The Stability of Cracked Walls subjected to Lateral Loading

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Summary

The paper compares the design of vertical spanning strip walls by flexure and as cracked sections. Partial safety factors are discussed and the design procedures are applied to 18 strip walls tested by the authors. Significant increases in the strength of cavity walls with adequate shear connection between the leaves are achieved by using the cracked wall analysis compared to flexural analysis.

1. INTRODUCTION

It is usual to design vertically spanning strip walls subjected to transverse lateral loading and a minimum of vertical loading as elements with a limiting flexural stress. The selection of the appropriate flexural stress should take account of the wide variations that occur in the flexural properties of masonry and any enhancement that might result from such vertical loading that exists. It is usual to apply large partial safety factors (γ_m) to the material properties when determining the design strengths if the limit state design procedures are used due partly to the difficulty of specifying precise flexural strengths.

The British Standard Code for Unreinforced Masonry BS 5628: Part I[1] recommends partial safety factors for materials in flexure in the range 2.5-3.5 and characteristic flexural strengths (f_{kx}) based upon a 95% confidence limit, the recommended flexural stresses in Table 3 of BS 5628 are related to water absorption of clay bricks and unit compressive strength of concrete blocks. If laboratory wallette tests are carried out in accordance with Appendix A3 of BS 5628 then the 95% confidence limit characteristic flexural strength is calculated assuming a log-normal distribution and the designer is allowed to reduce the appropriate partial safety factor by 10%. Ultimate strength moments of resistance are calculated using the simple theory of bending and to enable the full vertical load stress (g_d) at the critical section to be utilised at the design load the characteristic flexural stress may be increased by $\gamma_m g_d$. Thus the design moment of resistance of a section will be given by

$$M_d = \frac{(f_{kx} + \gamma_m g_d)Z}{\gamma_m} = \left(\frac{f_{kx}}{\gamma_m} + g_d\right)Z \quad \text{..........(1)}$$

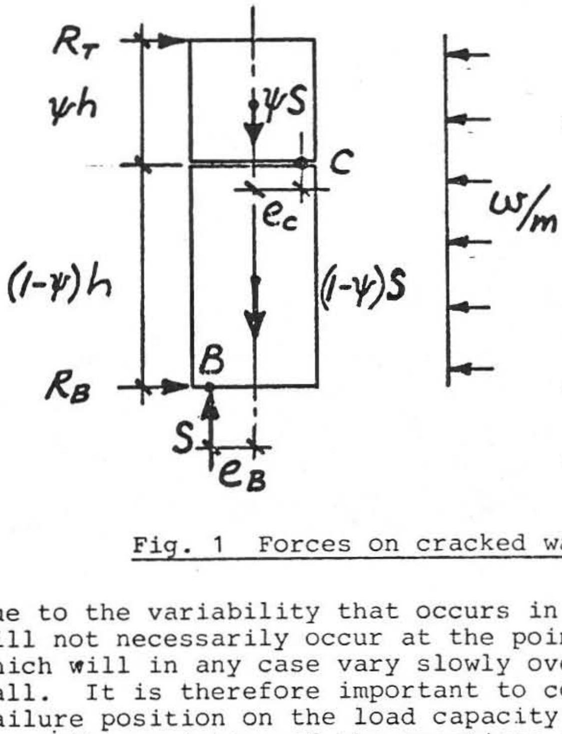
which must not be less than the design load moment which BS 5628 recommends should be taken as $\frac{W_k \gamma_f h^2}{8}$ for a vertical spanning strip wall where W_k is the characteristic wind load, γ_f the partial safety factor for load and h the vertical span of the wall. This moment may be modified by allowing for partial fixity at the base.

It is not uncommon to see vertical spanning masonry walls with a crack in a bed joint, especially where they are cavity walls with the two leaves built of materials with different responses to moisture movements, temperature changes and creep behaviour.

Walls with such cracks in the bed joints are not necessarily condemned although it could be argued that they no longer have any flexural strength. The following theory deals with the analysis of the stability of single leaf and cavity walls with cracked bed joints and compares the predicted strengths with test results and flexural design. Bradshaw and Foster[2] suggested in 1969 that design based on a cracked section was practical.

