THE BEHAVIOUR AND STRENGTH OF BRICK AND REINFORCED CONCRETE COMPOSITE WALL BEAMS

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ABSTRACT This paper describes 69 tests of brick and reinforced concrete composite wall beam specimens. Test results are analyzed and discussed. The parameters of the test series involve 4 wall height-span ratios (H1/L), 5 reinforced concrete beam height-span ratios (h/L), 9 shear-span ratios (a/H) and various steel percentages, masonry strengths, flange wall widths and loading types. Test results have made clear the deformation and structural behaviour of the composite wall beams under loads up to failure. Various modes of failures were observed and clarified. Expressions for the prediction of flexural strength, local bearing strength and shear-resisting strength by considering the composite action, have been theoretically and statistically formulated in accordance with the test results.

1. INTRODUCTION

Wall beams are load carrying members which consist of the reinforced concrete beam (called supporting beam) and its upper brick wall. Due to the composite action of the supporting beam and the brick wall, wall beams possess good structural behaviour and economic effect, and they have made much emphasis by the people gradually. They are widely used in building construction such as the multistorey residence with ground floor for shopping, the foundation beam to support brick masonry, the tie beam and lintel over window or door openings.

Methods to design wall beams which were adopted for years are the elastic foundation beam method by Jemochkin, lintel method and the equivalent moment method by Wood. But the structural calculation based on all these methods simply regards the supporting beam as a flexural member and does not consider the load carrying characteristics of the brick wall. Although researchers in various countries have investigated wall beam behaviour for some time, due to the lack of systematic tests and studies and the complexity of the problem, the understanding of wall beam behaviour, especially when cracks have appeared, still remains imperfect. Theory and design methods for wall beams are not yet well established.

This paper describes the results of 69 model tests of clay brick and reinforced concrete composite wall beams. By test, it has made clear the regular change of the internal force, deformation and the modes of final failure under self-weight and loadings. Test results have made clear the deformation and structural behaviour of the composite wall beams under loads up to failure. Various modes of failure were observed and clarified. Based upon the test results, the authors suggest a method of strength calculation for wall beams by considering the composite action of the supporting beam and the brick wall.

2. THE TEST MODELS AND TEST METHOD

All of the test models were of the same span L of 240 cm. The thickness of walls were 11.5 cm and 24 cm. The parameters of the test series involve the wall height-span ratios (H1/L), the reinforced concrete beam height-span ratio (h/L), the reinforcement percentage in the beam, the width of flange wall (B), shear-span ratio (a/H) and masonry strengths R and types of loading, see Table. 1. The loading types and the distributions of the survey

points are shown in Fig. 1.

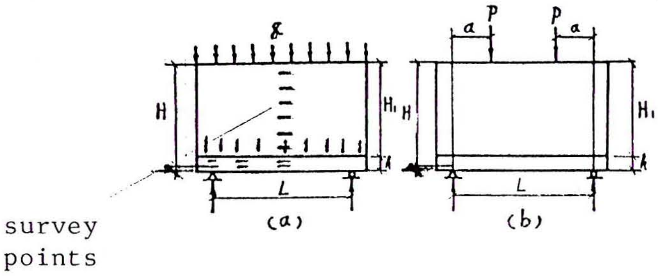


Fig. 1.

3. TEST RESULTS, ANALYSIS AND DISCUSSION

3.1 The characteristics of the composite action of wall beams under external loadings

The characteristics of the composite action of wall beams under external loadings may be described by means of the deformation, the distribution of internal force, the occurrence and development of cracks and the patterns of failure in wall beams. They are now described as follows:

(a) The strain distribution in the vertical cross section.

The distribution of horizontal strain ε_x of the wall beams in vertical cross section reflect the rule of distribution of stress σ_x. Fig. 2 shows the ε_x distribution in the vertical section of the centre span with models L5-1 (H1/L=0.25), L5-2 (H1/L=0.5) and L5-3 (H1/L=0.75) under the uniform total loads of 4T and 6T respectively (about 40% of the failure loads). These continuous curves of distribution ε_x reveal that the upper section of the wall beam is principally in compression and that more than half the height of the supporting beam section is in tension. The resultant forces of the compression region and the tension region form a couple to act against the external loading.

Tests make clear that in wall beams with masonry of rather high strength, the position of the neutral axis basically lies within the wall region. Only in those wall beams with a low brick wall and the masonry strength not so high, the position of the neutral axis will be in the supporting beam. This is the status of the test models before cracking. When the loads were increased until the cracks appeared, the neutral axis rose and tension force was taken gradually by the steel bars. The neutral axis goes up as the cracks appear and then develop, and the length of internal couple arm is increased accordingly. Ordinarily the couple arm length is more than half the height of wall beam, so the longitudinal steel bars in the supporting beam can be reduced considerably. Test results show that the wall beam which consists of the brick wall and the supporting beam is a composite load carrying member, during the whole process of loading.

