STRENGTH OF EARTH MASONRY (ADOBE) WALLS SUBJECTED TO LATERAL WIND FORCES

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SUMMARY Unfired earth masonry, mud-brick or adobe is being used for building construction at an increasing rate in Australia, and also in many other countries. This paper presents a rational method for calculating the lateral strength of earth masonry wall panels. Lateral loads on earth panels are produced by wind forces for example, and assumed uniformly distributed. Experimental verification of the theoretical method is presented using full scale laboratory and field tests. The effect of roof uplift forces transmitted to earth walls is also considered.

1. INTRODUCTION

Earth has been used as a building material since prehistoric times. With rammed earth construction dating back to Neolithic times, Steen (1). Under favourable conditions earth structures can be very durable. For example, portions of the Zigguart of Agar Ouf, which is over 2000 years old still exists in Iran, Bruno (2).

Earth as a building material was widely used during early settlement of Australia; wattle and daub, pise and adobe being the most commonly used methods of construction. Earth has continued to be used as a building material, particularly adobe, in areas such as Eltham, Victoria, Australia, and is now growing in popularity over most of Australia. The reasons for the gain in popularity of earth construction are both emotional and economic; a "mud-brick" house can be very efficient in both "capital" and "operating" energy consumption in the Australian climate.

Considerable research work has been devoted to the measurement of the strength and durability of earth bricks. For example Clifton (3) looked at methods of preservation of old adobe buildings in southern U.S.A. Mechanical properties of adobe was studied in depth by Clifton and Davis (4) and Clifton, Brown and Robbins (5). Testing methods for adobe bricks and the difficulty of interpreting strength testing data was explored by the author, Yttrup (6). However, no recent work has been published where the structural adequacy of wall panels has been considered. The only published data on lateral strength of full earth masonry wall panels appears to be that by Whittemore (7) in 1941, and by the author, Yttrup (8) in 1981.

Building Authorities need to be assured of the structural adequacy of earth buildings when issuing a building permit. Traditionally compressive strength tests have been required for earth bricks. However, little guidance exists to gauge the adequacy of wall panels against lateral wind loads. Consequently many modern earth buildings have a very low margin of safety with respect to lateral wind loads on wall panels; even though the earth bricks are of adequate strength. The guidance given by Bulletin No. 5, Commonwealth Experimental Building Station, Middleton (9) which is based on the author's work (8) is useful but incomplete. Likewise many Building Codes from the South-West of the U.S.A. do not consider wall stability correctly.

A method of calculating the resistance of vertically spanning earth masonry panels to lateral wind pressure is derived and compared with experimental results in the following sections. The effect of roof weight and uplift transmitted to the wall panels is also considered.

2. NOTATION

a	Roof hold down bolt, anchoring depth (m).
C	Compressive force at the contact between failure blocks in the earth wall (kN).
C_b	As for C, but at the base of the wall (kN).
H	Wall height measured from wall footing to roof plane wall support (m).
P	Wall surcharge load, assumed to act centrally at the top of the wall (kN m^{-1}).
S	Height of wall equivalent to surcharge load P (m).
t	Wall thickness (m).
T	Uplift force at top of wall (kN m^{-1})
w_f	Ultimate lateral strength of a wall panel (kN m^{-2}).
W^*	Virtual work for whole structure.
X	Distance from top of wall to upper tension crack (m).
α	Distance to resultant compressive force at a wall tension crack, measured in from compressive face (m).
α_b	As for α but at the face of the wall.
β	S/H surcharge height 'S' as a fraction of wall height 'H'.
γ	Unit weight of earth wall material (kN m^{-3}).
θ^*, ϕ^*	Virtual work displacements.

3. THE WALL MODEL

Earth masonry walls in real buildings resist lateral wind forces by a combination of vertical and horizontal flexural action in the wall and interaction with window and door frames and cross-walls. For modern earth buildings in Australia full height window and door openings are common, therefore vertical flexural action of the wall becomes important. For this reason the wall "model" chosen for analysis is a vertical "propped cantilever" as shown in Fig 1. This model allows fairly simple analysis by both elastic theory and an ultimate strength formultion.