2. SINGLE LEAF STRIP WALLS

Consider the idealised wall shown in Fig. 1 which has cracked due to flexure or some other cause. Experimental measurements at the Polytechnic have shown that single leaf strip walls with a vertical span of 3m have a maximum deflection of the order 2mm at flexural failure which suggests that deflections may be ignored in the stability calculations. If it is assumed that the resultant vertical loads at the crack and the base pass through points C and B at eccentricities e_C and e_B respectively and that these points act as hinges, then by simple statics



$$\omega = \frac{2S}{(1-\psi)h^2}\left(e_c + e_B\right) \quad \ldots.(2)$$

$$R_T = \frac{S}{(1-\psi)h}\left(e_c + \psi e_B\right) \quad \ldots.(3)$$

$$R_B = \frac{S}{(1-\psi)h}\left(e_c + (2-\psi)e_B\right) \quad \ldots.(4)$$

$$\text{if } e_c = e_B = e$$

$$\omega = \frac{4Se}{(1-\psi)h^2} \quad \ldots.(5)$$

$$R_T = \frac{Se}{h}\left(\frac{1+\psi}{1-\psi}\right) \quad \ldots.(6)$$

$$R_B = \frac{Se}{h}\left(\frac{3-\psi}{1-\psi}\right) \quad \ldots.(7)$$

Fig. 1 Forces on cracked wall

Due to the variability that occurs in bed joint strengths failure will not necessarily occur at the point of maximum bending moment which will in any case vary slowly over the central region of the wall. It is therefore important to consider the effect of the failure position on the load capacity of the cracked wall. Fig. 2 shows the variation of the reactions and the load with crack position when e_B and e_C are equal. Note that when the crack is at mid-height the top and bottom reaction are not equal. An analysis of the probability of a flexural failure occuring at a particular position in the wall will not be attempted in this paper but a study of positions at which failure has occurred in tests on the 18 vertical spanning walls with a total of 32 leaves, shown in Table 1 gives an average value for $(1-\psi)$ of 0.59 with a 5% upper exclusion limit of 0.65. A value of 0.7 is used for $(1-\psi)$ in the following calculations.

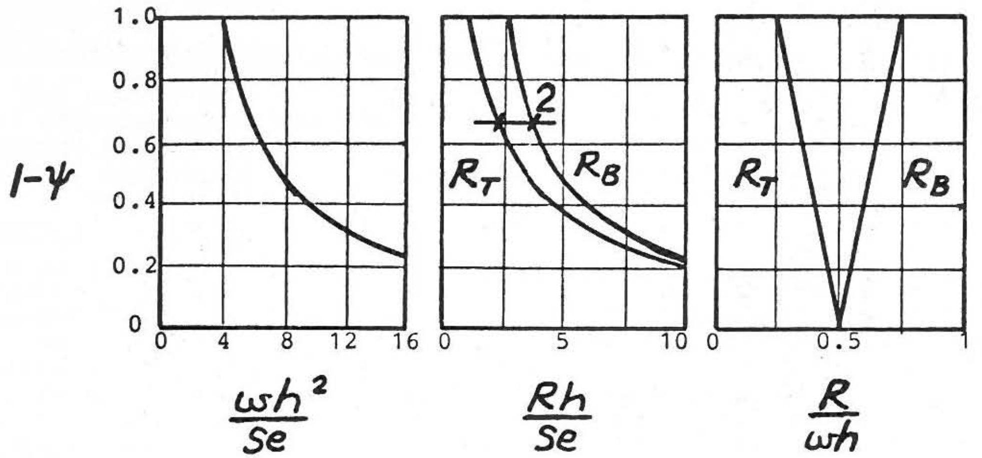


Fig. 2 Variation of strength and reactions with crack height

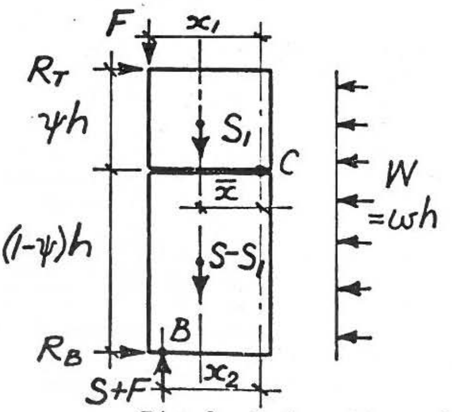
The eccentricity of the vertical load at a cracked section may be calculated in accordance with BS 5628 by the formula

$$\frac{t}{2} - \frac{n_w \gamma_m}{2f_k}$$

which for cases where the vertical load is only due to the self weight of the wall gives eccentricities of the order of 0.48t. Experimental measurements taken by the authors show that an eccentricity value of 0.45t or 0.9 x distance from the centroid to the face is a reasonable value and allows for a certain amount of irregularity in the mortar joints.

