5.3.8 Vertical load resistance of solid walls and columns

It should be noted that there are no separate provisions for columns in EC6 Part 1-1[1]. They are simply considered as short walls.

The design resistance of a single leaf wall per unit length is N_{Rd} given by:

$$N_{Rd} = \Phi t f_d$$

Where: Φ is a capacity reduction factor allowing for the effects of slenderness and eccentricity of loading

t is the thickness of the wall

f_d is the design compressive strength of the masonry.

When the cross-sectional area of the wall is less than 0.1m^2, f_d should be multiplied by:

$$(0.7 + 3A)$$

Where: A is the loadbearing horizontal gross cross-sectional area of the wall in square metres.

For a faced wall, the wall may be designed as a single leaf wall constructed entirely of the weaker unit with a longitudinal joint.

For a double-leaf wall, if the leaves are tied together adequately, the wall may be designed as a single leaf wall (assuming that both leaves are similarly loaded), or alternatively as a cavity wall.

Chases and recesses should be allowed for, see Section 6.4.

The slenderness reduction factor Φ is applied at the top or bottom of the wall Φ_i and at mid height of wall, Φ_m.

$$\Phi_i = 1 - 2\frac{e_i}{t}$$

Where: e_i is the eccentricity at the top or bottom of the wall, see Section 5.3.6.2

t is the thickness of the wall.

$$\Phi_m = A_1 e^{-\frac{u^2}{2}}$$

Where: $A_1 = 1 - 2\frac{e_{mk}}{t}$

$$u = \frac{\lambda - 0.063}{0.73 - 1.17\frac{e_{mk}}{t}}$$

$$\lambda = \frac{h_{ef}}{t_{ef}}\sqrt{\frac{f_k}{E}}$$

As discussed in Section 5.3.6.3, e_{mk} is taken as equal to e_m where e_m is the eccentricity in the mid height of the wall.

Walls spanning horizontally are treated in a similar manner; in the formulae and limitations outlined earlier in this subsection substitute l for h (where l is the length of the panel), and use f_{xk2} (bending perpendicular to the bed joints); in practice, σ_d would be taken as zero. There is no base stability moment to take advantage of, although if the wall is continuous across its vertical supports it may be possible to design it as a continuous beam. It is important to check that the location of vertical movement joints (particularly in concrete blockwork walls) has been considered in the analysis of continuous beam bending moments.

5.4.7 Cavity walls

Provided that the wall ties used are capable of transmitting the forces to which they are subjected, the design lateral strength of a cavity wall may be taken as the sum of the design strengths of the two individual leaves, allowing for the additional strength of any piers bonded to one or both of the leaves.

5.4.8 Geometric walls

EC6 Part 1-1[1] does not specifically deal with the design of geometric walls subject to lateral loading, such as diaphragm and fin walls (see Figures 5.25 and 5.26). However, UK practice[60] for the design procedure for these walls is essentially the same at that given in Section 5.4.5 for vertically spanning walls.

The geometric profile of the wall provides considerable enhancement to its resistance to lateral load at base level, with an increased lever arm for the gravitational mass, and within the wall height, with an increased section modulus to minimise tensile stresses. Note that this *Manual* assumes that the building does not rely on the flexural strength of these walls for its overall stability.

In assessing the section modulus of a geometric wall, the outstanding length of flange from the face of the fin or cross-rib should be taken as one tenth of the clear height of the wall, for walls spanning vertically between restraints. For cantilever walls the length should be taken as one fifth of the clear height. In neither case should the length be taken as more than half the clear distance between fins or cross-ribs.

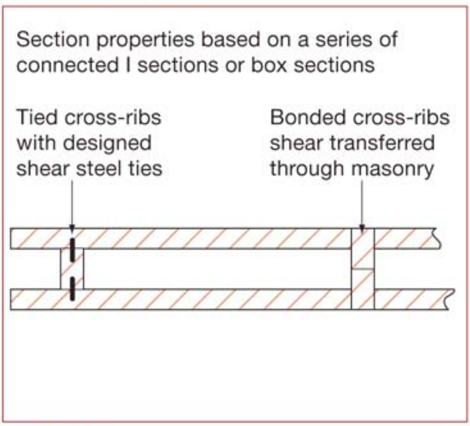

Fig 5.25 Typical diaphragm wall section

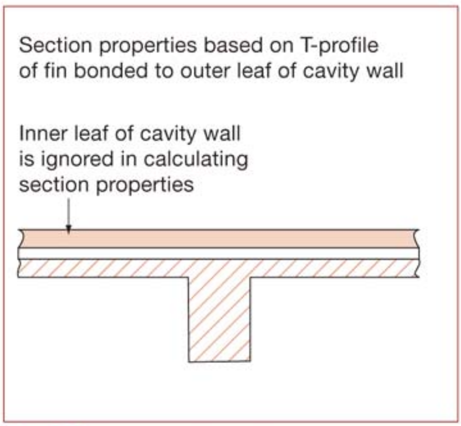

Fig 5.26 Typical fin wall section

A suggested design procedure applicable to both diaphragm and fin walls is given below.

i) Calculate loadings (dead, imposed and wind).

ii) **Select trial section of wall profile and masonry strength; guidance on trial section selection procedures is given in reference 60.**

iii) Calculate applied bending moment at base of wall and compare with stability moment of resistance M_{Rds}.

iv) Calculate position and magnitude of maximum applied bending moment within height of wall and compare with flexural resistance of wall at this level.

v) Check that the wall is stable when its strength is derived from gravity forces alone (i.e. ignoring the flexural tensile strength of the masonry) with partial factors of 1.0 on the permanent and wind actions.

vi) Calculate shear stresses at junctions of cross-ribs (in diaphragm walls) and fin/flange (in fin walls).

vii) Design shear ties or calculate shear resistance of the bonded masonry at these shear interfaces.

Note that because of the nature of differential movements between clay units and concrete units the mixing of these units within geometric wall profiles should be considered with caution.

Note also that the deflection of the structure providing horizontal support to the top of the wall should be taken into account in calculating the bending moments in the wall.

The design procedure is one of trial and error.

5.4.9 Arching

The development of lateral load resistance through the in-plane arching action of a wall panel may be considered. Shrinkage effects in concrete blockwork and potentially inadequate frame support details, etc. can make this approach unreliable, and it is not, in general, recommended.

However, where a masonry wall is built solidly between supports (vertical or horizontal) which are capable of resisting an arch thrust without significant deflection, movement or distortion, the wall may be designed to resist lateral loading applied to it by assuming that it behaves as a three-pin arch. Figure 5.27 illustrates the structural model to be adopted.

Thus, the arch rise, r, is given by:

$$r = 0.9t - d_a$$

Where: t is the thickness of the wall, taking into account the reduction in thickness resulting from recessed joints

d_a is the deflection of the arch under the design lateral load; it may be taken to be zero for walls having a length to thickness ratio of 25 or less.