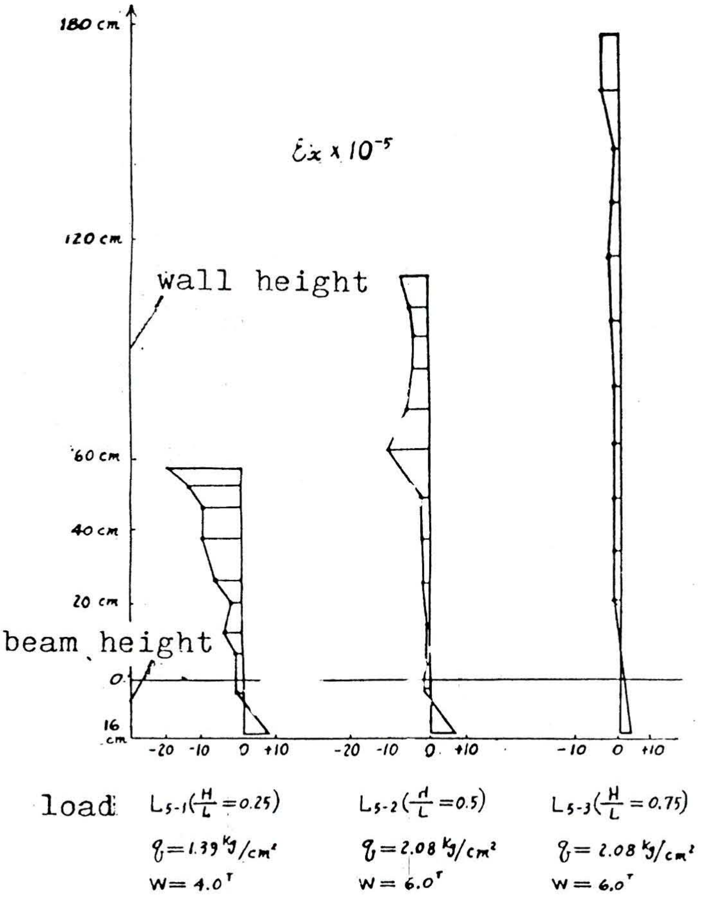


Fig. 2. The ϵ_x distribution in vertical section of centre span

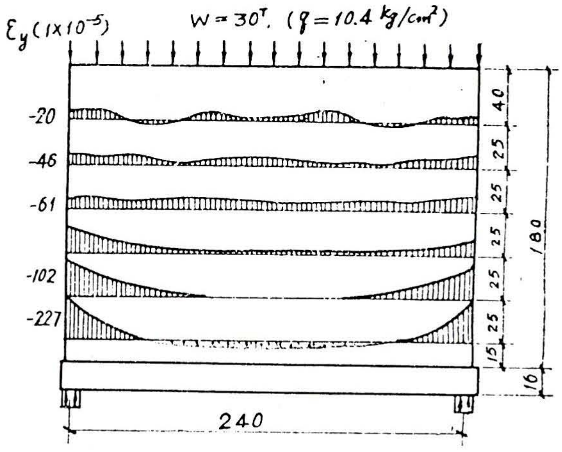


Fig. 3. The ϵ_y distribution on the horizontal section

(b) Distribution of vertical strain in the wall

The distribution of vertical strain ϵ_y along the height of the wall reflects the rule of distribution of the vertical stress σ_y in the horizontal section of the wall. As a typical example, Fig. 3 shows the distribution of vertical strain ϵ_y on the horizontal section at every 25 cm along the wall height which were recorded from the test of model B-12 of H1/L=0.7 under loading $q=10.4kg/cm^2$. It can be seen from Fig. 3 that the distribution of ϵ_y is rather even on sections at heights above half of the wall height, and that on sections below the half height, the value of ϵ_y at the wall end is increased downward section by section and the distribution of ϵ_y is similar to a triangle. The distribution of ϵ_y along the height of the wall is fundamentally coincided with the distribution of ϵ_y obtained from deep beam theory. At the centre span on the top of the supporting beam, the σ_y in a small region of the masonry calculated by theory and finite element method, is in tension, the value of which will increase with loading. This is the cause of the horizontal cracks in masonry. When the horizontal cracks on top of supporting beam occur or the diagonal cracks in masonry occur with the increase of loading, the length of the compression region at the ends decreases gradually, and the compression strain at supports increases rapidly and a triangular ϵ_y distribution is formed in a range 1 to 0.15L to 0.25L.

(c) The stress variation in longitudinal steel bars of the supporting beam.

