35, which will assist in counterbalancing the nett uplift reaction from the roof beams.

Dead load counterbalance from reinforced capping beam = area x length x density x γ_f
= 0.60 x 0.55 x 4.8 x 24 x 0.9 = 34.20 kN
(OK: 31.20 kN uplift).

Hence the capping beam is adequate to counterbalance the roof beam uplift force. Alternatively, a shallower capping beam could be used and anchorage made into the masonry to provide the required counterbalance mass.

If a capping beam is not to be used, the wind uplift force may be resisted by means of metal straps built into the masonry using the dead load of the masonry as the uplift counterbalance. The designer must allow for the effect of the reduced dead load, resulting from this uplift force, in the stress calculations for bending across the wall section and in the calculation of the stability moment of resistance at the base of the wall.

Stage 4. Specification of blocks and mortar

The blocks to be used throughout have been specified as having a compressive strength of 7.0 N/mm^2 and a density of 2000 kg/m^3 and are to be set in a designation (iii) mortar (1 : 1 : 6). The work size of the blocks to be used is 440 x 215 x 100 mm.

Stage 5. Check external leaf spanning between cross-ribs

Condition (a), Section 2.2.1, for checking the spacing of the cross-ribs relates to the leaf acting as a continuous slab spanning between the cross-ribs to support the lateral loading; in this case, wind.

$M = \gamma_f W_k B_d^2/10$
$= 1.4 \times 0.65 \times 1.18^2/10$
$= 0.127$ kNm

Design moment of resistance, $MR = f_{kx}Z/\gamma_m$

For the masonry specified,

f_{kx} = 0.6 N/mm^2 (7.0 N/mm^2 blocks in designation (iii) mortar with plane of failure perpendicular to bed joints)

γ_m = 3.5 (from Table 4 of BS 5628 : Part 1, for normal control of both construction and manufacture of structural units)

Z = $1 \times t_f^2/6 = 1 \times 1.0^2/6 = 0.00167$ m^3

hence $MR = (0.6/3) \times 0.00167 \times 10^3$
= 0.286 kNm which is greater than the applied bending moment of 0.127 kNm

Stage 6. Design wind moment and MR_s at base of wall

Consider a 1 m length of wall:

Design wind moment at base $= \gamma_f W_k h^2/8$
$= 1.4 \times 0.65 \times 7.5^2/8$
$= 6.40$ kNm

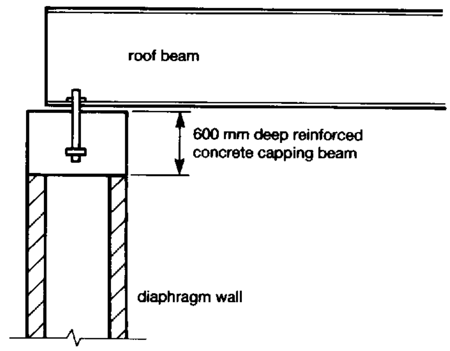

Figure 35: Roof anchorage to reinforced concrete capping beam

Stability moment of resistance = axial load x lever arm

The axial load for this design example comprises only the design dead load of the masonry (as the wind uplift cancels out the dead loading from the roof and the capping beam) and is calculated as:

γ_f x area x density x height = 0.9 x 0.23 x 20 x 7.5
= 31.05 kN

The lever arm of the axial load (see Figure 24) is calculated by first establishing the minimum width of the stress block at the point of rotation.

Minimum stress block width w_s = axial load/P_{ubc}

where P_{ubc} = allowable flexural compressive stress

Concrete blocks with a compressive strength of 7.0 N/mm^2 set in a designation (iii) mortar have been specified. Therefore, from Table 2d of BS 5628 : Part 1, $f_k = 6.4$ N/mm^2. The foundation is assumed to comprise a reinforced raft, and a section through the edge beam is shown in Figure 36. The raft foundation affords full restraint to the wall at this level and hence β may be taken to be 1.0.

Hence, $P_{ubc} = 1.1 \times 6.4/3.5 = 2.01$ N/mm^2

Now, assuming the stress block to be within the leaf thickness, the minimum width of stress block, w_s, is given by

$w_s = 31.05 \times 10^3/(1000 \times 2.01)$
$= 15.45$ mm
(i.e. assumption correct – within leaf thickness)

Hence lever arm = wall thickness/2 – w_s/2
= 550/2 – 15.45/2 = 267 mm
and stability moment of resistance, MR_s
= 31.05 x 0.267
= 8.29 kNm (see Figure 37)

This is greater than the applied design wind moment, at the base of the wall, calculated earlier as 6.40 kNm.

Stage 7. Design flexural stresses

Since the stability moment of resistance at the base of the wall exceeds the applied design wind moment, the wall is assumed to act as a true 'propped cantilever' and the maximum applied design wind moment in the height is, therefore, located at 3/8 h down from the roof prop.