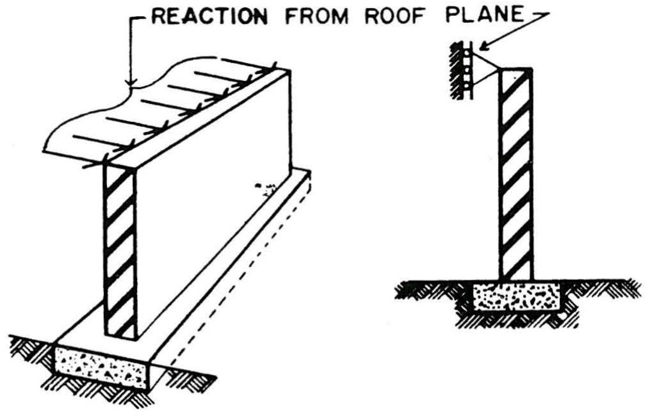


Fig 1 The Wall Model

3.1 Elastic Analysis

The wall can be analysed elastically using standard propped cantilever solutions for bending combined with gravity axial stresses. Structural Engineers usually assume no tensile strength in the bed joints of the wall and therefore no allowable tensile stress and calculate a 'safe lateral load' for the wall panel. Shortcomings with this method are (a) an unknown post-cracking strength and therefore unknown margin of safety, and (b) neglect of the stiffness of wall reactions which markedly affects elastic solutions.

If the roof construction in a building leads to a flexible reaction at the top of the wall the elastic analysis is sensitive to the degree of flexibility. For a very flexible, low stiffness reaction, the wall will tend to act as a vertical cantilever. Some elastic solutions for safe lateral load are presented in Fig 2. The solutions consider the top wall reaction as completely stiff and completely flexible; a real building would lie between these limits.

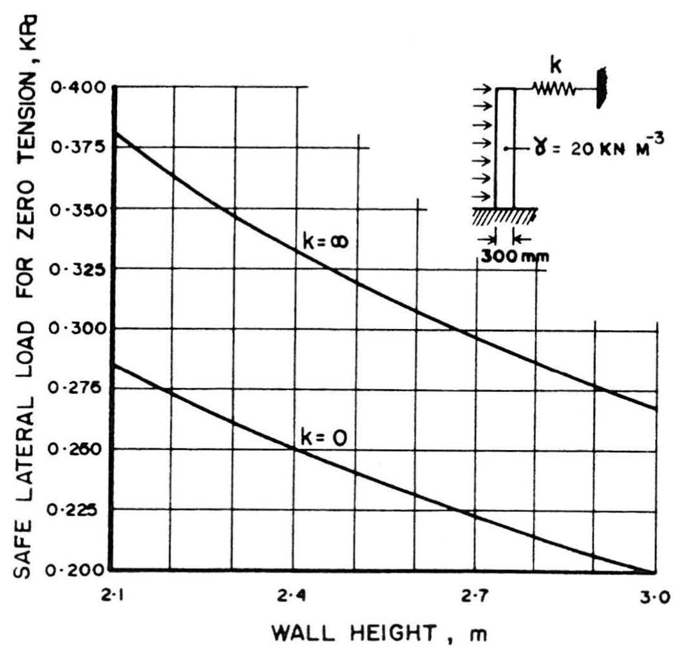


Fig 2 Lateral Strength From Elastic Theory

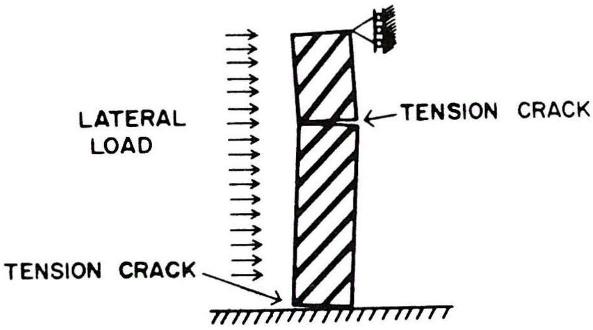


Fig 3 Wall Failure Mechanism

3.2 Ultimate Strength Analysis

Earth walls subjected to lateral load fail by forming a 'two-block' mechanism as shown in Fig 3. This was also observed by Schultze (10) for dry stacked concrete blocks. At failure the forces acting for the two block mechanism are shown in Fig 4. The blocks are not rigid therefore the compressive force at the block contacts is not located at the compressive face but at a small distance α from it.

