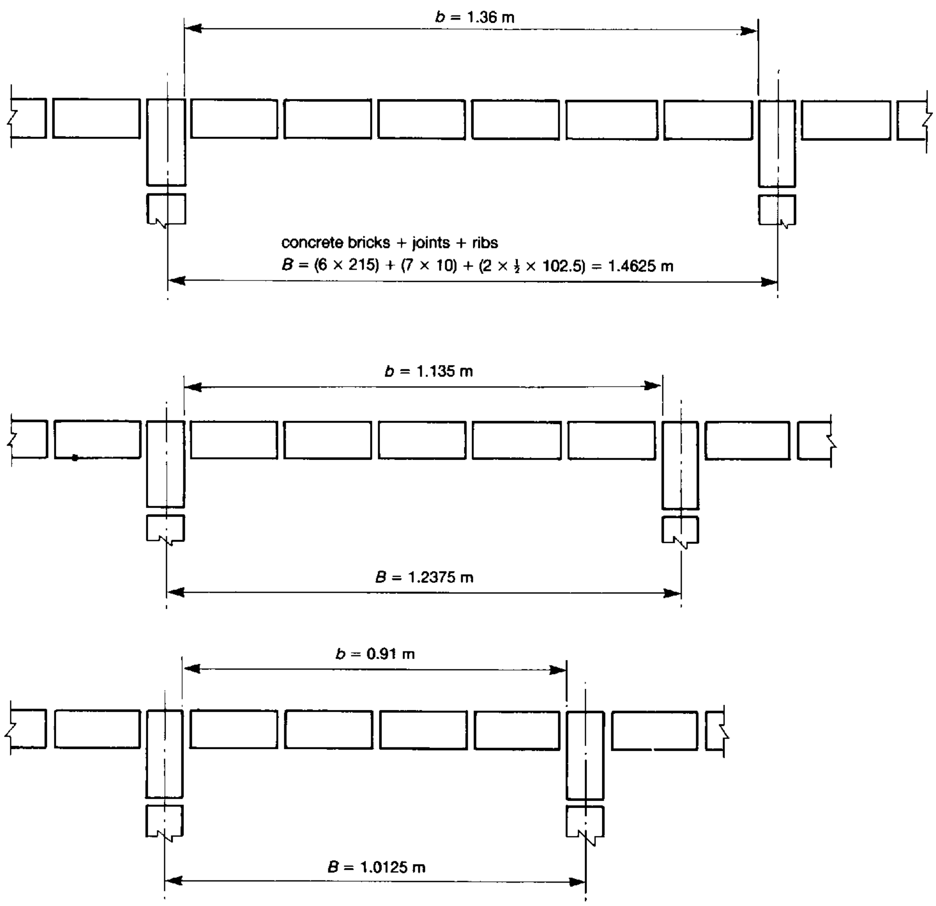
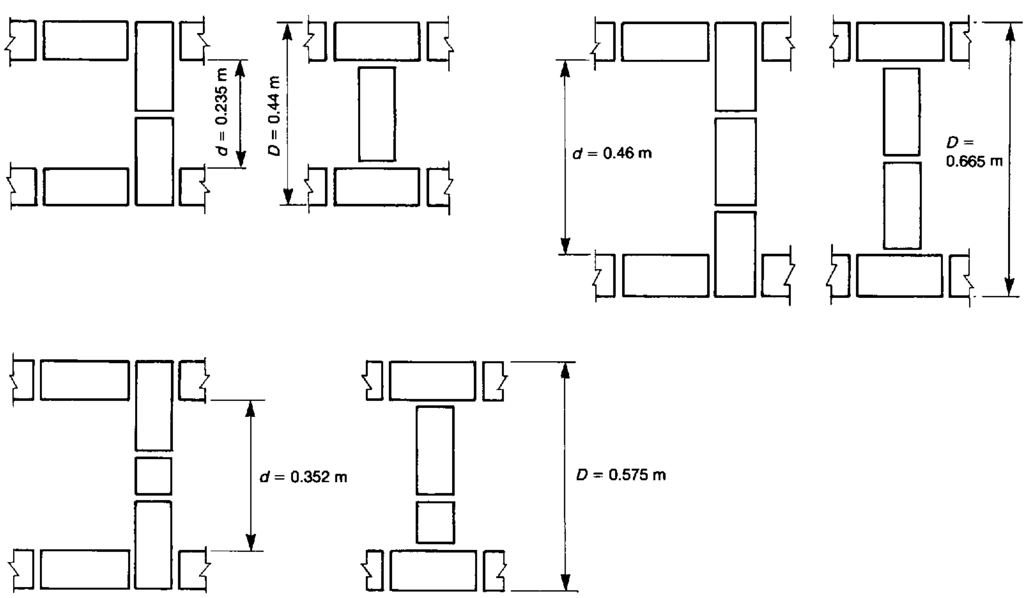


Rib spacing



Common depths

Figure 21: Typical bonded concrete brick diaphragm wall arrangements used for Table 2 data

