Composite action between brick panel walls and their supporting beams

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Mr V. A. Morgan, Consultant, Haymills Contractors Ltd

I read the Paper with interest; it is a valuable contribution to the literature on panel loading of brickwork.

48. It is evident from the tests that for H/L of 0·6 and over, full arching action takes place with maximum compression in the brickwork at the ends of the beam, with the beam acting as a tie to the arch. Also A/L is near the limiting value of 0·55 for H/L equal to unity with very small bending moment action on the beam.

49. The centre of pressure appears thus to be a parabola of central height 0·55, springing from near the ends of the beam. The tests appear to show that the strength of the corner brickwork controls the failure load and this would account for the high failure load value of 94·5 tons of test (10). The corner brickwork in this case was hard brick in cement, but the major portion of the brickwork was fletton brickwork in 1:1:6 mortar similar to that used in the other tests.

50. It is to be noted that for low values, such as $H/L = 0{\cdot}3$, beam action takes place. This probably occurs because the increased thrust due to the flatness of the arch causes the beam to fail in tension, at the same time overcoming shear and frictional resistance of the mortar. This would appear to have caused the diagonal tension cracks; beam action then increases the tensile force in the already cracked beam.

51. The value of $H/L = 0{\cdot}6$ recommended by Wood appears to be justified by the no-beam action and is a valuable guide to the designer. It would appear that if return ends such as $18 \times 4\frac{1}{2}$ in. brickwork were bonded to the fletton brick panels, these would considerably increase their load-bearing capacity. This could of course be verified by tests on panels with return ends. Where panels are built between columns capable of resisting the thrust or in continuous panels with columns between, high loading of individual panels appears possible. Even in these cases, however, the crushing of the corner brickwork appears to be the critical factor. In this latter case only vertical loading will occur.

Dr M. Amaratunga, Kingston College of Technology

The behaviour of reinforced concrete beams carrying brickwork has been the subject of some investigations carried out at Kingston College of Technology.[5] The particular aspect of the problem which has been under investigation is that of a precast beam during the process of bricklaying.

53. Several years ago, I designed some lintels based on the procedure given by Wood. Although these were meant to be propped during bricklaying, the props were inadvertently omitted, but no undesirable effects were visible. On another occasion, a reinforced concrete window frame, with a clear opening of 3 ft 3 in., was constructed to investigate the possibility of using such frames in low-cost housing projects in tropical countries. All the members of the frame were 2 in. thick and 9 in. wide, the top member being designed to serve as a lintel as well. No detailed tests were performed, but the deflexions caused by loading the frame with unbonded courses of bricks to a height of over 5 ft were not sufficient to affect the operation of the windows.

Paper published: *Proc. Instn civ. Engrs*, 1969, **43** (June) 175–194; Corrigendum, (Aug.) 657–663.

54. If precast reinforced or prestressed units such as lintels and door or window frames are to be used, their behaviour under all possible conditions of loading needs to be studied. The preliminary work carried out at Kingston College has been to determine primarily the deflexion characteristics of simply supported beams. The dimensions and reinforcement details of three beams are given in Table 13. All the beams had a width of 9 in. Equal areas of steel were provided at the top and bottom of the beams with a cover of $\frac{1}{2}$ in. The cube strength of the concrete was 4700 lbf/sq. in. and the modulus of elasticity 4×10^6 lbf/sq. in. Unbonded walls 9 in. thick were built on these beams. In some cases where the wall tended to become so high as be unstable before causing any appreciable damage to the beam, loads were applied through the walls to cause beam failure.

55. The range of beams tested so far is inadequate for definite conclusions to be drawn. The central deflexions were quite small and, in some cases, the load/deflexion curves tended to take the shape shown in Fig. 11. This seems to indicate that the friction between the bricks permits a certain amount of composite action to take place. Failure was characterized by cracks extending throughout the depth of the beams.

56. In the tests described by the Author, the stress and deflexion readings during bricklaying would have been very small, but I would like to know whether these observations are available, if the beams were unpropped.