3. IRREGULAR CROSS-SECTIONS, COMPOSITE OR CAVITY WALLS

Practical strip walls usually have additional forces acting to improve their stability, the two most important being dead load from a floor or roof resting on the top of the wall and friction between a lateral support and the wall face. As this paper is concerned with walls with a minimum of vertical load, only the frictional effect will be considered (Fig.3). The formulae will be applicable for prismatic walls of an irregular cross-section or composite construction provided that any vertical joint is capable of carrying the shear necessary to hold the components together



$$w = \frac{2}{h^2}\left[\frac{F}{\psi}x_1 + \left(\frac{S+F}{1-\psi}\right)x_2\right] \quad \ldots .(8)$$

$$R_T = \frac{1}{2\psi h}\left[2Fx_1 + 2\psi S\bar{x} + w\psi^2 h^2\right] \ldots (9)$$

where $\bar{x}$ is the distance from "hinge" C to the centre of gravity of the wall

Fig.3 Prismatic wall with vertical force

If F is a friction force dependent on R_T i.e. $F = \mu R_T$ then for the case when $\psi = 0.3$

$$\omega = \frac{S}{h^2}\left\{\frac{\frac{S_1}{S}\left[\bar{x}h - \mu\bar{x}(x_1-x_2)\right] + (x_2-\bar{x})(0.3h-\mu x_1)}{0.105h - \mu(0.455x_1 + 0.045x_2)}\right\} \quad \ldots(10)$$

$$R_T = \frac{S_1\bar{x} + 0.045\omega h^2}{0.3h - \mu x_1} \quad \ldots\ldots(11)$$

A general equation allowing for additional vertical forces, the wall extending above the support and variations in the crack position is too involved for inclusion in this paper.

4. TESTS ON CRACKED WALLS

A number of strip walls were included in an extensive wall testing programme carried out at the Polytechnic of the South Bank for the Property Services Agency. The walls were tested over a vertical span of 3m and were subjected to a series of equal lateral point loads which induced bending moments which closely simulated those induced by a uniformly distributed load. The loads were applied to the brick face of all the cavity walls except 1/1R which was reversed. Initially all the walls were supported at the top by a horizontal steel beam, the face of which was oiled to eliminate bond and reduce friction. The last four walls in the series were re-tested after failure with roller bearings inserted between the wall and the beam to virtually eliminate friction and enable the effect of friction in the initial tests to be determined.

The walls were firstly tested to ultimate load in flexure with strain gauge readings being taken across bed joints and the loads which the walls carried as cracked bodies after flexural failure were recorded (See Fig.4). In the last six walls in the series deflections and rotations across the d.p.c. at the base were recorded to determine the area of contact at the base.

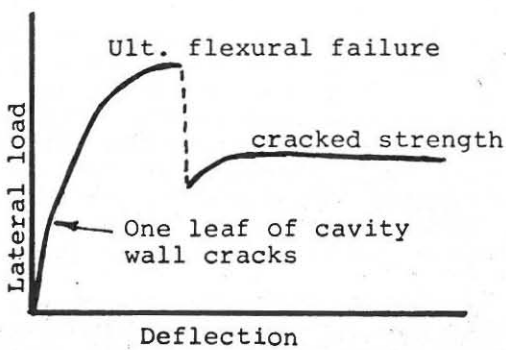


Fig. 4 Typical load-deflection curve

Brief details of the test walls are given in Table 1 and their cross-sections are shown on Fig. 5. All the walls were built on to a hessian based bituminous d.p.c. set in a mortar bed.

The cavity walls with the vertical edges closed (Figs. 5b, c and e) had a hessian based bituminous d.p.c. built into the vertical joint between the leaves. All the cavity walls except 7/2B and 7/3B had wire butterfly type wall ties at spacing not greater than recommended in BS 5628: Part I. The other two cavity walls had double triangle type ties. Both wall tie types conformed to BS 1243(3).

