The Cracking, Deflection and Collapse Behaviour of a Series of Reinforced Brickwork Beams

T.G. Garwood, M.A., Ph.D., C.Eng., M.I.C.E., Senior Lecturer

A. Tomlinson, B.Sc.(Hons)., Research Assistant

Department of Civil Engineering,
Bolton Institute of Technology,
Bolton, England.

SUMMARY

This paper describes the construction and testing of a series of eight reinforced brickwork beams in which the quantities of tension and shear reinforcement were varied. The test performance of the beams is examined from the points of view of safety and serviceability, and the current design recommendations are assessed. An empirical load-deflection relationship for reinforced brickwork beams is proposed.

1. INTRODUCTION

During the past fifty years, the general behaviour of reinforced brickwork beams has been investigated in several research projects [1,2,3,4,5]. However, relatively little has been published with regard to deflection and cracking. If reinforced brickwork beams are to be used more widely as structural flexure members, then it would be desirable to be able to predict the extent of deformation and local damage that is likely to occur under service conditions. Therefore, a series of beam tests was planned in which particular attention would be paid to these serviceability limit states.

When designing the beams, certain factors were taken into account. Firstly, the construction of the beams had to be reasonably simple and practicable; secondly, the arrangement of brickwork had to be such that both tension and shear reinforcement could be accommodated in a straightforward manner; and, thirdly the number of test variables was to be kept to a minimum. With these considerations in mind, a suitable design was evolved. An important feature of the design was that the tension and shear reinforcement could be varied without having to alter the brickwork details.

2. CONSTRUCTION OF THE BEAMS

The beams were 3905 mm long, 328 mm wide and 290 mm, or four courses, in depth. The first two courses were laid in stretching bond and the top two courses in Quetta bond. The details of a typical beam are shown in Fig. 1.

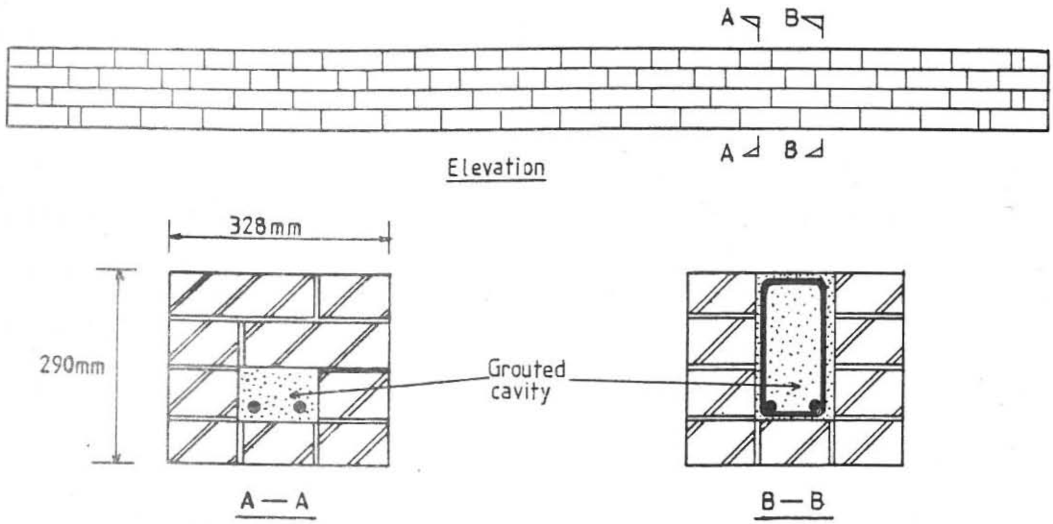

Fig 1

The beams were built on a steel channel supported centrally and at the ends. After applying a coat of mould oil to the channel, the two courses in stretching bond were laid. By omitting the central line of bricks in the second course, a longitudinal cavity, 130 mm wide and 75 mm deep was created; the reinforcement cage was then placed in this cavity. The upper two courses were then laid in Quetta bond, leaving the shear reinforcement in a series of pockets 130 mm wide and 67.5 mm long at 167.5 mm intervals. Because two types of bonding were used, care had to be taken when setting out the brickwork to avoid co-incident vertical joints in the second and third courses of the beam. After completion of the brickwork, the void surrounding the main reinforcement and the pockets enclosing the shear reinforcement were grouted.

