## SOLUTION LATERAL DESIGN EXAMPLE L2

### Unreinforced Wall

Design bending moment per unit height wall, $M_{Ed} = \alpha W_k \gamma_f L^2$

And $f_{xk1}$ = 0,25 N/mm$^2$ failure parallel to bed joints (Table NA.6)

$f_{xk2}$ = 0,45 N/mm$^2$ failure perpendicular to bed joints (Table NA.6)

Orthogonal ratio $\mu = f_{xk1}/f_{xk2}$ = 0,25/0,45 = 0,556

Aspect ratio, $h/L$ = 3,375/4,500 = 0,75

Support conditions are simple along base and on two vertical sides.

From BS EN 1996-1-1Annex E, panel configuration type A:

For $h/L$ = 0,75 :

$\alpha$ = 0,069 with $\mu$ = 0,6

$\alpha$ = 0,073 with $\mu$ = 0,5

By linear interpolation $\alpha$ = 0,071 with $\mu$ = 0,056

As $\gamma_f$ = 1,5 and $L$ = 4,50 m

Then applied design bending moment per unit height wall is:

$M_{Ed}$ = 0,071 x $W_k$ x 1,5 x 4.50$^2$ = 2,1566 $W_k$ kN.m/m

And design moment resistance of blockwork panel, $M_{Rd} = f_{xk2}Z/\gamma_m$

As $f_{xk2}$ = 0,45 N/mm$^2$ and $t$ = 100 mm

Thus $M_{Rd}$ = (0,45 x 100$^2$ x 10$^{-3}$)/(2,4 x 6) = 0,3125 kN.m/m wall height

Equating $M_{Ed}$ to $M_{Rd}$ :

2,1566 $W_k$ = 0,3125

Therefore $W_k$ = 0,14 kN/m$^2$ for the unreinforced blockwork panel

The characteristic wind load that can be carried by the concrete blockwork panel is 0,14 kN/m$^2$ with no allowance for self weight enhancement and with no consideration of partial propped cantilever effect due to partial base fixity.

**Reinforced Walls**

Limiting dimensions

An overall assessment of serviceability by limiting panel dimensions is required for the reinforced panel (see PD 6697 clause 6.6.2.3)

Actual panel height (*h*) x length (*L*) = 3,375 x 4,500 = 15,188

Limiting ratio = $1600t_{ef}^2 = 1600 \times 0,1^2 = 16,000 > 15,188$

And 60tef = 60 x 0,1 = 6m > 4,50m and 3,375m

Therefore, overall panel limiting dimensions are acceptable.

Method 1 Horizontal spanning reinforced member.

Block strength = 3,6 $N/mm^2$ and based on Equation 3.1 of BS EN 1996-1-1

$f_k = 3,8\ N/mm^2$

Using proprietary 50 mm wide bed joint reinforcement (10 $mm^2$) per wire, then

$d = 75mm$

now $z = d\{ -(0,5A_s f_{yk} \gamma_m / bd f_k \gamma_{ms})\} \leq 0,95d$

Using reinforcement at 450 mm vertical centres $A_s = 22\ mm^2/m$ height

then $z = d\{1 - (0,5 \times 22 \times 500 \times 2,7)/(1000 \times 75 \times 3,8 \times 1,15)\} = 0,95d$

Design moment, $M_{Rd} = A_s f_{yk} z / \gamma_{ms} = 22 \times 500 \times 75 \times 0,95 / 1,15 \times 10^6 = 0,68\ kN.m/m$

As panel is being designed to span horizontally between simple supports then:

$m = W_k \gamma_f L^2 / 8$ and letting $m = M_d$

then $W_k = 0,68 \times 8 / 1,5 \times 4,5^2 = 0,18\ kN/m^2$

The maximum enhancement must not exceed 50% without a serviceability and deflection check. Since some self-weight will be acting then the enhancement does not exceed 50% and, therefore, a value of $W_k$ = 0,18 kN/m$^2$ is acceptable.

Method 2   Excess load taken on reinforcement.
The maximum enhancement that can be taken with this method, without a serviceability and deflection check, is 30%. From the example shown under Method 1, it may be seen that the maximum lateral load that can be taken on the unreinforced block wall is 0,14 kN/m$^2$.
Now 30% of 0,14 = 0,042 kN/m$^2$ which would require a reinforced capacity of:

$M = W_k \gamma_f L^2/8$ = 0,042 x 1,5 x 4,5$^2$/8 = 0,16 kN/m

By comparison with the previous example only about one quarter of the reinforcement would be required. However, both wire and spacing cannot be reduced below 14mm$^2$ at 450mm centres as the minimum requirement (Clause 8.2.3(2)) and therefore in this example the same reinforcement as used for Method 1 would need to be used.

Method 3    Modified orthogonal ratio.

From the unreinforced example:

$f_{xk1}$ = 0,25 and $f_{xk2}$ = 0,45 from Table NA.1 and μ = 0,55

now moment capacity in the horizontal plane:

= $f_{xk1} b\ t^2/6\ \gamma_m$ = 0,25 x 100$^2$ x 10$^{-3}$/6 x 2,4 = 0,17 kN.m/m

Again using 10 mm$^2$ bars at 450 mm centres, then from the example given under Method 1 the moment capacity about the vertical plane:

= 0,68 kN.m/m

Thus, the modified orthogonal ratio = 0,14/0,68 = 0,20

From Annex E of EN 1996-1-1 bending moment coefficient = 0,089

Now the bending moment coefficient is used in conjunction with the strength about the vertical axis. With an unreinforced panel this is taken to be $f_{xk1}$, but with a reinforced panel the strength must be based on the reinforced section which has been previously shown to equal
0,68 kN.m/m.

Now since $m = \alpha W_k \gamma_f L^2$ and $m = M$

then $W_k = M/a\ \gamma_f L^2$ = 0,68 / 0,089 x 1,5 x 4,5$^2$ = 0,25 kN/m$^2$

Allowing for self-weight, $\mu$ = 0,23 and $\alpha$ = 0,087

then $W_k$ = 0,32 kN/m$^2$

The capacity increase is 128% and since it exceeds 50% requires a check on serviceability and deflection.

Method 4  Cracking load.

From the example of the unreinforced panel:

then $W_k$ allowable without self-weight = 0,14 kN/m$^2$

Reducing the safety factors to unity, i.e. $\gamma_f$ and $\gamma_m$ = 1,0

Then characteristic cracking load = 0,14 x 1,5 x 2,4 = 0,50 kN/m$^2$

Allowing for a partial safety factor (1,0) for material strength at the serviceability limit state (NA.2.2)

gives a cracking load of 0,50/1,0 = 0,50 kN/m$^2$

To allow a capacity of 0,50 kN/m$^2$, the wall must be shown to resist this amount by one of the reinforced design Methods 1 to 3 and the walls deflection must be shown to be less than span/250.

**Note**

An elastic analysis may be used to estimate deflections. The following assumptions may be made:

1. The section to be used for the calculation of stiffness is the gross cross section of the masonry, no allowance being made for the reinforcement.
2. Plane sections remain plane.
3. The reinforcement, whether in tension or compression, is elastic.
4. The masonry in compression is elastic. The short term elastic modulus and long term elastic modulus may be obtained from Clause 3.7.2.

The deflection at the appropriate design bending moment may be estimated directly or from the estimated curvature.