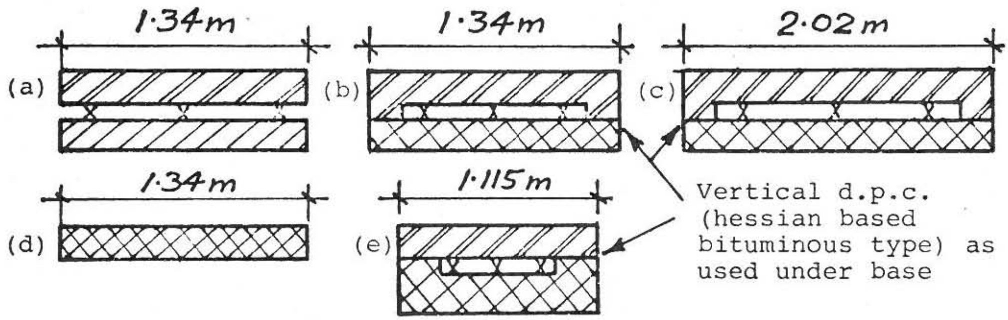


Fig. 5 Sections of vertically spanning test walls

The design strengths of the walls (i.e. ultimate strength/γ_m), have been calculated

(i) in accordance with BS 5628 for γ_m values of 3.5 and 2.5 using the flexural stresses specified in Table 3 of BS 5628 with the full self weight allowance, and

(ii) by using characteristic flexural strengths obtained from wallette tests on the respective masonry materials with the full self weight allowance and 10% reduction in the γ_m values.

The results are shown in Table 2 with the ratios of the test load to the design loads shown in parentheses.

In the cavity walls in which the vertical edges were closed, a certain amount of vertical movement between the leaves occurred after flexural failure but this was not sufficient to prevent the "tension leaf" lifting off the cracked joint and being carried by the "compression leaf". Fig. 6 demonstrates the strain distribution across a typical cavity wall. The cracked wall analyses for these walls was therefore carried out assuming composite action between the leaves. Cavity walls without the edges closed (Fig. 5a) were analysed as two separate leaves and their strengths added.

The calculation of the design strengths of the cracked walls has been based on reducing the self weight of the walls by 10% which is in effect using a self weight load factor of 0.9 as a partial safety factor for materials of $\frac{1}{0.9}$ · i.e. 1.11. This is the approach used

TABLE 1 DETAILS OF STRIP WALLS TESTED

All walls 3m vertical span, overall height 3.15m
Figures in parenthesis refer to wall with roller support at top
* cracks developed during curing

Wall Ref.	Cross-section Fig. 5	Loaded leaf		Second leaf		Ultimate loads kN/m^2		Crack height $1-\psi$	
		t mm	density kg/m^3	t mm	density kg/m^3	flexure	cracked	Block leaf	Brick leaf
1/1	a	102.5	1830	100	1300	1.47	0.65	0.45	0.53
1/1A	a	102.5	1830	100	1300	0.82	0.52	0.60	0.70
1/1R	a	100	1300	102.5	1830	1.14	0.50	0.53	0.62
1/2	b	102.5	1830	100	1300	1.69	1.27	0.45	0.43
1/3	b	102.5	1830	140	1300	3.21	1.37	0.53	0.53
1/4	c	102.5	1830	100	1300	1.49	0.99	0.53	0.53
1/5	d	102.5	1830	-	-	0.78	0.27	-	0.53
1/6	d	100	1300	-	-	0.52	0.25	0.53	-
1/7	b	102.5	1830	100	2030	2.01	1.12	0.75	0.88
1/8	b	102.5	1830	100	880	1.18	1.12	0.45	0.47
1/9	d	100	2030	-	-	0.75	0.37	0.68	-
1/10	d	100	880	-	-	0.41	0.22	0.60	-
7/1A	e	102.5	1830	215	820	3.32	2.80	0.53	0.55
7/1B	e	102.5	1830	215	820	3.62	3.00	0.68	0.65
7/2A	e	102.5	1830	190	820	3.95	3.3 (2.5)	0.56	0.56
7/2B	e	102.5	1830	190	820	3.53	2.6 (1.9)	0.68*	0.65
7/3A	e	102.5	1830	100	900	1.44	1.3 (1.1)	0.60*	0.65
7/3B	e	102.5	1830	100	900	1.60	1.3 (0.9)	0.75*	0.68

