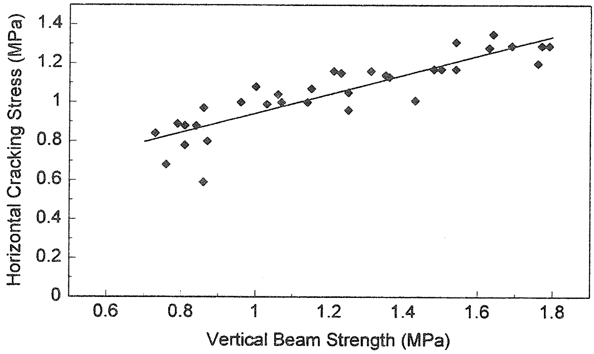

Fig. 3 Cracking Stress for Horizontal Beams vs Failure Stress for Vertical Beams

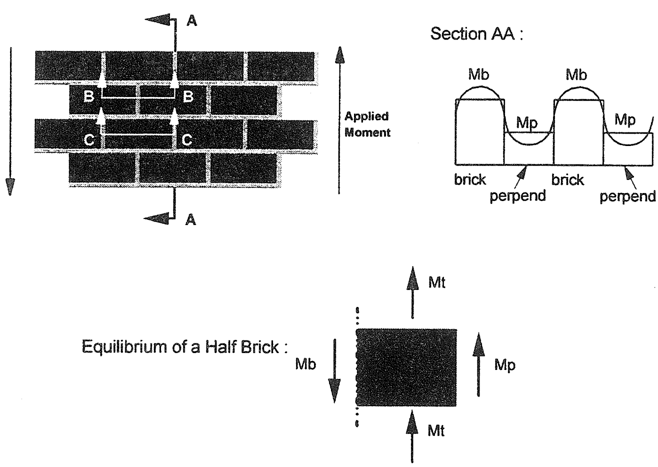

Fig. 4 Moments Acting Within a Section of Masonry Under Flexure

For simplicity, this analysis will deal with the average moment across any perpend, M_p and the average across any brick, M_b, as shown on section AA of Fig. 4. Test results show that bricks are stiffer than mortar joints and so M_b is shown greater in magnitude than M_p. Clearly, the average moment per course is given by:

$$M = 0.5\,(M_p + M_b) \qquad [1]$$

Consider a half brick as shown in Fig. 4. Because M_b is greater than M_p the torsional moments M_t will act in the direction shown, and the expression of equilibrium is:

$$M_b = M_p + 2\,M_t \qquad [2]$$

By considering the relative rotations on sections BB and CC in Fig. 4 it is possible to derive a third equation involving M_b, M_p and M_t which, in conjunction with Equations 1 and 2, allows the relative magnitudes of the moments to be calculated. A simplified analysis is presented and is shown to give reasonable predictions of the distribution of moments between bricks and perpends.

The dimensions of the model are defined as t = thickness of a mortar joint; b = length of a brick; h = course height of a brick. The moment-curvature relationships are defined as:

For bricks: $M_b = K_b\phi_b$ [3]

For perpends: $M_p = K_p\phi_p$ [4]

For torsional action on a half-brick bed joint: $M_t = K_t\phi_t$ [5]

Let: M = applied moment per course and define:

$$\alpha_b = \frac{K_b}{K_p} \qquad [6]$$

$$\alpha_t = \frac{K_t}{K_p} \qquad [7]$$

Figure 5 shows the assumed linear variation of curvature in the model. The brick curvature at the interface with the mortar is determined from the requirement to match brick and mortar moments at this point. Hence:

$$K_b\phi'_b = K_p\phi_p \qquad [8]$$

$$\text{Therefore } \phi'_b = \frac{\phi_p}{\alpha_b} \qquad [9]$$

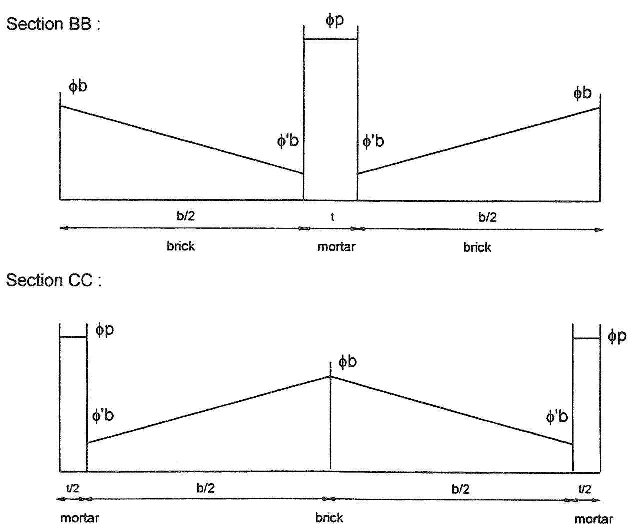

Fig. 5 Variation of Curvature Assumed for Brick and Mortar Segments

The rotation of any element of section BB or section CC, relative to the left-hand end of the section, can be calculated and the relative rotation between these sections, divided by the distance between them (h + t) gives the twist which is resisted by torsion on the bed joints. Hence it can be shown that:

$$\phi_t = \frac{1}{6C}\left[\phi_b\left(C_1 + 2 - 8\frac{t}{b}\right) - C_1\phi'_b + \phi_p\right] \qquad [10]$$

Where: $C = \frac{2(h+t)(b+t)}{tb}$ [11]

and: $C_1 = \left(\frac{t}{b}\right)^2 + 3 - \frac{b}{t}$ [12]

(Note that the first term in C_1 will usually be negligible.)

Replacing ϕ_b from Equation 3, ϕ_p from Equation 4, ϕ'_b from Equation 9 and substituting in Equation 5 gives an expression for M_t which can be solved with Equations 1 and 2 to give:

$$M_t = \frac{2b - 8t + 6b\alpha_b}{6bC\frac{\alpha_b}{\alpha_t} - 2b(C_1+1) + 8t + 6b\alpha_b} \qquad [13]$$

$$M_b = M + M_t \qquad [14]$$

$$M_p = M - M_t \qquad [15]$$

This result gives a means of estimating the moments in the bricks and perpends and the torsional moments on the bed joints, given the geometry of the beam, the moment-curvature relationships for bricks and perpends, and the torque-twist relationship for the bed joints.

For the brickwork beam tests the average brick dimensions over all seven batches are t=0.010 m, b=0.230 m, h=0.078 m. The average gross-section elastic modulus for the bricks in this mode of bending is 20 GPa (Lawrence, 1983). Using this value to calculate the stiffness of the bricks gives $K_b = EI = 1.73 \times 10^5$ Nm/rad/m.

The modulus of elasticity for the mortar was 13.7 GPa (Lawrence, 1983). Using this value to calculate the stiffness of the perpend joints gives $K_p = EI = 1.19 \times 10^5$ Nm/rad/m.

Measurements of the torque/rotation behaviour of half-brick torsion specimens have been reported (Lawrence, 1983). The overall average stiffness for the specimens tested is 3.94×10^6 Nm/rad. Rotation was measured in these tests over an average length of 0.088 m so the average torque/twist ratio is $K_t = 3.47 \times 10^5$ Nm/rad/m.

Therefore $\alpha_b = 1.45$ and $\alpha_t = 2.92$. This leads to $M_t = 0.102$ M, $M_b = 1.102$ M and $M_p = 0.898$ M.

The ratio of brick moment to perpend moment is therefore $M_b/M_p = 1.23$.

Experimental Verification

Tests were carried out to measure the distribution of moments in a horizontal beam for comparison with this analysis. The tests also provided the opportunity to investigate whether the occurrence of cracks in perpend joints could be detected. Nine strain gauges were attached to one brick of a test specimen which was then subjected to various cycles of load, extending beyond the point where the slope of the load-deflection plot changed. The gauges were electrical foil rosettes of 5 mm gauge length and were aligned with the axis of the brick, measuring flexural strains in the direction of the span and at ninety degrees to this direction. The brick surface was carefully filled with epoxy and ground smooth before the gauges were attached. The positions were chosen to give a measure of relative bending moments at the centre of the brick and in the perpend joints adjacent to the ends of the brick. Three gauges were placed across each section, to measure the distribution of strain through the thickness of the brick and to allow for expected random variations from point to point. The following loading regime was applied to the specimen:

- Loading to 7.3 kN with the gauges on the compression face, followed by unloading.
- Loading to 6.7 kN with the gauges on the tension face, followed by unloading.
- Reloading to 6.7 kN with the gauges on the tension face, followed by unloading.
- Loading to failure at 8.1 kN with the gauges on the compression face.