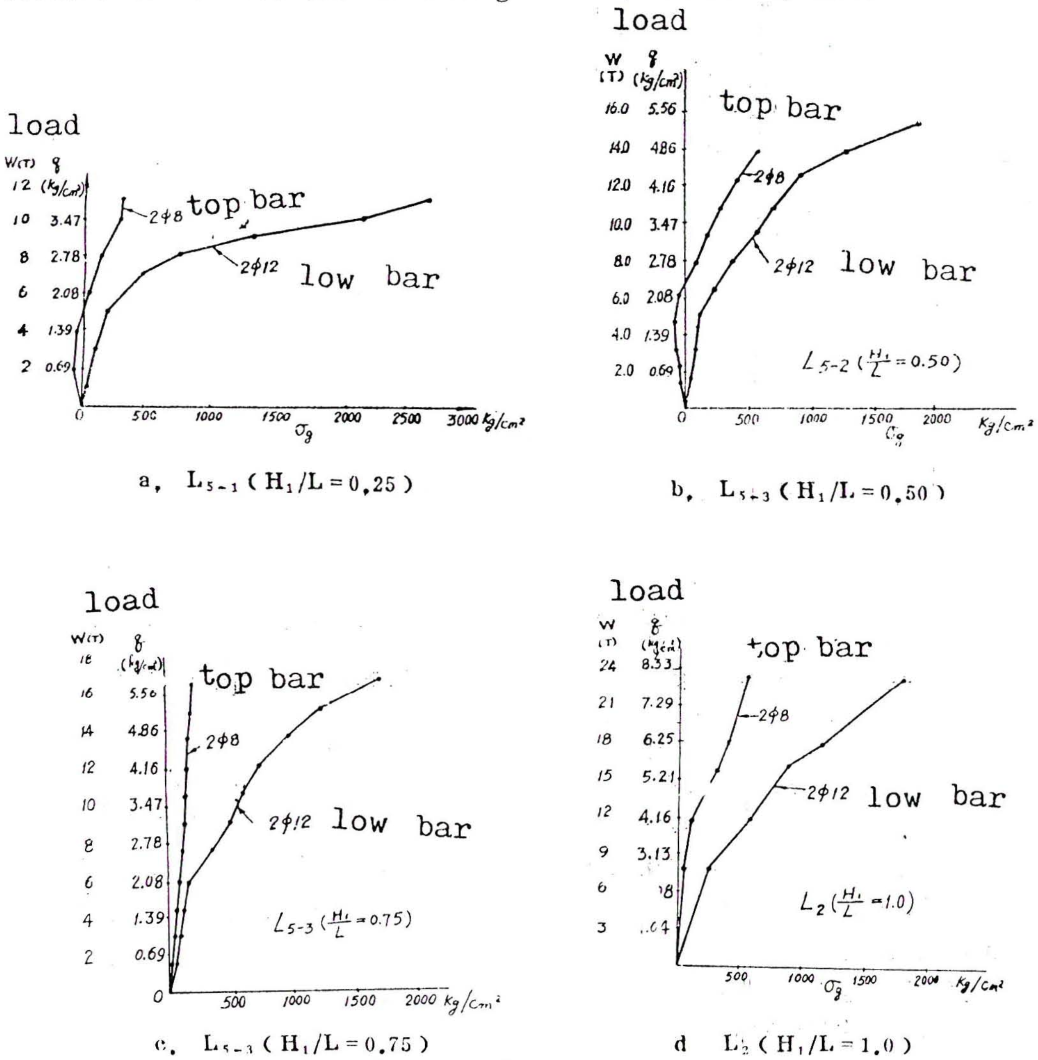
The typical w(loading)- σ_g (stress in steel bar) curves of models with different ratio of H1/L are shown in Figs. 4a, b, c, and d.

It can be seen in Fig. 4a and b, that for wall beam with $H1/L \lesssim 0.5$, before the supporting beam begins to crack the neutral axis lies below the top bars, and in this case, the top bars are in compression. Once the cracks appear and extend, the neutral axis goes up and the top bars change to be in tension.

For wall beam with $H1/L \geqslant 0.75$, all the longitudinal bars in the supporting beam are in tension and the neutral axis always lies within the masonry wall region. For the same loading conditions, in the low wall, because the couple arm is comparatively short, so the stress in steel bars will be higher. Test shows that with the occurrence and extension of the diagonal cracks in the masonry wall and the increase of the vertical cracks in the supporting beam, the stresses in bars of the supporting beam gradually tend to the same value along the span length. In this case the bar acts as a tie rod, and the wall beam acts as a tie-rod arch.

(d) Deflection of wall beams.

Due to the composite action of the supporting beam and masonry wall the stiffness in wall beam is very large, the amount of the deflection obtained from the test at centre span of the wall beam is very small. For all models, the amount of deflections at failure are in a range of L/500 to L/1000.



a, L_{5-1} ($H_1/L = 0.25$) b, L_{5-3} ($H_1/L = 0.50$) c, L_{5-3} ($H_1/L = 0.75$) d L_2 ($H_1/L = 1.0$)

Fig. 4. W–σ_g curves

(e) Occurrence and extension of cracks and the pattern of failure.

All models in the test behave basically as elastic composite members before they begin to crack.

In each test model the initial vertical cracks occur generally at centre or ¼ to 1/3 span of the supporting beam, regardless of the value of H1/L and type of loading. The load the vertical cracks occur in the supporting beam is 40%-70% of ultimate load, mostly about 50%. The extension of the beam cracks and the occurrence and extension of wall cracks are closely related to the bar percentage in the beam, strength of masonry wall, value of H1/L, and type of loading. Different failure patterns are obtained. It is now described separately as follows:

(i) Failure due to bending.

For a test model with high strength in the masonry wall and low bar percentage in the supporting beam, with test load increased a number of vertical cracks will occur in the supporting beam. The wider cracks extend throughout the whole section of the supporting beam and also extend up into the masonry wall. The bars in the beam now take all the tension forces. If the stress in the bars reaches the yield limit, the cracks enlarge rapidly and extend upward, the deflection increases quickly and the wall beam fails. Typical examples of failure in bending are shown in Fig. 5. No crushing of the masonry was observed in the compression zone for all the test models.