Mean value of $(1-\psi)$ 0.59

TABLE 2 DESIGN STRENGTH OF TEST WALLS

The figures in parenthesis are the ratios of ultimate flexural test load to design load for the flexural design and ultimate cracked wall test load to design load for the cracked wall design

Wall Ref.	Flexural Design kN/m^2				Cracked wall analysis kN/m^2 (equation (10))	
	BS 5628 stresses		Ch. wallette stresses		No friction at top	With friction at top
	$\gamma_m = 3.5$	$\gamma_m = 2.5$	$\gamma_m = 3.5\times0.9$	$\gamma_m = 2.5\times0.9$	$\gamma_m = 1.11$	$\gamma_m = 1.11$
1/1	0.309 (4.76)	0.408 (3.60)	0.475 (3.09)	0.637 (2.31)	0.270 (2.41)	0.293 (2.22)
1/1A	0.309 (2.65)	0.408 (2.01)	0.475 (1.73)	0.637 (1.29)	0.270 (1.93)	0.293 (1.77)
1/1R	0.309 (3.69)	0.408 (2.79)	0.475 (2.40)	0.637 (1.79)	0.270 (1.85)	0.293 (1.71)
1/2	0.272 (6.21)	0.358 (4.72)	0.422 (4.00)	0.566 (2.99)	0.742 (1.71)	0.936 (1.36)
1/3	0.401 (8.00)	0.529 (6.07)	0.727 (4.42)	0.982 (3.27)	0.995 (1.38)	1.324 (1.03)
1/4	0.260 (5.73)	0.342 (4.36)	0.405 (3.68)	0.544 (2.74)	0.732 (1.35)	0.923 (1.07)
1/5	0.173 (4.51)	0.23 (3.39)	0.243 (3.21)	0.324 (2.41)	0.161 (1.68)	0.175 (1.54)
1/6	0.136 (3.82)	0.178 (2.92)	0.232 (2.24)	0.313 (1.66)	0.109 (2.29)	0.118 (2.12)
1/7	0.288 (6.98)	0.376 (5.35)	0.313 (6.42)	0.406 (4.95)	0.913 (1.23)	1.168 (0.96)
1/8	0.263 (4.49)	0.346 (3.41)	0.285 (4.14)	0.378 (3.12)	0.645 (1.74)	0.803 (1.39)
1/9	0.152 (4.93)	0.196 (3.83)	0.123 (6.10)	0.153 (4.90)	0.170 (2.18)	0.184 (2.01)
1/10	0.127 (3.23)	0.166 (2.47)	0.095 (4.32)	0.125 (3.28)	0.074 (2.97)	0.080 (2.75)
7/1A	0.760 (4.37)	0.997 (3.33)	0.641 (5.18)	0.898 (3.70)	1.229 (2.28)	1.790 (1.56)
7/1B	0.760 (4.76)	0.997 (3.63)	0.641 (5.65)	0.898 (4.03)	1.229 (2.44)	1.790 (1.68)
7/2A	0.631 (6.26)	0.829 (4.76)	0.472 (8.37)	0.661 (5.98)	1.016 (2.46)	1.416 (2.33)
7/2B	0.631 (5.59)	0.829 (4.26)	0.472 (7.48)	0.661 (5.34)	1.016 (1.87)	1.416 (1.84)
7/3A	0.300 (4.80)	0.396 (3.64)	0.268 (5.37)	0.375 (3.84)	0.616 (1.79)	0.766 (1.70)
7/3B	0.300 (5.33)	0.396 (4.04)	0.268 (5.97)	0.375 (4.27)	0.616 (1.46)	0.766 (1.70)
Mean	(5.00)	(3.81)	(4.65)	(3.44)	(1.95)	(1.71)
C of V%	(26)	(27)	(39)	(38)	(24)	(27)

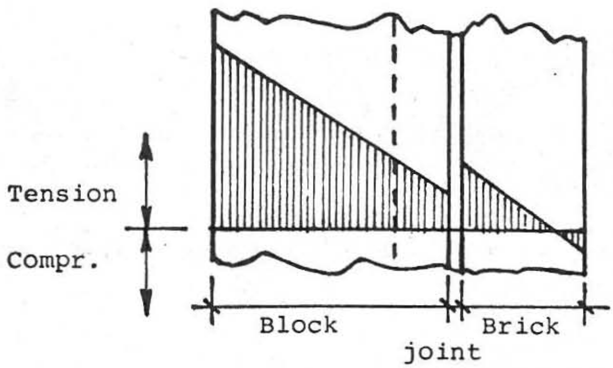


Fig. 6 Strains across crack in wall

in designing free-standing walls which cannot carry tension across their base and it is now proposed for this method of design as the weights and dimensions of the wall materials can be predicted accurately.

