$\gamma_m = 3.5$

The effective length of the flanges may be taken as 0.75 times the length of the internal void and the effective thickness as the actual thickness of the flange.

Hence, slenderness ratio, $SR = 0.75 \times 1.080/0.10 = 8.10$

The stressed area is trapezoidal in shape but its centroid is unlikely to fall outside 0.05 of the flange thickness as an eccentricity. At this stage of the design the eccentricity cannot be accurately computed as the stress value has not yet been determined. To simplify the calculation, an eccentricity of $0.1 t_f$ will be catered for. Therefore, with $SR = 8.10$ and $e_x = 0.1 t_f$, from BS 5628 : Part 1, Table 7, $\beta = 0.88$.

$P_{ubc} = 1.1 \beta f_k/\gamma_m = 1.1 \times 0.88 \times 6.4/3.5 = 1.77$ N/mm^2

which is greater than the applied $f_{ubc} = 0.1453$ N/mm^2; thus the flexural compressive stresses are also acceptable.

Stage 9. Shear stresses in cross-ribs

Horizontal reaction at base = design shear force V

$V = 5/8 \times \gamma_f \times W_k \times h$
$= 5/8 \times 1.4 \times 0.65 \times 7.5$
$= 4.266$ kN/m
or $= 4.266 \times 1.180 = 5.034$ kN per cross-rib

Design shear stress at cross-rib/leaf interface = v_h

$v_h = K_1 V$
$= 21.24 \times 5.034/10^3$
$= 0.107$ N/mm^2

The maximum shear stress on the bedjoints of the cross-ribs is shown in Figure 40 and is calculated from

$v_h = VA_2\bar{y}/Ib_r$

A_2 = area of hatched portion of wall in Figure 40
$= (1.18 \times 0.1) + (0.175 \times 0.1) = 0.1355$ m^2

$\bar{y}$ = distance from centre line of wall to centroid of area A_2
$= [(1.18 \times 0.1 \times 0.225) + (0.175 \times 0.1 \times 0.0875)]/0.1355 = 0.207$ m

$v_h = (5.034 \times 0.1355 \times 0.207)/(0.0125 \times 0.1 \times 10^3)$
$= 0.113$ N/mm^2

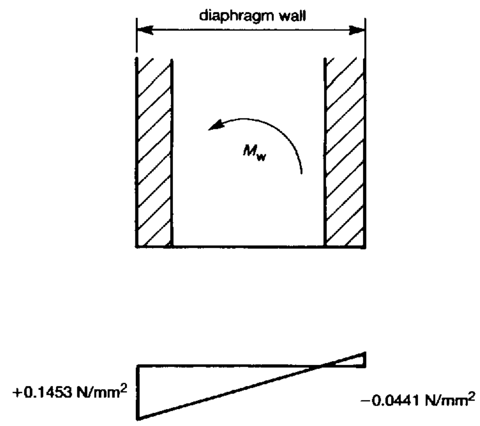

Figure 39: Design flexural stresses

Stage 10. Shear resistance of cross-ribs

The characteristic shear strength of masonry, f_v, is given in BS 5628 : Part 1, Clause 25 as $f_v = 0.35 + 0.6\, g_A$ for mortar designation (iii).

This relates to the shear stress in the bed joint and includes the benefit of the vertical load in the wall.

Hence design shear strength = $(0.35 + 0.6\, g_A)/\gamma_m$

where g_A = design vertical load per unit area
$= \gamma_f \times A \times \rho \times h/A = 0.9 \times 0.271 \times 20 \times 7.5/(0.271 \times 10^3) = 0.135$ N/mm^2

therefore design shear strength $= [0.35 + (0.6 \times 0.135)]/3.5$
$= 0.123$ N/mm^2

which is greater than the maximum shear stress calculated earlier as 0.113 N/mm^2.

BS 5628 : Part 1 does not give shear strengths of blocks subjected to the mode of failure at the bonded vertical interface of cross-rib to leaf in which the blocks themselves, as well as the vertical mortar joints in alternate courses, are in shear.

Until the results of research into this matter are available the shear strength of concrete masonry must be estimated. It is evident that its shear strength will be considerably greater than that of the mortar joints alone (calculated above as 0.123 N/mm^2 for this example). However, the designer must also consider the implications of the bonding arrangement and the