There are several points about this beam design which are worth noting. Firstly, by laying the two lower courses in stretching bond, the line of the tension reinforcement is unobstructed by headers. Secondly, by using Quetta bond for the upper two courses there is adequate ceramic interlock in the beam and wire ties are not necessary. Thirdly, with this arrangement of brickwork, the volume of grout is minimised and thus the effects of grout shrinkage are reduced. Finally, it should be pointed out that, because of the headers in the top course, there were no longitudinal top bars to anchor the stirrups as in a conventional reinforced concrete beam.

With one bricklayer and one labourer, a beam could be built in one and a half days.

3. MATERIALS

3.1 Bricks

These were Class B engineering bricks, complying with BS 3921:1974[6] and manufactured by George Armitage and Sons Ltd. The bricks were nominally 215 mm X 65 mm X 102 mm and were perforated with three 30 mm diameter holes.

The average compressive strengths of the bricks, determined in accordance with BS 3921, were 118 N/mm^2, 52 N/mm^2 and 45 N/mm^2 on the bed, stretcher and header faces respectively.

3.2 Mortar

The mortar used for the beams was a $1:\frac{1}{4}:3$ ordinary Portland cement, hydrated lime and sand mix by volume. The average 28 day cube strength of the mortar was 21 N/mm^2.

3.3 Grout

After several trial mixes, the grout selected for the beams was a 1:2.5 cement:sand mix by volume, with a water cement ratio of 0.8. The average 28 day cube strength of this grout was 26 N/mm^2.

3.4 Reinforcing Steel

The tension reinforcement consisted of 12 mm, 16 mm and 19 mm ($\frac{3}{4}$ in. Imperial size) Helibond deformed cold worked bars; the shear reinforcement was bent from 8 mm plain mild steel bars. The properties of the reinforcement are given in Table 1 below.

Diameter & Type	Modulus of Elasticity (kN/mm^2)	Yield Stress (N/mm^2)	Ultimate Tensile Stress (N/mm^2)
Y12	205.8	512.2	616.6
Y16	194.5	453.5	572.9
Y19	196.8	475.4	599.6
R8	205.2	435.6	520.0

Table 1 Properties of the reinforcement.

4. THE BEAM SERIES

The details of the reinforcement in the beams are given in Table 2. The quantity of main tension steel expressed as a percentage of the breadth times the effective depth (100 As/bd) varied from 0.34 to 1.33.

Also shown in Table 2 are the design moments of resistance of each beam as calculated using the formulae in the Design Guide [7] with the appropriate characteristic strengths and partial safety factors (γ_m values) being used. In Table 2, Mb refers to the moment of resistance based on the flexural compressive strength of the brickwork and Ms is the moment based on the tensile strength of the steel. In these calculations, the characteristic compressive strength of the brickwork was taken as $0.4 \times 24 = 9.6$ N/mm^2, the factor 0.4 being used because the direction of the compressive stress was not perpendicular to the bed joints.

Beam	Main Steel	100 $\frac{As}{bd}$	Shear links	Design moments of resistance (kNm)	
				Mb	Ms
1	2 Y12	0.34	R8 at 167 mm	20.24	15.51
2	2 Y12	0.34	None	20.24	15.51
3	2 Y12	0.34	None	20.24	15.51
4	2 Y16	0.62	None	19.64	23.55
5	2 Y16	0.62	R8 at 167 mm	19.64	23.55
6	2 Y19	0.89	None	19.24	27.06
7	2 Y19	0.89	R8 at 167 mm	19.24	27.06
8	3 Y19	1.33	R8 at 167 mm (in pairs)	19.24	30.20

Table 2 Details of Test Beams.

In the usual limit state design process the smaller design moment of resistance would be slightly greater than, or equal to, the moment produced by the design ultimate load. With γ_f factors of 1.4 and 1.6 for the dead and imposed loads respectively, the design ultimate load would be approximately one and a half times the service (working) load. In other words, the service moment would be about two thirds of the design moment of resistance. Using this approach, nominal working loads were calculated for the test beams. For the loading arrangement used, the nominal working load was taken as the value which, together with the self weight, produced a mid-span bending moment equal to two thirds of the moment of resistance. For each beam, two nominal working loads were obtained: one corresponding to the flexural compressive strength of the brickwork and the other corresponding to the steel strength. The larger of these values is referred to as the higher working load (H.W.L.) and the lower value is referred to as the lower working load (L.W.L.).