Analysis of the results of tests on walls 7/2A, 2B, 3A and 3B with the needle rollers inserted at the top beam gave values for the coefficient of friction between the wall and the beam as 0.79, 0.74, 0.78 and 0.97 respectively so a figure of 0.75 has been used in the cracked wall calculations which include a friction allowance.

The design strengths of the walls with and without friction at the top support are shown in Table 2 with the ratios of the design strengths to the cracked test wall strengths shown in parenthesis.

5. DISCUSSION

Table 2 shows that for the single leaf walls and the cavity walls in which the cavity was not closed at the edges the cracked wall analysis does not give design strengths as great as the flexural design methods except for the case of wall 1/9 which was built of dense concrete blocks.

The cracked wall design strengths for the closed cavity walls both with and without friction at the top beam were all greater than the flexural design strengths and the ratio of the post-cracking test load to the design strength without friction was always greater than the γ_m value adopted in the design, confirming that the method gave safe values. It will be noticed that the coefficient of variation for the ratio of test results to design strengths was least for the cracked wall analyses.

The strength of each test wall was analysed by taking account of the actual position of the failure cracks, the full weight of the masonry and the tensile forces carried by the vertical d.p.c. membrane and significantly more consistent values were obtained for the ratio of test to design loads than shown in Table 2, which are based on a conservatively fixed crack position. Table 2 does

demonstrate that the assumptions made for the design are reasonably satisfactory and safe.

The friction forces referred to on page 3 contribute to the strength and stability of the wall, but if the wall were subjected to suction it is unlikely that the frictional force could be relied upon. It is obviously important to ensure that a realistic assessment is made of all the forces acting on a wall to give the most severe situation.

The wide variation in the ratios of test loads to flexural design strengths based upon wallette tests demonstrated in Table 2 are not typical of the results of tests on two-way spanning walls which have been tested at the Polytechnic, which showed that the strength of wall panels with adequate edge fixity can be predicted with reasonable accuracy(4) by using wallette stresses and the bending moment coefficients given in BS 5628: Part I. One reason for the wide variations in the consistency of the flexural design for the strip walls is that a certain amount of composite action would have taken place in all the cavity walls, this action being least in the walls shown in Fig. 5(a). A further complication with predicting flexural behaviour of composite walls is that when the leaves are of different materials it is possible for differential moisture movements to pre-tension one leaf and pre-compress the other, the pre-tension often being sufficient to cause tensile failure before lateral load is applied.

Using the stability of the cracked wall as the limit state eliminates the uncertainties inherent in the flexural analysis but for the method to be effective for longer lengths of wall it is necessary to know the vertical shear capacity of the composite walls and to decide on a suitable partial safety factor to be applied to the shear stress. The highest shear stress in the vertical joints in the wall tests reported in this paper was 0.05 N/mm^2 for wall No. 1/4, this was not an ultimate stress. The majority of the walls tested were built with wire butterfly type wall ties which are the weakest and most flexible of the metal ties in use in the U.K. and hence the results would be valid for stiffer ties.

6. CONCLUSIONS

For vertical spanning walls with a minimum of vertical loading the lateral load strength of the walls may be assessed conservatively by

(a) assuming a crack has developed at 0.7 of the span above the base

(b) assuming hinges form at the crack and the base at an eccentricity from the centroid of 0.9 times the distance to the respective face for single leaf walls or 0.9 times the distance from the centroid of each leaf to the respective face in the case of cavity walls

(c) applying a factor to the minimum vertical loads of 0.9 which acts as a partial safety factor for the materials, and

(d) ensuring that the vertical joint in any composite or cavity wall can carry the necessary shear force.

Where cavity walls have their leaves adequately tied together their cracked section design strength will generally be greater than the flexural design strength. The flexural design method cannot be relied upon as a cracking control method due to differential shrinkage problems and therefore the cracked wall analysis is preferred.

More investigation of the vertical shear strength of composite sections is required to realise the full potential of the method.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

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