At limiting equilibrium the ultimate lateral load w_f can be related to the wall geometry, unit weight, distance α and surcharge load P. By applying the theorem of virtual work to the mechanism in Fig 4, the following equation results for a unit width of wall:

$$W^* = \gamma t^2 H\theta^* + Pt\left[\frac{(H-X)}{2X} + 1\right]\theta^* - C_b\alpha_b\theta^* - c(t-\alpha)\theta^*$$

$$+ C\left[t + \frac{(H-X)}{X}\right]\theta^* - w_f\frac{H}{2}(H-X)\theta^* = 0 \qquad \text{......(1)}$$

The surcharge load P can be replaced by an equivalent height of wall surcharge S, as shown in Fig 4:

$$S = \frac{P}{\gamma t} \qquad \text{......(2)}$$

From vertical equilibrium:

$$C_b = \gamma t(S+H) \qquad \text{......(3)}$$

$$C = \gamma t(S+X) \qquad \text{......(4)}$$

Preliminary work by Van Kempen (11) suggests that α is directly related to the compressive force C, then:

$$\alpha = \alpha_b\left(\frac{S+X}{H+S}\right) \qquad \text{......(5)}$$

Substituting equations (2), (3), (4) and (5) into equation (1) gives:

$$w_f = \frac{2\gamma t^2}{H(H-X)} \Bigg[H + S[\frac{(H-X)}{2X} + 1] - \frac{\alpha_b}{t} (S+H)$$

$$- \frac{(S+X)}{t} (t - \alpha_b \frac{(S+X)}{(H+S)} + \frac{(S+X)}{t}(t + \alpha_b (\frac{S+X}{H+S})(\frac{H-X}{X})) \Bigg] \qquad ......(6)$$

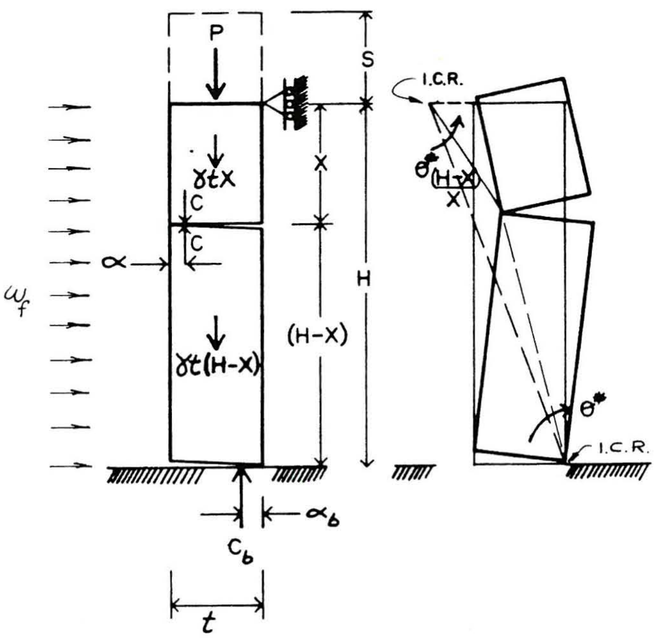


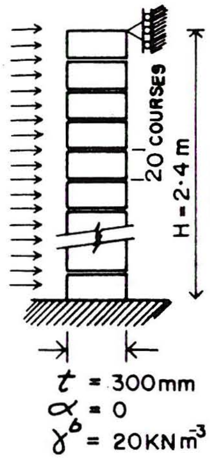
Fig 4 Forces Acting At Wall Failure

Equation 6 can be solved at all bed joints in the wall panel. The joint with the lowest value of w_f defines the failure mechanism.