According to the Design Guide, of the eight beams in the series, beams 4 and 5 would be the closest to a balanced section; beams 1, 2 and 3 would be considered under-reinforced and beams 6, 7 and 8 over-reinforced.

In beams 1, 5 and 7 the shear reinforcement consisted of 8 mm links at 167.5mm centres, this quantity providing an Asv/sv value* slightly less than the minimum nominal amount suggested in the Design Guide. In beam 8, the shear reinforcement was twice that in beam 7.

When the first two beams had been completed, another bricklayer was employed to build the remainder of the series. Thus, in order to assess the effects of possible differences in workmanship, the quantity of reinforcement in beams 2 and 3 was kept the same.

* Asv = cross-sectional area of legs of a link
sv = spacing of links

5. TESTING OF THE BEAMS

The first seven beams of the series were tested in a 500 kN capacity "self-straining" test frame. The load was applied by a hydraulic jack, via a proving ring and spreader beam, at two points on the beam; the loading configuration is shown in Fig. 2. The eighth beam was tested in a 2500 kN capacity Avery Denison universal machine, the same four point loading being applied.

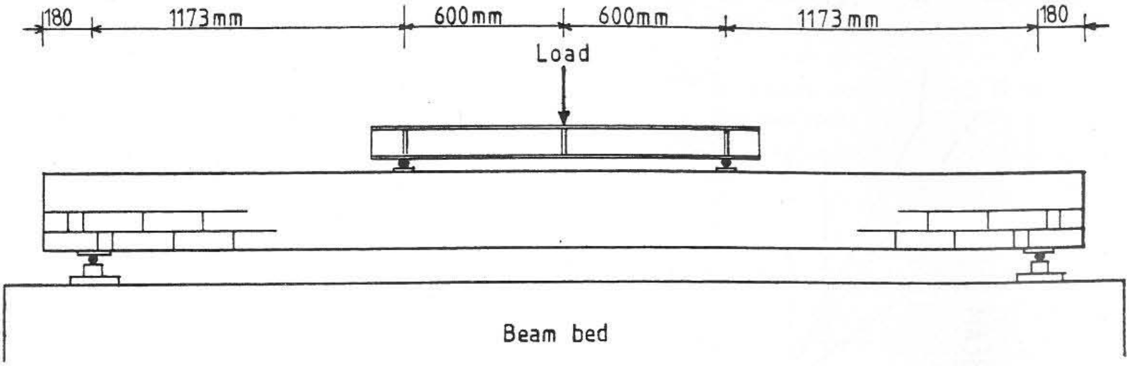

Fig 2 Loading arrangement

All beams were subjected to an initial loading cycle in which the load was increased from zero to the higher working load and then returned to zero. The load was applied in suitable increments at each of which the central deflection, steel and brickwork strains and crack widths were measured. The same procedure was used in the second loading when the beams were tested to destruction.

The central deflection at each side of the beam was measured with a dial gauge. The brickwork strains at mid-span were measured with a 900 mm Demec gauge, there being seven gauge lengths on each side of the beam. The strains in the tension steel were obtained from electrical resistance gauges. Crack widths were measured with a microscope having a magnification of 40; in some cases the crack width could be calculated indirectly from the Demec readings.

6. DISCUSSION OF TEST RESULTS

6.1 Deflection

The experimental load-deflection curves for the eight beams are shown by the unbroken lines in Fig. 3. These curves refer to the initial loading from zero to the higher working load; above this, the curves refer to the second loading.