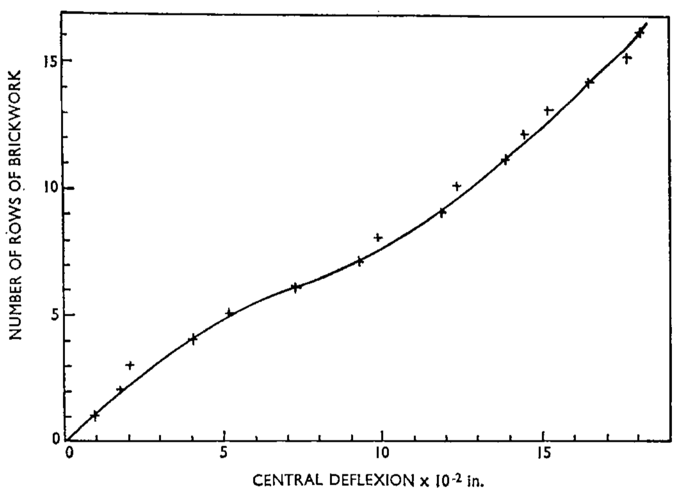

Fig. 11. Load/deflexion curve for 6 ft span lintel

Table 13. Principal details of some beams tested

Span *ft*	Beam depth *in.*	Reinforcement	Brick load *tonf*	Central deflexion *in.*
2·75	2	4 No. 10 s.w.g.	0·38	0·013
6·0	2	4 No. $\frac{1}{4}$ in. dia.	0·75	0·181
10·0	3	6 No. $\frac{1}{4}$ in. dia.	0·59	0·136

Mr D. J. Male, Civil Engineering Department, University of Adelaide

Mr Burhouse has contributed some useful experimental results in an important but neglected field. As he points out, however, more work will be required before firm design recommendations can be made.

58. I have recently carried out a numerical analysis of composite action between walls and beams using finite element methods.[6] These methods provide a more versatile approach to the problem than the lattice analogy used by Colbourne since it is possible to study the effects of anisotropy, non-linear stress/strain behaviour and shrinkage.

59. Fig. 12 shows the results obtained from the finite element analysis of a series of walls for which $H/L=0{\cdot}5$. These include some of the cases analysed by Colbourne and, where appropriate, his results (taken from Table 11) have been included for comparison. The agreement between the two methods is generally good. Values of E_{beam}/E_{wall} of 30 and 3 have been used and for each modular ratio beams of three different depths have been considered. From these results it will be seen that, as far as the compressive stress in the wall is concerned, this is influenced far

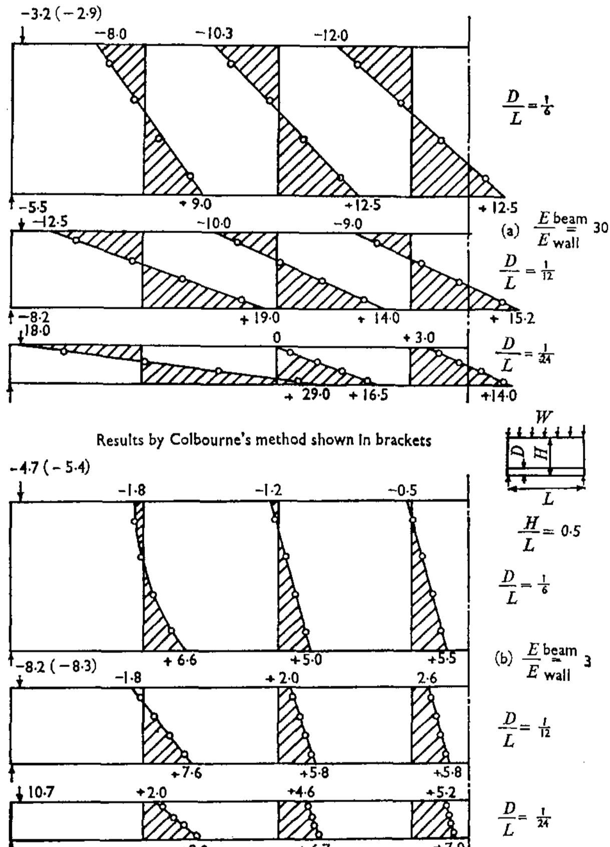

Fig. 12. Beam stresses for point supported wall (stresses in terms of σ/W)

more by the depth of the beam than by the modular ratio. It will also be noted that the horizontal stress in the beam is far from uniform, as would be expected if the beam acted solely as a tie, and for the very stiff beam the stress distribution corresponds, very nearly, to pure bending.