Solutions of equation (6) are shown in Fig 5 for different values of surcharge load P. These solutions agree with analytical solutions derived by Baker, Padhye and Schultze (12), for the special case of $\alpha = \alpha_b = 0$. If P = 0 the critical joint will always be the first joint below the top reaction and bond strength is zero.

Typical solutions are shown in Fig 6 for a unit weight $\gamma = 20$ kN m^{-3}. The value of α_b does not effect the solution significantly. However, the effect of joint erosion or raked joints can be considered by using appropriate values of α_b.

Unlike elastic theory predictions, the ultimate lateral strength of the wall is practically independent of the stiffness of the upper wall reaction restraint.



JOINT Nº	w_f KPa P = 0	P = 1 KN m⁻¹	P = 2 KN m⁻¹
1	1.58 *	2.73	3.88
2	1.67	2.30	2.94
3	1.76	2.23 *	2.70
4	1.88	2.27	2.66
5	2.00	2.35	2.69 *
6	2.14	2.47	2.79

* CRITICAL JOINTS

Fig 5 Ultimate Wall Strength at Different Bed Joints

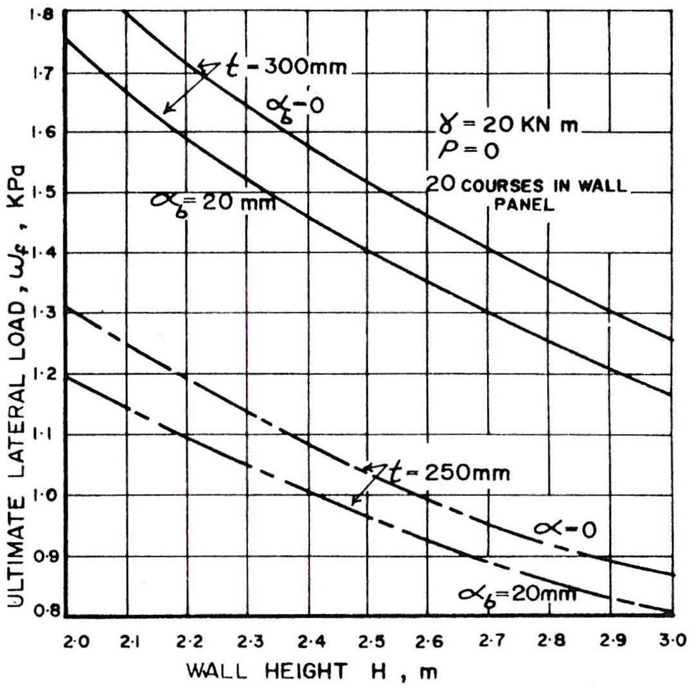


Fig 6 Ultimate Lateral Strength of Wall Panels

It can be demontrated that the difference between the rigid brick solutions for wall strength where α_b = 0 and the non-rigid brick solutions where $\alpha_b \neq 0$ differ by a constant; 0.92 in the case of Fig 6. This being the case the simpler rigid block analysis can be used and simply adjusted by multiplying by the appropriate correction factor; 0.92 in the above case. Rigid brick solutions for different surcharge conditions are presented in non-dimensional form in Fig 7.