The failure due to bending of the wall beam often takes place in models made of high strength masonry wall and supporting beam with insufficient reinforcement - known as a "strong wall with a weak beam".

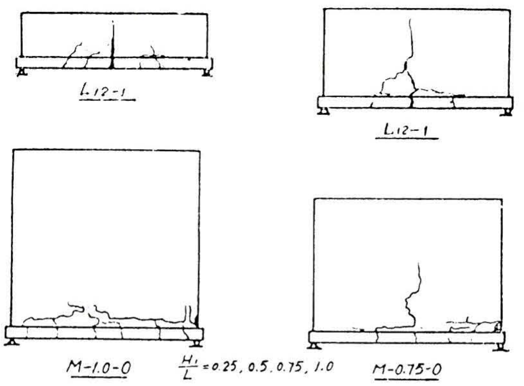


Fig. 5. Typical examples of failure in bending

(ii) Failure due to local compression.

For a test model of brick wall with its height-span length ratio H1/L greater than or equal to 0.5 and with low strength, when test loads are applied gradually, some small vertical cracks will first appear on faces of brick wall at supports due to the highly concentrated pressure. The cracks which occurred in bricks 2-4 layers away from the top surface of the supporting beam are in a range of ¼ to ½ brick length, as shown in Fig. 6. The width and length of cracks as well as the number of cracks will increase with loading, and numerous rhombus shapes are formed. The cracked bricks are then stripped and crushed to pieces and finally lead to failure. In some models after vertical cracks occurred in the supporting beam, horizontal cracks develop and in the brick wall along mortar joints 1-2 layers apart from the top of the supporting beam at the

centre span as the loading increases. The test result shows that the occurrence of the horizontal cracks makes no significant effect on the ultimate loading, since the horizontal cracks themselves are not the pattern of failure. The test result also shows that if the flange walls are built at the ends of the wall beam, the intensity of bearing pressure will be reduced due to the increase of bearing area, thus the strength against local compression considerably increases.

The load at the initial vertical cracking is approximately 70–90% of the failure load under local pressure and the load of the initial horizontal cracking varies between 45–85%, mostly about 80%.

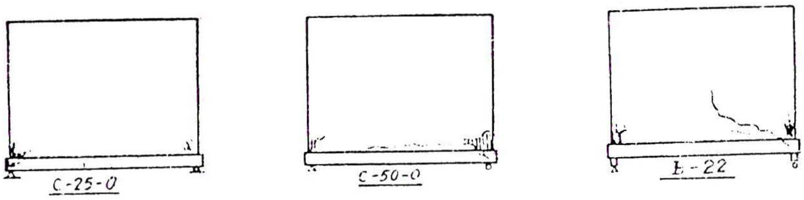


Fig. 6. Examples of failure due to local compression.

(iii) Failure due to shear

(a) Tensile shear (diagonal tension) failure.

When the load on test model is increased step by step, diagonal cracks will appear in the brick wall because the principal tensile stress developed in it exceeds its tensile strength. These cracks step upwards along mortar joints toward the middle span and downwards to the supports. In some cases, several cracks will occur simultaneously, one of which is the principal crack extending longer and wider than the others. When the principal crack reaches the top, the wall beam fails. This is one pattern of shear failure in the wall beam called diagonal tension failure (tensile shear) (see Fig. 7). Ordinarily failure in this pattern takes place in test models whose $H1/L$ value is less than 0.5 and the brick work quality is poor and whose bearing capacity is rather low.

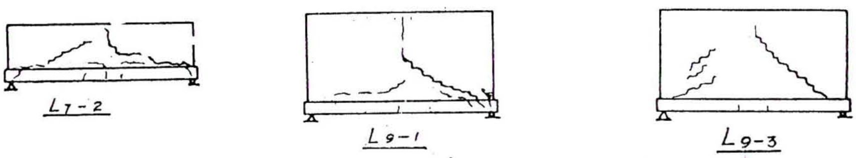


Fig. 7. Examples of failure due to tensile shear

(b) Compressive shear (diagonal compression) failure.

When the wall beam test models are of good quality and $H1/L \geq 0.5$ diagonal cracks will develop from wall top at points L/4 to L/3 away from the supports and they will extend continuously downwards toward the supports. These cracks extend throughout the bricks and mortar joints, most of them are parallel and one or two of them are the principal cracks. If diagonal cracks extend downward to reach the supports or the brick wall is crushed at the centre, the test model will fail. Shear failure in this pattern is called diagonal compression failure (compression shear) (see Fig. 8). The bearing capacity of this pattern is greater than that of diagonal tension failure.

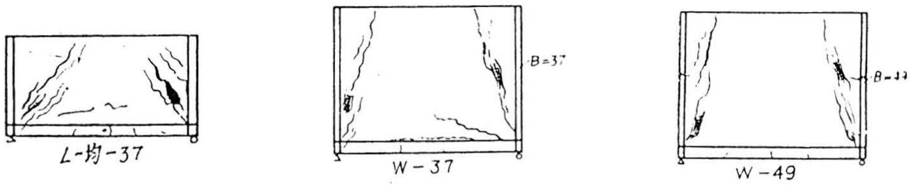


Fig. 8. Examples of failure due to compressive shear

(c) Failure due to shear under concentrated load.