To draw a comparison between strains at the centre and those at the ends, the three measurements across the brick were averaged at each section. For the first load case, with the gauges on the compression face, the ratio of centre to end strain was nearly constant. In the second case, where the gauges were on the tension face, the ratio was similarly constant up to a load of approximately 5 kN, at which stage the ratio increased sharply. This increase is interpreted as due to the initiation of cracks in the perpends at the ends of the instrumented brick, relieving stresses at those points. On unloading and reloading (the third case) the ratio remained higher than the initial level, indicating that a permanent change had occurred in the specimen. In the fourth loading case, when the specimen was inverted again to place the gauges on the compression face, the ratio was lowered to approximately the original level. The cracks were then in compression and were therefore not affecting the bending stiffness. As the load increased in this case, the pattern of a sudden increase in the strain ratio was not repeated and failure occurred suddenly at a load of 8.1 kN. It is believed that when cracks were initiated on the face opposite to existing cracks formed in the second load case, the perpends lost all moment capacity and failure occurred immediately.

Examination of the relationships between centre and end strains for the first and second loading cases, in the linear portions before cracking, shows an average ratio of centre to end strain of 1.33. This compares quite well with the ratio of brick to perpend moments of 1.23 calculated from the stress analysis, and confirms that the analysis, although simplified by considering average strains across brick and mortar, and by assuming linear variations of curvature, gives a useful estimate of the distribution of moments within the beam.

FLEXURAL STIFFNESS

The analysis presented in the last section predicts a distribution of bending moments in the horizontal beam from a knowledge of the relative stiffnesses of the component elements. It is possible to use these stiffnesses, and the distribution of moments, to calculate the overall flexural stiffness of the beam. The method of Virtual Work has been used (Lawrence, 1983).

The result of this analysis, using the average properties for all seven batches, is a beam deflection of 0.0170 mm/kN, representing an equivalent gross-section elastic modulus for the beam of 17.6 GPa. This compares well with the overall average initial tangent elastic modulus measured for the 310 horizontal beams of 18.1 GPa (E_{hl} in Table 1). This agreement provides further verification of the simplified moment analysis and the linear curvature model and shows that it is possible to calculate the bending stiffness of masonry from the properties of its constituent bricks and mortar.

UNFILLED PERPEND JOINTS

By using the method of analysis presented in the last section it is simple to analyse a horizontal beam with no mortar in the perpend joints. This is useful because it is often said of laboratory and theoretical investigations of brickwork that they bear little resemblance to the conditions on a building site, where workmanship plays such an important role in determining the quality of the material. Furthermore, it is said that one of the principal discrepancies between laboratory and site brickwork is that the perpend joints on site are often not properly filled with mortar. The major effect of this departure from good practice will be seen in the behaviour of the brickwork in horizontal flexure. Hence an analysis of a horizontal beam with unfilled perpend joints will provide some insight into the magnitude of these effects.

The analysis of a beam without perpends is simpler than the analysis of a complete beam. For this case the physical model of the beam is the same, but the distribution of moments is such that $M_t = M$, $M_b = 2\,M$, and $M_p = 0$. Using the same brick and torsion stiffnesses as before, the calculated beam deflection is 0.0265 mm/kN, equivalent to a modulus of elasticity of 11.3 GPa.

It has been shown above that the perpend joints in horizontal beams tend to crack before the ultimate condition is reached. On the assumption that all perpends have lost their effectiveness prior to failure we can compare the predicted stiffness for beams with unfilled perpends to the test results for ultimate secant modulus on the normal beams. The average ultimate secant elastic modulus for all batches of test beams was 11.8 GPa (E_{hs} in Table 1) compared to the value of 11.3 GPa derived above. The ratio of calculated stiffness of beams without perpends to that of horizontal beams with full joints is 0.64, compared with the ratio of ultimate secant modulus to initial tangent modulus for the test beams of 0.65.