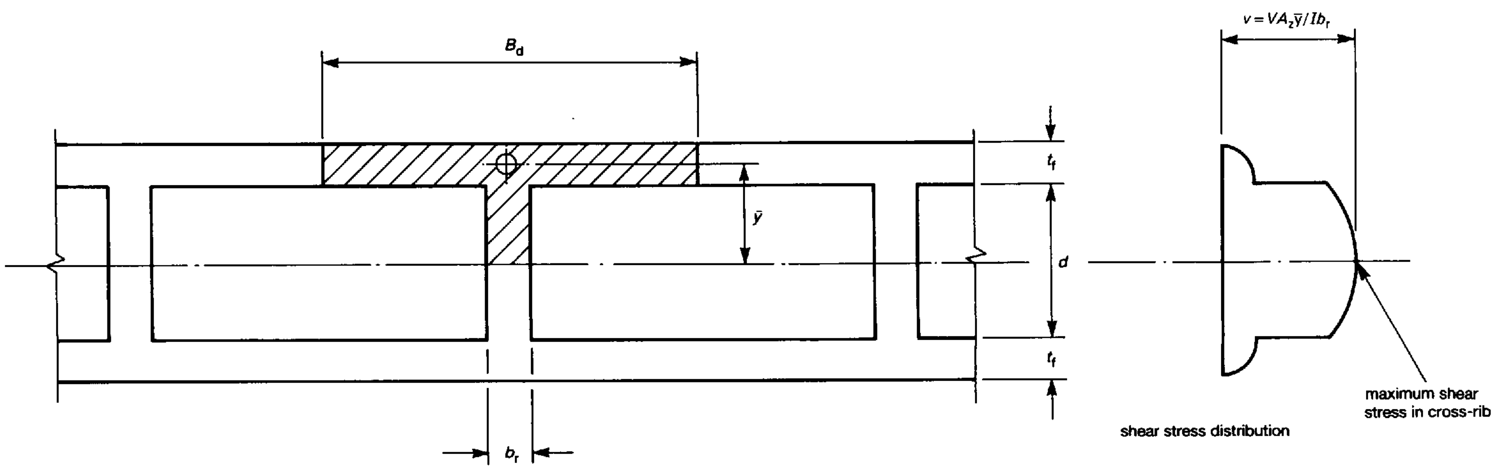

Figure 40: Maximum shear stress in the bed-joints of the cross-ribs

effectiveness of the perpendicular mortar joints at the cross-rib/leaf interface.

It is recommended that where the vertical shear resistance is to be provided by bonded masonry, only units with a minimum compressive strength of 7 N/mm^2 should generally be used. The use of less dense units in this bonded form of construction should be avoided until research into shear reaches a conclusion or adequate test information is provided by the designer. Several design alternatives are available including the use of tied cross-ribs with designed metal shear ties, reducing the spacing of the cross-ribs or increasing the spacing of the leaves. Each of these latter two options reduces the shear stresses on the cross-ribs.

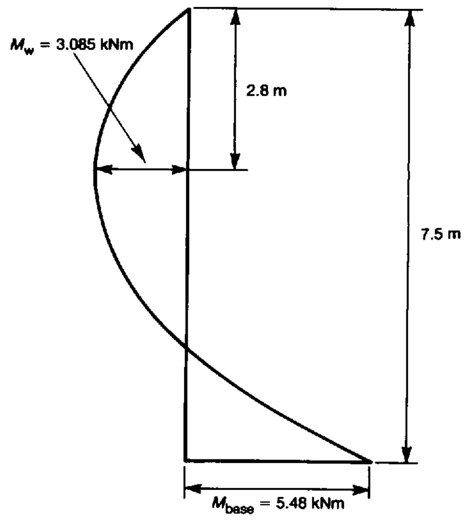

Figure 41: Design bending moment diagram for combined loading

Stage 11. Summarize cross-rib spacing conditions (see Section 2.2.1)

Condition (a) – satisfactory (see stage 5).

Condition (b) $B_{max} = 27 \times t_f$
$= 27 \times 0.10$
$= 2.70$ m

This is greater than the breadth adopted for the trial design, 1.180 m (see stage 2).

Condition (c) $B = 6t_f + 6b_r + 6t_f$ (but $t_f = b_r$)

$\therefore B = 0.10 \times 13$
$= 1.30$ m which is greater than 1.180 m

or $B = h/3$
$= 2.50$ m, which is greater than 1.180 m

Condition (d) was shown to be acceptable in stages 9 to 10.

Stage 12. Roof plate and shear walls

The design of the roof plate and transverse shear walls is outside the scope of this design guide.

Stage 13. Check loading combinations 'dead + superimposed + wind', and 'dead + superimposed'

The most crucial design condition, as stated earlier, is generally that of dead plus wind loading. The flexural tensile stresses are likely to be the limiting factor in the design, as was shown by calculation.

The designer should now check the 'dead + superimposed + wind' loading combination in which the flexural compressive stresses will be greater than those for the 'dead + wind loading' combination. It is expected that they will remain comfortably within the allowable values calculated. The design assumes that the roof beam bearing is detailed such that its loading is applied on the centre-line of the diaphragm wall section.

'Dead + superimposed + wind'

Design loads

Dead + superimposed + wind	$= 1.2\,G_k + 1.2\,Q_k + 1.2\,W_k$
Roof dead load	$= 1.2 \times 0.65 = 0.78$ kN/m^2
Roof superimposed load	$= 1.2 \times 0.75 = 0.90$ kN/m^2
Wind loading on walls	$= 1.2 \times 0.65 = 0.78$ kN/m^2
Wind uplift on roof	$= 1.2 \times 0.65 = 0.78$ kN/m^2

(Dead + superimposed load is greater than wind uplift)

Base wind moment

$M_{base} = \gamma_f W_k h^2/8 = 0.78 \times 7.5^2/8 = 5.48$ kNm

This is less than that for the 'dead + wind' loading combination previously calculated. The stability moment of resistance, allowing for the additional vertical loading, will be greater than previously calculated. Thus, the wall will again be considered to act as a true propped cantilever for this loading combination.

Wall wind moment

$M_w = 9\gamma_f W_k h^2/128 = 9 \times 0.78 \times 7.5^2/128 = 3.085$ kNm

The design bending moment diagram for this loading combination is shown in Figure 41.

Stresses at level of M_w

Design load + superimposed at 3/8 h (40 m span of roof beam)

roof dead load	$= 0.78 \times 40/2$	$= 15.6$
superimposed load	$= 0.90 \times 40/2$	$= 18.0$
self-weight of masonry	$= 1.2 \times 0.23 \times 20 \times 3/8 \times 7.5$	$= 15.5$
	Total	$= 49.1$ kN/m