When the wall beam models are tested under concentrated loads, one or more continuous diagonal cracks throughout the point of loading and supports will occur suddenly, which results in brittle failure. Then load carrying capacity is quite low, and then shear strength is approximately ½ to 1/3 of that of models under uniform loading, see Fig. 9.

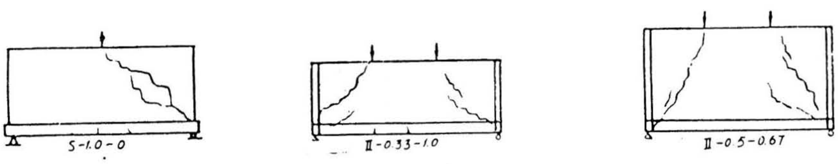


Fig. 9. Failure due to shear under concentrated loads

3.2 Load carrying mechanism

According to the above statements, the performance of wall beam under test during the whole loading process, reflects the fact that the R.C. supporting beam and the brick wall always act well together to form a load carrying mechanism composed of arch (brick wall) and tie rod (R.C. supporting beam). The loads on the top of wall beam are transferred by arch action of the brick wall down toward supports and the supporting beam acts as a tie rod. The failure of wall beam is essentially a result of failure in certain position of the tie rod-arch action mechanism, as shown in Fig. 10.

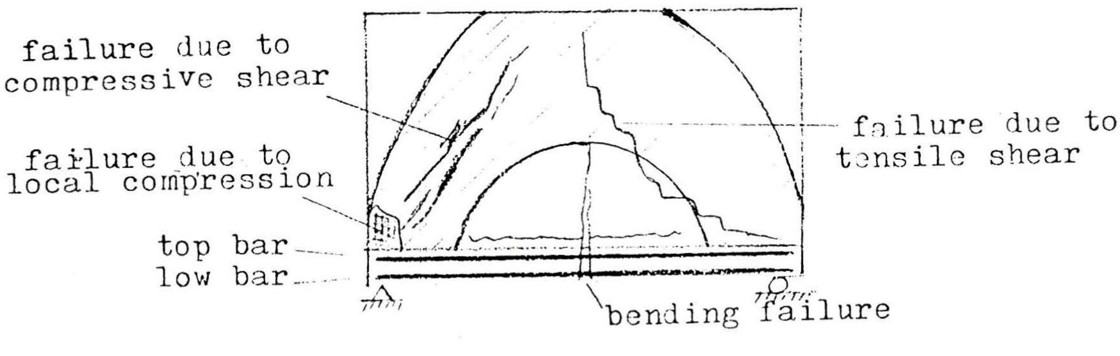


Fig. 10. The failure of wall beam

Therefore a reasonable design of the wall beam should be based on the combined load carrying behaviour to account separately for its strength due to bending, shearing and local compression. The appropriate means of construction must be adopted.

4. CALCULATION OF STRENGTH

4.1 Bending strength

It is revealed both from the ÷ x graph and the pattern of bending failure that the moment M caused by external loads was resisted by a couple formed by a resultant compressive force D in the upper portion and a resultant tensile force T in the lower portion of the vertical section at centre span (see Fig. 11). All the tensile force was taken by the longitudinal steel in the supporting beam after cracks developed. The steel reinforcement yielded as soon as the beam failed. Let the couple arm Z be equal to H, then H is the distance from the centroid of reinforcement up to the top of brick wall and it is called the effective depth (generally take $H = H1+h$), thus

$$M = T * Z = T * (\gamma * H) \qquad (1)$$

or $$T = M / (\gamma * H)$$

in which

M-- Max. moment caused by external loads which involve selfweight of wall and loads on top in wall beam

T-- total tensile force in longitudinal steel s+Ag * g

γ-- factor of couple arm, taken by test

$\gamma = 0.1\ (4.5+L/H)$
when $H/L \geqslant 1$, $Z = \gamma H = 0.55L$, see Fig. 12.

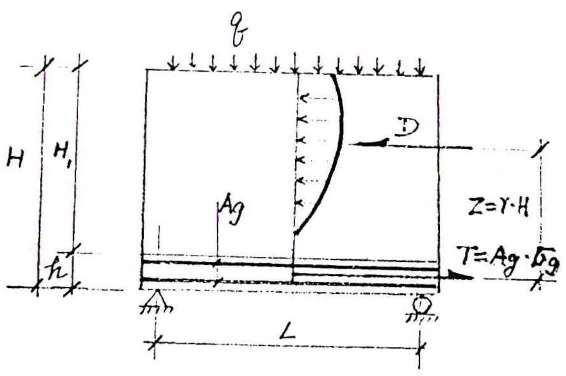


Fig. 11.