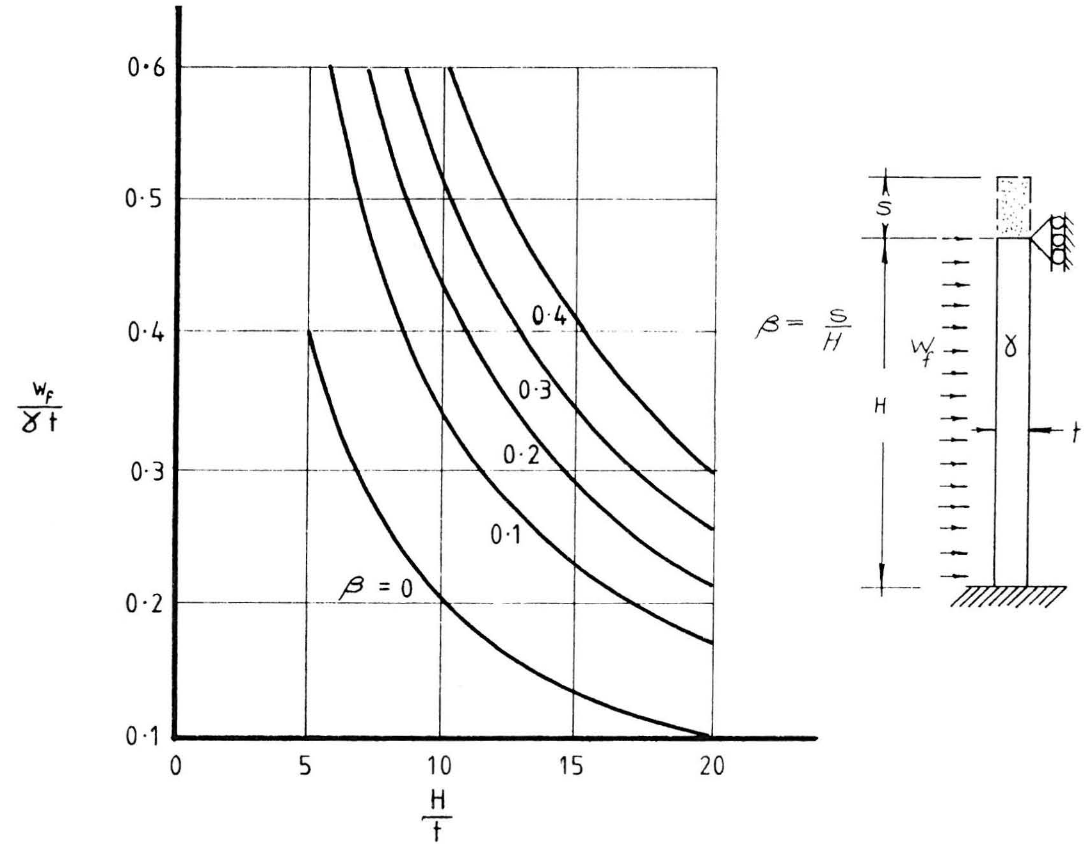


Fig 7 Rigid Brick, α_b=0, Solutions For Lateral Wall Strength

4. EXPERIMENTAL VERIFICATION

Numerous wall panels have been tested by Whittemore et al (7). The adobe walls tested are shown in Fig 8. The average observed lateral strength for three walls was 2.92 kPa. Using equation (6) the wall capacity is predicted as 2.99 kPa, ie. within 6% of the observed value.

A wall panel tested by the author shown in Fig 9 failed at 1.81 kPa. The predicted capacity of the wall using equation (6) is w_f = 1.58 kPa which is 15% under the observed strength. If allowance is made for a nominal bond strength of 10 kPa, then the predicted capacity of the wall is 1.80 kPa and the predicted failure geometry matches the observed.

An insitu wall panel in an old adobe building in Western Victoria was tested by the author and had a measured lateral strength of 0.68 kPa and predicted strength using equation (6) of 0.65 kPa; excellent agreement. Bond wrench tests at this building confirmed the bond strength was zero.

Earth wall panels supported on four sides have been tested by Baker (13). The panels were 6 m wide and 3.1 m and 4.35 m high, and 250 mm thick. The observed strength of these panels was 1.50 kPa and 1.0 kPa respectively. The predicted capacity using equation (6) which allows only for vertical flexure is 0.8 kPa and 0.6 kPa respectively. The predicted values are conservative due to the two-way action and small bond strength in these wall panels.