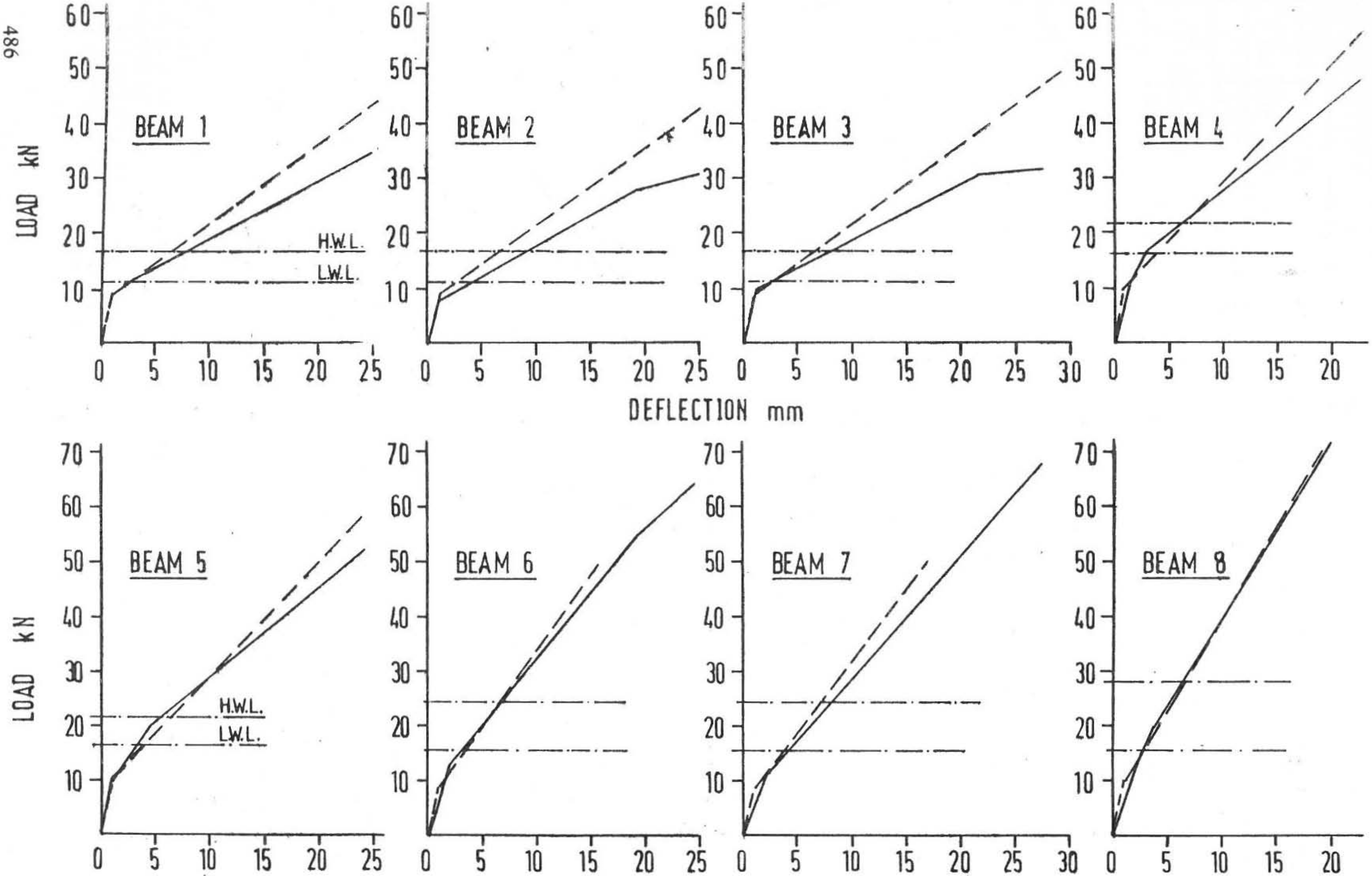

Fig.3 LOAD-DEFLECTION CURVES (Experimental ——— Proposed empirical - - - - - -)

Generally, the curves are approximately bilinear. The change of gradient occurs at the instant of the first substantial flexural cracking in the brickwork. Fig. 3 shows that this cracking took place at an applied load of about 10 kN and this is equivalent to a flexural tensile stress of approximately 2 N/mm^2. The experimental load-deflection curves also show how increasing the quantity of tension steel results in a greater flexural rigidity of the cracked section.

The maximum permissible deflection of, say, $\frac{\text{span}}{250}$, equal to 14 mm, did not occur in any of the beams until the applied load was well in excess of the higher working value. For example, in beam 1, the deflection at the higher working load of 16.9 kN was 8.5 mm, whereas the permissible deflection of 14 mm was not reached until the applied load was 22kN.