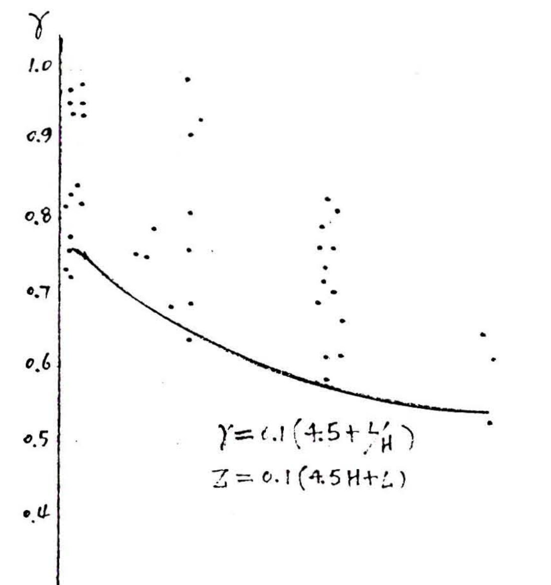


Fig. 12. H/L --γ curve

4.2 Strength of local pressure

The strength of local pressure is affected by factors such as the degree of concentration of vertical compression at supports, by the enhancement of compressive strength in brick wall under local pressure, as well as the width of flange B and the length of support W.

The factor of stress concentration C is the ratio of maximum compressive stress on the supporting beam, maximum σ_y, to the loading intensity q, (see Fig. 13).

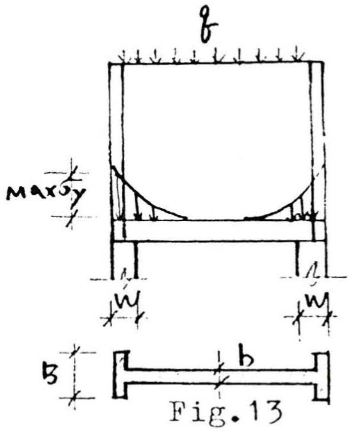


Fig. 13

$$C = \frac{\max\sigma_y}{q} \tag{2}$$

From test, the value of C lies between 3.0 and 5.0 mostly about 4.0. The enhancement factor due to local pressure is the ratio maximum σ_y to R (compressive strength in brick wall),

$$\gamma = \frac{\max\sigma_y}{R} \tag{3}$$

From 16 test models in typical local pressure failure test, the value of γ has an average value of $\gamma = 1.507$.

The ratio of γ to C is called local pressure factor,

$$\beta = \frac{\gamma}{C} = \frac{\max\sigma_y}{R} \Big/ \frac{\max\sigma_y}{q} = \frac{q}{R} \tag{4}$$

The above equation clearly states that β directly shows the relation between the wall strength R and the ultimate load intensity q. From test results, the value β of wall beam without flange varies from 0.31 to 0.414, and the strength of local pressure can be expressed as

$$q = \beta * bR \tag{5}$$

in which q is the intensity of linear load.

In calculating strength of local pressure, considering the effect of flange width B and supporting width W on C and γ, the value β can be calculated by

$$\beta = 0.275(1+3.3\ W/L)(0.76+0.24\ B/b) \tag{6}$$

and $\beta \leq 0.7$.

4.3 Calculation of shear resistance

The shear resistance of the wall beam depends on H1/L, h/L, and R. By analysis of test results, shear resistance is increased with the increase of H1 and R. Fig. 14 is a graph giving the relation of Qu/bHR and H1/L (Qu is ultimate bearing reaction). In masonry construction, considering high dispersion in strength, it is suggested for practical design to adopt a line of lower limit, i.e.

$$Q = 0.265 \ bH1R \tag{7}$$

According to test results, if values h/L adopted varies from 1/15 to 1/10, the factor affecting h/L is about 12%. Introducing this parameter, then

$$Q = (0.2 + h/L) \ bH1R \tag{8}$$

Wall beams with H1/L less than 1/3 are of no practical use, because the diagonal tension failure may occur at an early stage. The use of a wall beam under concentrated loading should be avoided, because its shear resistance may reduce greatly, and brittle failure may occur without warning. It is advisable to keep the concentrated load less than 25% of the total load.

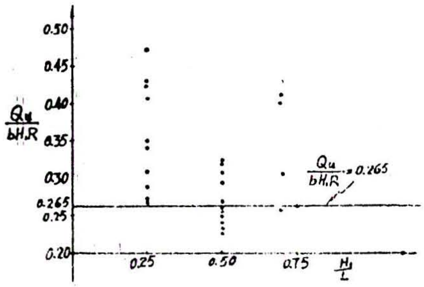


Fig. 14 $\frac{Q_u}{bH_1R} \sim \frac{H_1}{L}$

5. CONCLUSION

The following conclusions are obtained from the above analysis of test results:

1. The composition of the masonry wall and the R.C. supporting beam gives a high stiffness and provides a composite component with good mutually acting behaviour to undertake bending, shearing and compression forces caused by external loadings. Generally, the composite action of a wall beam is similar to a deep beam. The composite action of a wall beam helps to increase its load carrying capacity, and thus a good tech-economic result may be achieved. The design of a wall beam will be safe and reasonable if the composite action behaviour is taken into account.