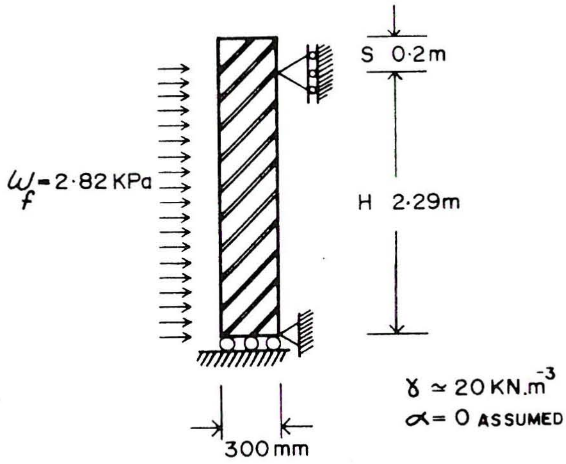


Fig 8 Walls Tested By Whittemore et al

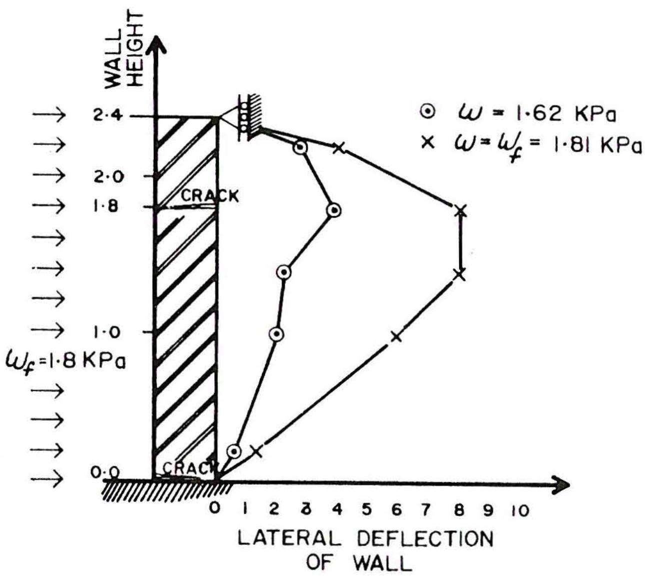


Fig 9 Wall Tested by the Author

5. EFFECT OF ROOF UPLIFT FORCES ON LATERAL STRENGTH

For lightweight roofs wind loading can produce large net uplift forces. It is common practice to place hold-down bolts into the supporting walls to such a depth that the weight of wall above the bolts can just resist the design uplift forces. In earth masonry, and also normal masonry, the interaction of simultaneous wind uplift forces and lateral wind forces is often ignored or not understood. The situation is shown diagramatically in Fig 10.

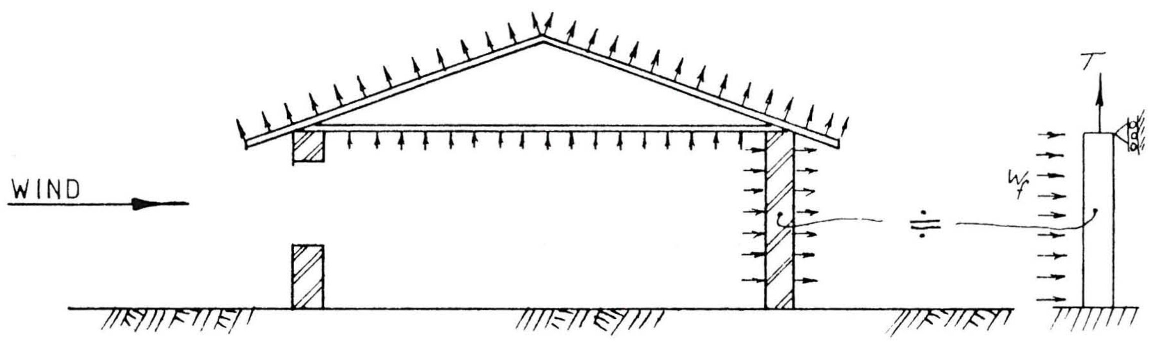


Fig 10 Simultaneous Wind Uplift and Lateral Forces

If the hold down bolt is such that weight of wall above the bolt just equals the uplift, then the capacity of the wall (for zero bond strength masonry) to resist lateral wind forces is zero.

To explore the effect of uplift forces on wall stability a simple "wall model" as shown in Fig 11 was adopted. Because of the anchor bolt the position of tension cracks forming the failure mechanism was assumed to occur at the anchor plate level. Using rigid brick analysis and zero bond strength, the depth of anchor "a" required so that the lateral strength of the wall with uplift equals the strength of the wall without any uplift has been calculated; the results in dimensionless form are presented as Fig 11. It can be seen that anchor bolt embedment must be significantly greater than that required to simply hold the roof down if wall stability is not to be reduced.