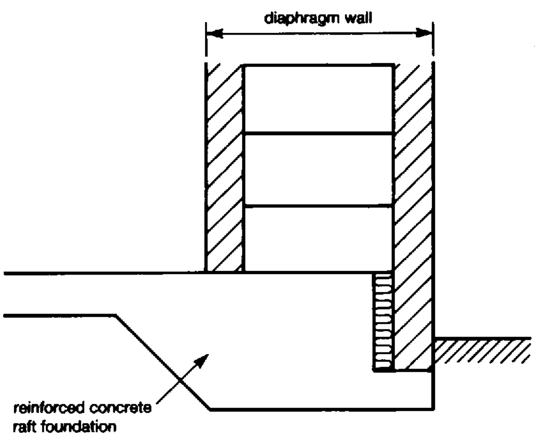

Figure 36: Raft foundation and edge beam

Design wind moment at 3/8 *h*,

$M_w = \gamma_f W_k h^2/128$
$= 9 \times 1.4 \times 0.65 \times 7.5^2/128$
$= 3.6$ kNm

The design bending moment diagram can now be drawn as shown in Figure 38.

For interest compare the calculated value of the stability moment of resistance, MR_s, with the value derived from the approximate lever arm method (suggested earlier for the calculation of the trial section).

Approximate $MR_s = \gamma_f$ x density x area x height x approximate lever arm
$= 0.9 \times 20 \times 0.23 \times 7.5 \times (0.475 \times 0.55)$
$= 8.11$ kNm

which is also greater than the design bending moment at base level and is close to the value of design MR_s calculated as 8.29 kNm.

Analysis of the stresses at the level of maximum wall moment assumes triangular stress distribution, using elastic analysis, but relates to the ultimate strength requirements at the extreme edges of the wall flanges. This assumption is believed to be reasonable when considering flexural tensile failure as opposed to the use of a rectangular stress distribution for flexural compressive failure.

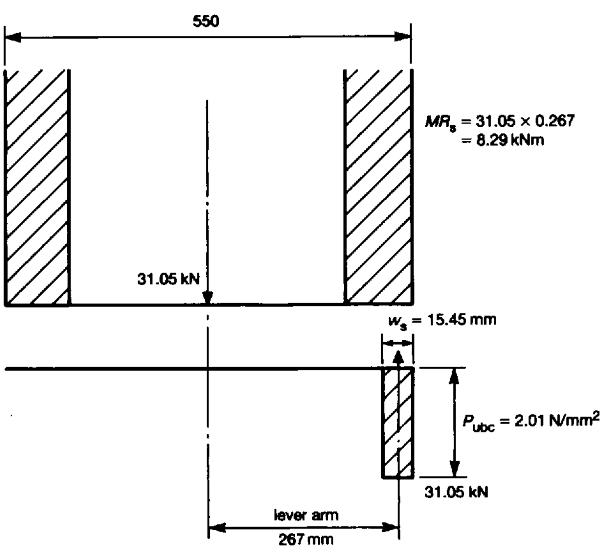

Figure 37: Data for calculating stability moment of resistance

The design axial load again comprises only the design self weight of the masonry; therefore, at the level of M_w:

design axial load $= \gamma_f$ x area x ρ x 3/8 x height
$= 0.9 \times 0.23 \times 20 \times 3/8 \times 7.5$
$= 11.644$ kN

Then, from load/area $\pm$ moment/section modulus, the design flexural tensile stress (f_{ubt}) and the design flexural compressive stress are calculated as follows:

$f_{ubt} = (11.644 \times 10^3)/(0.23 \times 10^6) - (3.6 \times 10^6)/(38.0 \times 10^6)$
$= 0.0506 - 0.0947 = -0.0441$ N/mm²

$f_{ubc} = 0.0506 + 0.0947 = 0.1453$ N/mm² (see Figure 39)

Stage 8. Allowable flexural stresses at level of M_w

(a) Allowable flexural tensile stress

$P_{ubt} = f_{kx}/\gamma_m$

f_{kx} $= 0.25$ N/mm² for concrete blocks with a compressive strength of 7 N/mm² set in a designation (iii) mortar, from Table 3 of BS 5628 : Part 1, for the plane of failure parallel to the bed joints.

γ_m $= 3.5$

$P_{ubt} = 0.25/3.5 = 0.071$ N/mm²

This is greater than the applied $f_{ubt} = 0.0441$ N/mm²: the flexural tensile stresses are therefore acceptable.

(b) Allowable flexural compressive stress

$P_{ubc} = 1.1 \beta f_k/\gamma_m$

f_k $= 6.4$ N/mm² for blocks with a compressive strength of 7 N/mm² set in a designation (iii) mortar (interpolated from Table 2d of BS 5628 : Part 1)

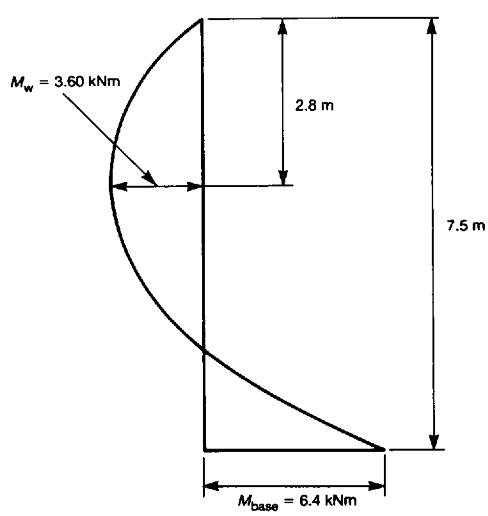

Figure 38: Design bending moment diagram