A series of special tests was conducted to provide further verification of this analysis of beams with unfilled perpend joints. These tests used the same type of specimen and loading arrangement as the normal horizontal beam tests, the only difference being that small sheets of polystyrene foam 10 mm thick were substituted for mortar in the perpend joints during construction. These foam pieces were pushed out prior to testing, leaving a gap in place of the mortar joint. An equal number of normal horizontal beams was constructed and tested at the same time as the special beams to give a direct comparison, without the confusing effects of workmanship and materials. The load-deflection behaviour of these specimens differs from that of the normal horizontal beams in that the well-defined change of slope is absent. Some specimens were subjected to six cycles of load before failure and their response confirmed that they did not undergo the progressive loss of stiffness that was observed in the normal horizontal beams. This is further evidence that the change of slope in horizontal beams is due to the occurrence of cracks in the perpend joints. Results of the tests on beams with unfilled perpend joints, and the corresponding normal beams, are summarised in Table 2. Although there is no reduction in strength due to unfilled perpends for brick number 2, bricks 1 and 3 show reductions of 19% and 40% respectively.

Table 2. Properties of Horizontal Beams With Unfilled Perpend Joints

		Unfilled Perpends		Normal			
		Eh1 (GPa)	Fhu (MPa)	Eh1 (GPa)	Ehs (GPa)	Fhc (MPa)	Fhu (MPa)
Brick 1	No.	9	9	9	9	9	9
	Mean	8.61	1.50	16.0	12.4	1.32	1.85
	C.V.	0.06	0.12	0.04	0.06	0.10	0.07
Brick 2	No.	9	9	9	9	9	9
	Mean	9.14	1.83	17.4	9.88	0.86	1.81
	C.V.	0.08	0.17	0.05	0.21	0.30	0.34
Brick 3	No.	8	8	9	8	7	9
	Mean	10.00	0.99	17.5	13.0	1.14	1.64
	C.V.	0.10	0.39	0.09	0.12	0.06	0.13

The average ratio of elastic modulus for the beams without perpends to that for the normal beams is 0.54, compared with 0.64 from the analysis. This result is reasonable, considering the degree of variability in test results and the fact that, for beams with unfilled perpends, the torsional stiffness of the bed joints plays a significant role in determining beam stiffness, and direct measurements were not made of this torsional stiffness for the special tests. The analysis could be expected to give even closer estimates if better data on the torsional stiffness were available. The average ratio of tangent elastic modulus for the beams without perpends to ultimate secant modulus for the normal beams is 0.80, which indicates that not all of the perpends in the normal beams were cracked at failure, and that the cracked perpends were still contributing some stiffness to the beam.

CONCLUSION

Results of tests on approximately 700 beam specimens have been presented and a detailed discussion of the load-deflection behaviour of masonry in horizontal flexure has been given. A simplified analysis of horizontal flexure has been used to estimate the distribution of moments between bricks, perpends and torsion on the bed joints. This analysis has been used to estimate bending stiffness for brickwork in horizontal flexure and the results agree well with tests. It has been demonstrated that perpend joints develop cracks prior to failure in horizontal bending and that beams with unfilled perpends behave in a manner similar to normal horizontal beams approaching the ultimate condition, that is with their perpends cracked.

The work presented in the paper provides a means of estimating the orthogonal strength ratio for masonry without expensive and cumbersome tests on beams or wallettes and highlights the importance of further studying the torsional behaviour of bed joints.

REFERENCES

Baker, L.R. (1979) A Failure Criterion for Brickwork in Bi-Axial Bending, Proc. of the Fifth International Brick Masonry Conference, Washington DC, pp. 71-78.

Baker, L.R. (1982) A Principal Stress Failure Criterion for Brickwork in Biaxial Bending, Proc. of the Sixth International Brick Masonry Conference, Rome, pp.121-130.

British Standards Institution (1992) Code of Practice for Use of Masonry. Part 1 - Structural Use of Unreinforced Masonry, BS5628:Part 1:1992, London.

Lawrence, S.J. (1975) Flexural Strength of Brickwork Normal to and Parallel to the Bed Joints, Journal of the Australian Ceramic Society, V2, N1, pp.5-6.

Lawrence, S.J. (1983) Behaviour of Brick Masonry Walls Under Lateral Loading, Thesis (PhD), University of New South Wales, Sydney, Australia.

Lawrence, S.J. & Morgan, J.W. (1975) Strength and Modulus of Elasticity of Brickwork in Lateral Bending, Proc. of the British Ceramic Society, N24, pp.79-90.

Sahlin (1971) Structural Masonry, Prentice-Hall, New Jersey.

Satti, K.M.H. & Hendry, A.W. (1973) The Modulus of Rupture of Brickwork, Proc. of the Third International Masonry Conference, Essen, pp.155-160.