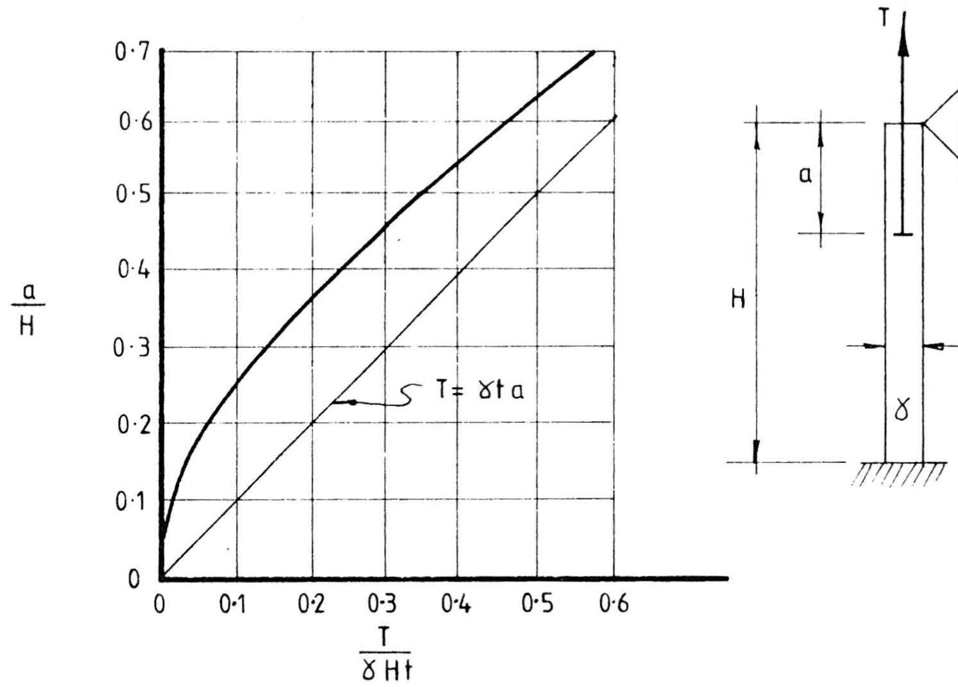


Fig 11 Anchor Depth Required for Uplift Forces

6. WALL DESIGN

From the failure mechanism observed, it is apparent that earth walls resist lateral loads principally due to their self weight. They are gravity or mass structures. This contrasts with other forms of masonry where material strength, specifically bond strength, dictates the design of the wall panels.

For a zero bond strength wall the ultimate load condition specified in the Australian Masonry Code (14) reduces to;

1.5 (WIND LOAD) with 0.95 (AVERAGE WALL DENSITY)

If these partial load factors are used with the rigid block solutions presented in Fig 7, then a computed wall strength should be multiplied by 0.9 to 0.95 to allow for actual earth walls not being rigid. For comprehensive strength of earth bricks about 1000 kPa use the 0.9 factor, and 0.95 for brick strengths above 2000 kPa.

If roof uplift forces are to be resisted by the earth walls the depth of anchor bolts required can be computed using Fig 11.

7. CONCLUSIONS

Earth masonry walls can be analysed and designed using simple structural mechanics to predict their strength under lateral wind forces. The simple vertical flexure analysis method proposed leads to accurate predictions for one-way vertically spanning walls, and a conservative estimate for walls spanning two-way.

Commonly used earth masonry walls in Australia are 250 mm and 300 mm thick, these are suitable for normal building construction provided adequate lateral support is supplied to the top of the wall panel. For sites with high winds and/or severe wind exposure, lateral stability of earth masonry wall panels need careful consideration.

For earth masonry wall panels resisting lateral wind forces simultaneous with roof uplift wind forces, special attention must be given to selecting hold down bolt lengths. It is not adequate to simply install roof hold down bolts to a depth such that weight of wall above bolts just equals uplift forces, as is often the case.

8. REFERENCES

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