Using the experimental curves, an idealised empirical load-deflection relationship was developed. This is as follows:

$$\Delta = \beta \left[\frac{W_1}{E_1 I_1} + \frac{(W - W_1)}{E_2 I_2} \right] L^3$$

where Δ = deflection at mid span.

W = applied load (assumed greater than W_1)

W_1 = cracking load, i.e. the load which, together with the beam self weight, produces a flexural tensile stress of 2 N/mm^2.

E_1 = modulus of elasticity of the brickwork in the uncracked section.

E_2 = modulus of elasticity of the brickwork when the section is cracked.

I_1 = the second moment of area of the uncracked brickwork section.

I_2 = transformed second moment of area of the cracked section based on a modular ratio equal to $\frac{E_s}{E_2}$ where E_s is the modulus of elasticity of the steel.

β = coefficient whose value depends on the loading arrangement.

L = span of the beam.

After studying the brickwork strains in the initial loading stages, the value proposed for E_1 was 12,000 N/mm^2. Further, by examining the neutral axis depth of the cracked sections, it was found that a suitable value for E_2 was 10,000 N/mm^2. These values of E_1 and E_2 are used in the above equation and the resulting empirical load-deflection curves are shown by the broken lines in Fig. 3. Generally, there is a reasonably close correlation between the experimental and idealised empirical curves.

6.2 Cracking

Fig. 4 shows the variation in the maximum crack width with applied load for the eight beams in the series. Although there is a considerable scatter in the results, it can be seen that for a given load, beams with higher percentages of steel tend to have smaller crack widths. For example, at an applied load of 20 kN, the maximum crack widths in beam 1 ($p = 0.34\%$), beam 5 ($p = 0.62\%$) and beam 7 ($p = 1.33\%$) are 0.46 mm, 0.38 mm and 0.14 mm respectively.

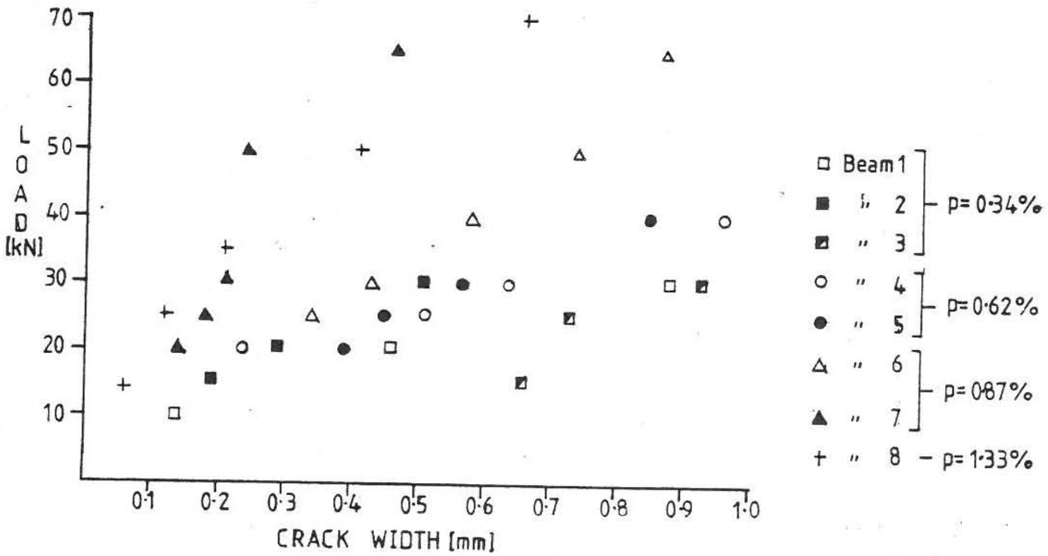

Fig 4 Crack widths

Table 3 gives the maximum crack widths in the beams at the two working load levels. At the lower working load (L.W.L.), the largest crack width was 0.14 mm and this occurred in beam 2. At the higher working load (H.W.L.), the assumed maximum permissible crack width of 0.30 mm was exceeded in beams 1, 3 and 5.