2. The failure patterns of a wall beam under loading on top are principally the bending failure, localized compression failure at the supports and shear failure in the masonry wall. If the yielding of longitudinal steel in the supporting beam, the occurrence of local compression failure at the supports and the occurrence and development of diagonal cracks in the masonry wall can be effectively avoided, the wall beam will be able to work and carry load safely. Therefore, the calculation of strength in a wall beam should include the determination of the amount of longitudinal steel, a check of the strength of local pressure at the supports and the calculation of the shear resistance.

6. REFERENCES

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Table 1. The parameters, failure load and patterns of the tests

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Group No.	Beam No.	Beam b×h (cm)	Steel bar top low	H_1 (cm)	b (cm)	$\frac{H_1}{L}$	$\frac{h}{L}$	B (cm)	a (cm)	Masonry strength $\frac{a}{H_1}$	R (kg/cm²)	Load type	Q (T)	Failure patter	Note
A.	L_{3-3}	12×24	2φ6 / 2φ6	59	11.5	0.25	1/10				17.0	uniform load	6.0	d.t.	d.t.- diagonal tension failure
	L_{4-1}	12×24	2φ8 / 2φ12	60	11.5	0.25	1/10				23.0		13.0	d.t.	d.c.-- diagonal compression failure
	L_{5-1}	12×16	2φ8 / 2φ12	60	11.5	0.25	1/15				24.0		11.0	d.t.	l.c.- local compression failure
	L_{6-1}	12×12	2φ8 / 2φ12	60	11.5	0.25	1/20				26.0		12.5	d.t.	ben.- bending failure
	L_{7-2}	12×16	2φ8 / 2φ12	60	11.5	0.25	1/15				24.0		14.8	d.t.	Q-- load at failure
	L_{8-1}	12×16	2φ8 / 2φ12	60	11.5	0.25	1/15				21.3		14.0	d.t.	
	L_{8-2}	12×16	2φ8 / 2φ12	60	11.5	0.25	1/15				16.8		10.0	d.t.	
	L_{8-3}	no beam	2φ12	81	11.5	0.30					18.2		7.0	d.t.	
$\frac{H_1}{L}=0.25$	L_{10-1}	12×20	2φ6 / 2φ6	60	11.5	0.25	1/12				24.0		9.0	d.t.	
	L_{10-2}	12×20	2φ6 / 2φ9	60	11.5	0.25	1/12				25.0		10.8	d.t.	
	L_{11-1}	12×16	2φ6.8 / 2φ6.8	60	11.5	0.25	1/15				36.3		11.0	ben.	
	L_{13-2}	24×16	2φ8 / 2φ12	60	24	0.25	1/15				30.5		23.5	d.t.	
	L_{13-4}	24×16	2φ8 / 2φ12	60	24	0.25	1/15				34.7		29.0	d.t.	
	L_{14-1}	24×16	2φ6.8 / 2φ8	60	24	0.25	1/15				34.8		13.5	ben.	
	L_{14-2}	24×16	2φ6.8 / 2φ6.8	60	24	0.25	1/15				34.0		9.3	ben.	
	L_{14-3}	24×16	2φ8	60	24	0.25	1/15				34.8		8.2	ben.	
	L_{14-4}	24×16	no bar	60	24	0.25	1/15				38.0		6.0	ben.	
A_1	L_{3-2}	12×16	2φ6 / 2φ6	64	11.5	0.37	1/15				17.0		5.6	d.t.	
$\frac{H_1}{L}=0.30-0.37$	L_{2-1}	12×8	2φ6 / 2φ6	72	11.5	0.30	1/30				23.0		6.95	d.t.	
B	L_{4-2}	12×24	2φ8 / 2φ12	120	11.5	0.50	1/10				18.0		16.0	l.c.	
	L_{5-2}	12×16	2φ8 / 2φ12	120	11.5	0.50	1/15				17.0		15.0	d.t. &l.c.	
	L_{6-2}	12×12	2φ8	120	11.5	0.50	1/20				27.0		17.0	d.t.&l.c.	
	L_{9-1}	12×12	2φ6 / 2φ9	120	11.5	0.50	1/20				27.5		19.0	d.t.	
$\frac{H_1}{L}=0.50$	L_{9-2}	12×12	2φ6 / 2φ6	120	11.5	0.50	1/20				24.6		16.0	d.t.	
	L_{9-3}	12×12	2φ6 / 2φ9	120	11.5	0.50	1/20				25.8		22.0	d.t.	
	L_{11-2}	12×16	2φ6.8 / 2φ8	120	11.5	0.50	1/15				38.4		21.0	ben.	
	L_{13-3}	24×16	2φ8 / 2φ12	120	24	0.50	1/15				27.3		48.0	d.t.	
	M-0.5-0	24×16	2φ5.6 / 2φ9	120	24	0.50	1/15				68.5		40.0	ben.	
	M-0.5-2	24×16	2φ5.6 / 2φ9	120	24	0.50	1/15				49.4		37.5	ben.	