60. The experimental results of the Author's tests (6) and (7) confirm that the beam is subjected to both axial load and bending. On the basis of no tension in the concrete the reinforcement stresses in test (6) corresponding to a wall load of 20 tons are equivalent to an axial load of 4000 lb and a moment of 22 000 lb in. In test (7), which only differs from test (6) in the amount of bottom reinforcement in the beam, the corresponding values are 4300 lb and 38 000 lb in. From the finite element results for a modular ratio of 3 and D/L of $\frac{1}{12}$ an axial load of about 16 000 lb and a moment of 12 000 lb in. would be predicted. Thus the measured axial load is significantly lower than that predicted while the measured moment is considerably higher. It would appear likely, from these results, that significant tensile loads are carried by the concrete, and further work is indicated to determine the mechanism of failure.

61. In view of the effect of modular ratio on the stress distribution in wall and beam, could the Author provide experimental values of the modulus of elasticity for the brickwork used and for the reinforced concrete beam?

62. From the results so far obtained from the finite element analysis it would appear that the moment arm method of design suggested by Wood, although satisfactory for low modular ratios (e.g. brick wall on reinforced concrete beam), could seriously underestimate the beam stresses when applied to high modular ratios (e.g. lightweight concrete block walls). Design based on an equivalent bending moment would appear to provide a more rational approach. The coefficients used in determining the equivalent bending moments should take into account (*a*) modular ratio between wall and beam, (*b*) beam span: depth ratio, and (*c*) effect of door and window openings.

Mr Burhouse

I should like to thank the contributors for their interest in the Paper and for the information given of other work on composite action.

64. **Mr Morgan** has pointed out the possibility of using return ends to increase the load capacity of composite walls. The corners of the wall in test (10) were strengthened by using an engineering brick, though the failure load of the wall did not increase in proportion to the increase in strength of brickwork, and experimental values of the effects of return ends would be useful. Some tests are reported in reference 2 in which columns were built at the edges of the brickwork.

65. **Dr Amaratunga** suggests that friction in unbonded brickwork permits a certain amount of composite action. It was shown in test (1) that in the later stages composite behaviour depended entirely on frictional forces. No measurements were made of deflexions or of stresses during bricklaying, though some such readings obtained in earlier tests were reported by Wood.[1]

66. The analytical treatment carried out by **Mr Male** is a very welcome addition to the theory of the behaviour of composite walls, particularly since it caters for anisotropy and a non-linear stress/strain relationship. In all the tests reported, bond between wall and beam was prevented over part of the span by the inclusion of a building paper joint. At the load in tests (6) and (7) for which Mr Male has calculated forces, the concrete beams had cracked through their full depth. These considerations may explain, in part at least, the discrepancy between measured forces and those calculated.

67. No direct measurements were made of modulus of elasticity of either the brickwork or the concrete, but the value for brickwork could be taken as about 1×10^6 lbf/sq. in., and based on a knowledge of cube strengths and of the mix design a reasonable value for the concrete is of the order of 3×10^6 lbf/sq. in.

68. Mr Male has shown in Fig. 12 the variation in behaviour of the beam between almost pure bending and direct force. There is, I think, a danger of losing sight of the composite nature of the behaviour of these walls in using an equivalent bending moment for design of the beam, and when it is possible to finalize design rules it may be preferable to give values for both bending moment and direct force. It will be necessary also to design for the concentration of stress in the brickwork.

References

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