Beam	L.W.L. (kN)	Crack width at L.W.L. (mm)	H.W.L. (kN)	Crack width at H.W.L. (mm)
1	11.5	0.12	16.9	0.40
2	11.5	0.14	16.9	0.20
3	11.5	0.10	16.9	0.66
4	16.2	None	20.6	0.24
5	16.2	0.04	20.6	0.38
6	15.8	0.02	24.6	0.26
7	15.8	0.12	24.6	0.18
8	15.8	0.06	28.2	0.16

Table 3. Maximum Crack Widths.

6.3 Collapse

The values of the maximum loads carried by the beams and the corresponding modes of failure are given in Table 4 below.

Beam	Main Steel	Shear Links	Max.Load (kN)	Mode of Failure
1	2 Y12	R8 at 167mm	38.6	Yield of steel, very little cracking outside c.m. zone*
2	2 Y12	None	41.9	"
3	2 Y12	None	41.6	"
4	2 Y16	None	78.0	Yield of steel
5	2 Y16	R8 at 167mm	76.8	Yield of steel, plus some sign of brickwork crushing
6	2 Y19	None	85.8	Yield of steel plus sudden shear failure
7	2 Y19	R8 at 167mm	101.8	Yield of steel followed by sudden shear failure
8	3 Y19	R8 in pairs at 167mm	93.6	Sudden shear failure at one end

Table 4. Collapse loads and modes of failure.

The first three beams failed by yielding of the tension steel, there being very little difference in the maximum loads. By comparing beams 1 and 2, it is evident that the shear reinforcement in beam 1 had little, if any, effect on its performance; when comparing beams 2 and 3, which were built by different bricklayers, it appears that the ultimate behaviour was not affected by any differences in workmanship.

In beams 4 and 5, yielding of the steel was again the primary cause of failure. In beam 5 there was also some slight crushing of the brickwork before the maximum load was reached. The maximum loads carried by beams 4 and 5 were practically identical; this suggests that the shear reinforcement in beam 5, as in beam 1, was not brought into action.

Beam 6, which contained no shear reinforcement but a greater area of main steel, failed suddenly in diagonal tension at a load corresponding to a shear stress of 0.74 N/mm^2. In beam 7, which contained the same main steel as beam 6, the apparent effect of the shear reinforcement was to increase the load capacity by nearly 19 per cent.

The final beam in the series, beam 8, contained fifty percent more tension steel and twice as much shear steel as beam 7. It was therefore surprising when this beam failed suddenly in shear at a load of 93.6 kN. The apparent cause of this unexpected failure was poor adhesion between the mortar and the bricks.

The overall factor of safety against collapse could be defined as: Collapse moment divided by working load moment. Two values for this factor, using the lower (L.W.L.) and higher (H.W.L.) working load respectively, were calculated for each beam; these values are given in Table 5 overleaf.

* c.m.: constant moment

Table 5. Factors of Safety.

Beam	Factors of Safety	
	Collapse Moment / Moment at L.W.L.	Collapse Moment / Moment at H.W.L.
1	2.53	1.94
2	2.72	2.08
3	2.71	2.07
4	3.77	3.14
5	3.72	3.10
6	4.20	2.97
7	4.93	3.51
8	4.56	2.90

The lower working load was based on the steel strength in beams 1, 2 and 3 and on the compressive strength of the brickwork in the remaining five beams. The relatively high factors of safety in beams 4, 5, 6, 7 and 8 in respect of the lower working load indicate that the Design Guide is rather conservative with regard to the flexural compressive strength of the brickwork.

6.4 Stresses in the brickwork and steel

The neutral axis depth and the maximum flexural compressive strain in the brickwork, as obtained by plotting the Demec readings for the last fully recorded load stage, are given for each beam in Table 6. (In the interest of personal safety, the measuring of brickwork strains was discontinued when it was considered that collapse of the beam was imminent.)

Also given in Table 6 are the stresses in the brickwork and steel at the last recorded load stage. Certain assumptions were made when calculating these stresses. As the largest compressive strain in the brickwork recorded in these tests was significantly less than the strains corresponding to maximum stress obtained by other investigators (8), it was considered reasonable to assume a linear stress distribution in the brickwork. Assuming also that there was no tension contribution from the brickwork below the neutral axis, the internal lever arm was taken as the effective depth minus one third of the measured neutral axis depth. Hence, by equating the total applied bending moment to the product of the steel force and the internal lever arm, the steel force and thus the steel stress could be calculated. Finally by equating the compressive and tensile forces, the compressive stress in the brickwork was calculated. An attempt was made to determine the compressive strength of the brickwork by crushing prisms 102 mm thick, by 290 mm (four courses) wide and 552 mm high, which were loaded so that the compressive stress was applied to the header (end) faces of the brick. The average compressive strength obtained was 25 N/mm^2.

