SOLUTION TO LATERAL DESIGN EXAMPLE L1

Flexural Design

The design bending moment per unit length of the wall is given by the following expression (Clause 5.5.5):

$$M_{Ed1} = \alpha_1 W_{Ed} \; l^2 \quad \text{eqn. 5.17}$$

Now, the bending moment coefficient depends on:

- orthogonal ratio
- aspect ratio h/L
- edge support conditions

(i) Try masonry unit thickness of 190 mm using 7,3 N/mm^2 compressive strength block (non-normalised strength)

Determine f_{xk1}

for 7,3 N/mm^2 at 100 mm thick f_{xk1} = 0,25
for 7,3 N/mm^2 at 250 mm thick f_{xk1} = 0,15

Therefore by linear interpolation at 190 mm (f_{xk1}) = 0,19

for 7,3 N/mm^2 at 100 f_{xk2} = 0,60
for 7,3 N/mm^2 at 250 f_{xk2} = 0,35

Therefore at 190 mm f_{xk2} = 0,45

Orthogonal ratio, μ $= \dfrac{f_{xk1}}{f_{xk2}} = \dfrac{0.19}{0.45} = 0.42$

(ii) Aspect ratio = h/L = 4,15/4,15 = 1,0

(iii) Support conditions-simple

From BS EN 1996-1-1 Annex E Table A:

for h/L = 1,0:

$\alpha_2 = 0{,}083$ with $\mu = 0{,}5$
$\alpha_2 = 0{,}087$ with $\mu = 0{,}4$

Therefore by linear interpolation:

$\alpha_2 = 0{,}0862$ when $\mu = 0{,}42$ (and $\alpha_1 = \mu\,\alpha_2$)

Since characteristic wind load, $Q_k = 0{,}45$ kN/m^2 and $\gamma_f = 1{,}5$ and $l = 4{,}15$m

then applied design moment per unit length is:

$M_{Ed1} = 0{,}42 \times 0{,}0862 \times 0{,}45 \times 1{,}5 \times 4{,}15^2 = 0{,}42$ kN.m/m

The design moment of resistance is given as (Clause 6.3.1):

$M_{Rd1} = f_{xd}\,Z$

For a panel bending in two directions:

$M_{Rd1} = f_{xk1}\,Z/\gamma_m$

Now, since $f_{xk1} = 0{,}19$ N/mm^2 and $t = 190$

then $M_{Rd1} = \dfrac{0{,}19 \times 190^2 \times 10^{-3}}{2{,}7 \times 6}$

$= 0{,}42$ KN.m/m $(= 0{,}42)$

The moment capacity is therefore adequate

<u>Slenderness Limits (Limiting Dimensions) [Annex F, Figure F3]</u>

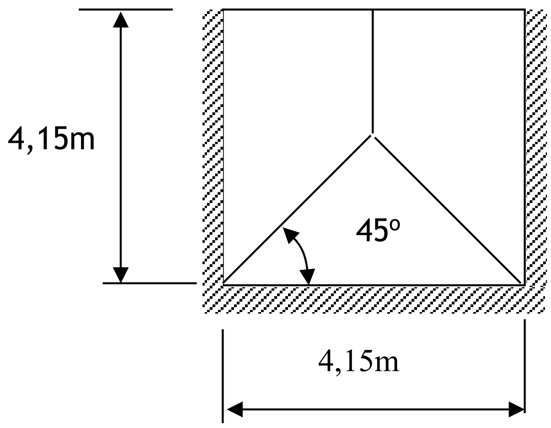
$h/t = 4{,}15/0{,}19 = 21{,}8$

$l/t = 4{,}15/0{,}19 = 21{,}8$

By inspection of Figure F3 this is acceptable

Design for Shear

Consider the wind load to be distributed to the supports as shown below:



Then total load to support is:

= $\gamma_f\, Q_k$ loaded area

Thus the total shear along base

$$= \frac{1{,}5 \times 0{,}45 \times (4{,}15 \times 2{,}075)}{2} = 2{,}91 \text{ kN}$$

Assuming that this load is uniformly distributed along base

The design shear force per m run (V_{Ed}) is therefore:

$= 2{,}91/4{,}15 = 0{,}701$ kN/m

And so the applied design shear stress $= \dfrac{0{,}701 \times 10^3}{190 \times 1000} = 0{,}0037 \text{ N/mm}^2$

The characteristic shear strength from clause 3.6.2 is:

$f_{vk} = f_{vko} + 0{,}4\sigma_d$

Since the self- weight is to be ignored characteristic strength becomes:

$f_{vk} = f_{vko} = 0{,}15$ N/mm^2 minimum (Table NA.5 UK National Annex)

And design shear strength $f_{vd} = (0{,}15/2{,}5) = 0{,}06$ N/mm^2 ($\geq 0{,}0037$ N/mm^2)

Therefore, shear resistance is adequate along base

Now total design shear force to each vertical support is:

$$= 1{,}5 \times 0{,}45 \times 2{,}075 \times \frac{(4{,}15 + 2{,}075)}{2} = 4{,}360 \text{ kN}$$

Again, considering load to be uniformly distributed along support

The design shear force per m run (V_{Ed}) is therefore:

$$= 4{,}36/4{,}15 = 1{,}051 \text{ kN/m}$$

Using 3 mm thickness anchors into dovetail slots in supporting column
Characteristic shear strength of each tie = 4,5 kN, [Manufacturers declaration].

Placing ties at 900 mm centres along vertical edges and taking the partial safety factor for anchor as 3,5 (Table NA.1 of UK National Annex). The design load resistance per metre run of edge support is:

$$= \frac{4{,}5}{3{,}5} \times \frac{1000}{900} = 1{,}43 \text{ kN/m} \quad (> 1{,}05)$$

This is greater than the design shear force and therefore adequate.