B_1	$L_{-0.5-24}$	12×16	2φ12 / 2φ16		11.5	0.50	1/15	24			27.38		27.0	d.t.	
$\frac{H_1}{L}=0.50$	$L_{-0.5-37}$	12×16	2φ12 / 2φ16		11.5	0.50	1/15	37			25.56		26.0	d.t.	
	$L_{-0.5-49}$	12×16	2φ12 / 2φ16		11.5	0.50	1/15	49			22.38		27.0	d.t.	
C	L_{4-3}	12×24	2φ8 / 2φ12	180	11.5	0.75	1/10				16.0		20.0	l.c.	
	L_{5-3}	12×16	2φ8 / 2φ12	180	11.5	0.75	1/15				17.0		18.0	l.c.	
	L_{6-3}	12×12	2φ8 / 2φ12	180	11.5	0.75	1/20				23.0		24.0	l.c.	
	L_{11}	12×24	2φ6 / 2φ9	180	11.5	0.75	1/10				24.6		26.5	l.c.	
	L_{13-1}	24×16	2φ8 / 2φ12	180	24	0.75	1/15				21.0		44.0	l.c.	
	C-25-0	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				29.5		30.0	l.c.	
	C-50-0	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				32.8		36.0	l.c.	
$\frac{H_1}{L}=0.75$	C-25-1	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				35.0		30.0	l.c.	
	C-25-2	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				$R_7=41.7$		32.5	l.c.	
	M-0.75-0	24×16	2φ9 / 2φ12	180	24	0.75	1/15				42.9		90.0	ben. &l.c.	
	M-0.75-1	24×16	2φ9 / 2φ12	180	24	0.75	1/15				53.2		130.0	l.c.	
	B-9.6	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				34.7		35.0	l.c.	
	B-12	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				37.7		42.5	l.c.	
	B-16	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				36.9		42.5	l.c.	
	B-22	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15				35.6		42.5	l.c.	
	C-25-3	12×16	2φ9 / 2φ12	180	11.5	0.75	1/15	24			31.2		42.7	d.c.	
C_1	W-24	12×16	2φ12 / 2φ16	166	11.5	0.70	1/15	24			23.3		45.0	d.c.&l.c.	
$\frac{H_1}{L}=0.69-0.75$	W-37	12×16	2φ12 / 2φ16	167	11.5	0.70	1/15	37			24.0		55.0	d.c.	
	W-49	12×16	2φ12 / 2φ16	168	11.5	0.69	1/15	49			30.4		60.0	d.c.	
	W-62	12×16	2φ12 / 2φ16	170	11.5	0.71	1/15	62			28.8		60.0	d.c.	
D	L_1	12×30	2φ8 / 2φ14	240	11.5	1.0	1/8				30.0		34.0	l.c.	
$\frac{H_1}{L}=1.0$	L_2	12×16	2φ8 / 2φ12	240	11.5	1.0	1/15				25.0		26.0	l.c.	
	M-1.0-0	24×16	2φ9 / 2φ12	226	24	1.0	1/15				39.2		85.0	ben.&l.c.	
S	$S_{-0.4}$	12×16	2φ9 / 2φ12	106	11.6	0.44	1/15		48	0.393	49.5	concentrated load	25.0	d.t.	
	$S_{-0.67}$	12×16	2φ9 / 2φ12	107	11.5	0.45	1/15		80	0.650	45.0		15.0	d.t.	
$\frac{H_1}{L}=0.43-0.51$	$S_{-1.0-0}$	12×16	2φ9 / 2φ12	105	11.5	0.44	1/15		120	0.992	56.8		11.8	d.t.	
	$S_{-1.0-8}$	12×16	2φ9 / 2φ12	104	11.6	0.43	1/15		120	1.000	45.6		12.5	d.t.	
	$S_{-1.25}$	12×16	2φ9 / 2φ12	122	11.5	0.51	1/15		150	1.087	21.3		6.0	d.t.	
L	$L_{-0.5-2/3A}$	12×16	2φ12 / 2φ16	120	11.5	0.5	1/15	37	80	0.558	23.78		10.0	d.t.	
$\frac{H_1}{L}=0.50$	$L_{-0.5-2/3B}$	12×16	2φ12 / 2φ16	120	11.5	0.5	1/15		80	0.558	28.26		11.8	d.t.	
	$L_{-0.5-1/2}$	12×16	2φ12 / 2φ16	120	11.5	0.5	1/15	37	60	0.441	25.80		12.0	d.t.	
I	$I_{-0.33-1A}$	12×20	2φ12 / 2φ16	80	11.5	0.33	1/12	37	80	0.800	37.35		10.0	d.t.	
	$I_{-0.33-1B}$	12×20	2φ12 / 2φ16	80	11.5	0.33	1/12	37	80	0.800	37.35		11.0	d.t.	
$\frac{H_1}{L}=0.33-1.0$	$I_{-0.5-0.67}$	12×20	2φ12 / 2φ16	120	11.5	0.50	1/12	37	80	0.571	44.5		17.5	d.t.	
	$I_{-0.67-0.13}$	12×20	2φ12 / 2φ16	150	11.5	0.625	1/12	37	80	0.471	46.52		21.5	d.t.	
	$I_{-0.75-0.44}$	12×20	2φ12 / 2φ16	180	11.5	0.75	1/12	37	80	0.400	50.78		29.0	d.t.	
	$I_{-1.0-0.33}$	12×20	2φ12 / 2φ16	240	11.5	1.00	1/12	37	80	0.308	41.32		30.0	d.t.	