Table 6. Flexural stresses in brickwork and steel.

Beam	Values at last recorded load stage					Failure Load (kN)
	Maximum compressive strain in brickwork (Microstrains)	Neutral axis depth (mm)	Maximum compressive stress in brickwork (N/mm^2)	Tensile stress in main steel (N/mm^2)	Load (kN)	
1	819	43.8	15.9	506	30	38.6
2	675	42.7	16.3	505	30	41.9
3	776	41.9	16.6	504	30	41.6
4	1033	57.4	19.7	461	50	78.0
5	999	55.9	20.1	459	50	76.8
6	1060	69.5	22.8	458	70	85.8
7	1414	75.0	21.3	463	70	101.8
8	1650	88.0	23.6	400	90	93.6

7. CONCLUSIONS

1. The tests have shown that cracking is likely to be a more critical serviceability limit state than deflection. In the working range of loads, there is a reasonable correlation between the deflections recorded in the tests and those calculated by using the proposed empirical load-deflection relationship. Cracking, however, is a more unpredictable phenomenon and the test results show a large scatter. In general, for a given bending moment, the crack widths are greater in beams with smaller quantities of reinforcement and it is reasonable to assume that the crack width is related to some extent to the steel stress. Therefore, when considering the cracking limit state in design, there may be a case for restricting the steel stress at working load; this could be achieved by providing an area of steel which is greater than that required for the ultimate limit state.

2. The Design Guide recommendations appear to be rather conservative with regard to the design moment of resistance based on the flexural compressive strength of the brickwork. By using, in effect, a smaller γ_m value for the brickwork, a more realistic value for the design moment of resistance would be obtained, and a greater quantity of tension steel would be required for a balanced section.

3. The shear reinforcement in the test beams was effective provided that there was good adhesion between the mortar and the bricks. With unsatisfactory adhesion, it is possible that shear failure will occur before the shear reinforcement is fully utilized.

ACKNOWLEDGEMENTS

The Authors would like to thank George Armitage and Sons Limited for providing the bricks for the project. The work was carried out in the Civil Engineering Department of Bolton Institute of Technology and the assistance given by the Technical Staff is gratefully acknowledged. The Authors would also like to thank Dr. J. Morton of the Brick Development Association, D. Foster of Structural Clay Products Limited and S. Bell of George Armitage and Sons Limited for their help and encouragement.

REFERENCES

1. WITHEY, M.O. "Tests of Brick Masonry Beams." ASTM Proceedings 33, Part II, 1933.

2. THOMAS, F.G. and SIMMS, L.G. "The Strength of Some Reinforced Brick Masonry Beams in Bending and Shear." Structural Engineer 17, July, 1939.

3. JOHNSON, F.B. and THOMPSON, J.N. "Correlation of Tests of Masonry Assemblages with Strength Characteristics of Reinforced Masonry Beams." Designing, Engineering and Constructing with Masonry Products. Gulf Publishing, Houston, Texas, 1969.

4. SUTER, G.T. and HENDRY, A.W. "Shear Strength of Reinforced Brickwork Beams." Structural Engineer, 53, (6), June, 1975.

5. GARWOOD, T.G. and TOMLINSON, A. "The Design, Construction and Test Performance of Reinforced Brickwork Beams Containing Tension and Shear Reinforcement." Paper presented to the Seventh International Symposium on Load-bearing Brickwork. British Ceramic Society, London, November, 1980.

6. BRITISH STANDARDS INSTITUTION BS 3921 1974. Clay Bricks and Blocks.

7. BRITISH CERAMIC RESEARCH ASSOCIATION. Design Guide for Reinforced and Prestressed Clay Brickwork. Special Publication No. 91, 1977.

8. POWELL, B. and HODGKINSON, H.R. "Determination of Stress Strain Relationship of Brickwork." British Ceramic Research Association Technical Note No. 249, January, 1976.