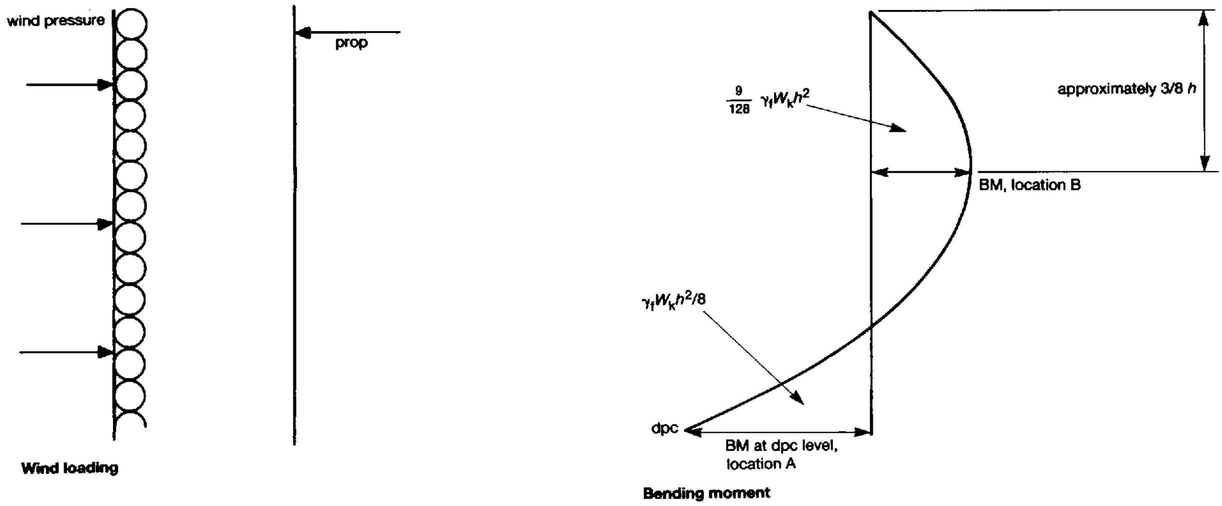


Figure 22: Critical levels for bending moment, *A* and *B*

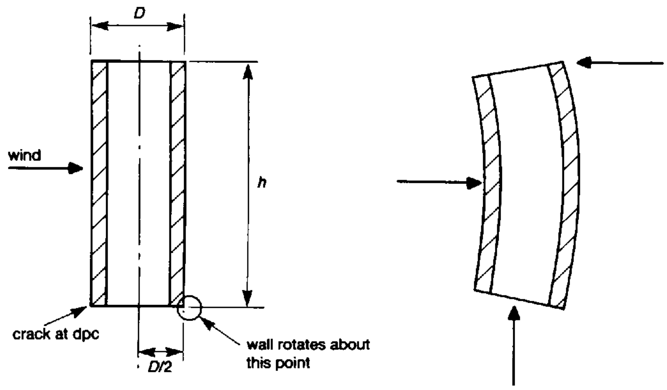


Figure 23: Discontinuity at dpc level

2.4.2 Design bending moments

In order to design the required masonry strengths, it is first necessary to determine the maximum forces, moments and stresses within the wall. If the applied wind moment at the base of the wall should, by coincidence, be exactly equal to the stability moment of resistance (MR_s), maximum forces, moments and stresses will be found either at the base of the wall or at a level 3/8 h down from the top of the wall. If the MR_s is less than the applied base wind moment of $\gamma_f W_k h^2/8$, or significant lateral deflection of the roof prop occurs, the wall will tend to rotate and 'crack' at the base. As no tensile resistance is assumed at this level the design MR_s does not decrease and any reduction in the lever arm of the vertical load due to rotation at the hinge is negligible and can be ignored. However, on the bending moment diagram, the level of the maximum wall moment will not now be at 3/8 h down from the top and its value will exceed $9\gamma_f W_k h^2/128$. The wall will become similar to a partially fixed ended beam. For example, suppose the value of a particular MR_s is equivalent to, say, $\gamma_f W_k h^2/10$, then the reactions at base and prop levels are:

$0.6\,\gamma_f W_k h$ at base level

$0.4\,\gamma_f W_k h$ at prop level (see Figure 26).

The stability moment of resistance chosen above is inadequate to resist the true propped cantilever base moment of $\gamma_f W_k h^2/8$. The section is 'cracked' and any reserve of strength available at the higher level is called upon. This modifies the bending moment diagram from that of a true propped cantilever. The modification for the example under consideration is shown in Figure 27.

The applied wind moment at the level of 0.4 h down is calculated as:

$$(0.4\,\gamma_f W_k h \times 0.4\,h) - (0.4\,\gamma_f W_k h \times 0.2\,h) = 0.08\,\gamma_f W_k h^2$$

This exceeds the true propped cantilever wall moment of $0.07\,\gamma_f W_k h^2$. The moment of resistance provided by the wall at this level must then be checked against the calculated maximum design bending moment.

In summary, the action of the wall may be described as that of a member simply supported at prop level and partially fixed at base level, where the partial fixity can be as high as that of a true propped cantilever.

The initial assumption of a perfectly rigid prop generally provides the most onerous design condition. Considering the two locations of maximum design moment, it is apparent that the *critical* design condition invariably occurs at the higher location. The resistance at this location is dependent on the development of both flexural compressive and flexural tensile stresses.

2.4.3 Allowable flexural stresses

(1) *Allowable flexural tensile stress,* f_{ubt}.
BS 5628 : Part 1, Clause 36.4.3 gives:

design moment of resistance $= f_{kx} Z/\gamma_m$

where f_{kx} = characteristic flexural strength (BS 5628 : Part 1, Clause 24)

and γ_m = partial safety factor for materials (BS 5628 : Part 1, Clause 27)

For the purpose of this design guide, f_{kx}/γ_m is termed *allowable flexural tensile stress,* f_{ubt}.

(2) *Allowable flexural compressive stress,* f_{ubc}.
BS 5628 : Part 1 gives no consideration to flexural compressive stresses in designing laterally loaded elements although in the derivation of β (in Appendix B) the code discusses the application of a rectangular stress block of $1.1\,f_k/\gamma_m$ to the resistance of bending moments produced by eccentric vertical loading. Consideration must also be given to the implication of the geometric form of the diaphragm