R1 Solution: Reinforced Brickwork Pocket-Type Retaining Wall

Characteristic earth loading, G_k = 41,27kN/m from earth loading

Characteristic imposed loading, $Q_{k,1}$ = 10,76kN/m from nominal surcharge loading

And $\gamma_{G,sup}$ = 1,35 and γ_Q = 1,50

Therefore design lateral load = $\gamma_{G,sup} G_k + \gamma_Q Q_{k,1}$

= (41,27 x 1,35) + (10,76 x 1,50)

= 71,85kN/m run of wall

Therefore design moment
= (41,27 x 3,65 x 1,35 / 3) + (10,76 x 3,65 x 1,50 / 2)

= 97,24kN.m/m run of wall

The design total lateral force at the ultimate limit state has been derived from the soil pressure, nominal surcharge loading and partial safety factors for loading as 71,85kN/m and the design bending moment is to be taken as 97,24kN.m/m.

Serviceability limiting span

From BS EN 1996-1-1 Table 5.2 the limiting ratio of the span to effective depth is 18.
Hence the effective depth, d, must exceed l_{ef}/18 which is 3650/18 = 203mm.

Using a 328mm thick brickwork reinforced pocket-type wall (1½-bricks thick) d provided is (328 - 57) = 271 mm, say d = 270mm with 113mm depth reinforcement pockets formed in rear of brickwork wall face.

Section bending capacity

The design bending moment, M_{Ed}, is 97,24kN.m/m run of wall

Design wall as flanged beam, assume pocket width along wall length of 235mm (1-brick)
Flange depth, t_f = d/2 = 270/2 = 135mm

From Clause 6.6.3 (2) width of flange, b_{eft} is lesser of:
a) 235 + (12 x 135) = 1855mm
b) 900 mm (actual pocket spacing to be used)

c) 3650/3 = 1217mm
Therefore flange width to be used, b_{eft} = 900mm

The declared compressive strength of the brick masonry units is 50N/mm^2, Group 1.
Thus normalised compressive strength of the bricks, f_b, is given by:
f_b = 1,0 x 0,85 x 50N/mm^2
f_b = 42,50N/mm^2
where test moisture conditioning factor is 1,0 and 0,85 is the shape factor δ, both taken from BS EN 772 Part 1.

The characteristic compressive strength of the brickwork is given by:
$f_k = K\, f_b^{0,70}\, f_m^{0,30}$ for General Purpose mortar use
where K is taken as 0,5x0,8 for Group 1 clay masonry units in accordance with Table NA.4 and text of the UK National Annex, hence:
$f_k = 0,5 \times 0,8 \text{ x } 42,5^{0,7} \times 12,0^{0,3}$
f_k = 11,63N/mm^2

Using Category I bricks with Class 1 Execution Control the partial factor for materials properties γ_M is 2,0 from Table NA.1 of the UK National Annex
Hence $f_d = f_k/\gamma_M$ = 11,63/2,0 = 5,82N/mm^2

Using the Q factor approach from BS5628 Part 2 which is compatible with the Eurocode 6 methodology then:
The moment of resistance factor Q is given by:

$Q = M_{Ed}/bd^2$ = 97,24 x 10^6 x 0,9 / 900 x 270^2 = 1,334

And $Q = 2c(1\text{-}c)f_d$

Therefore 1,334 = 11,63c – 11,63c^2
c = 0,868 which is the lever arm factor which does not exceed 0,95 limiting
Hence the lever arm z is 0,868 x 270 = 234mm

A_s required = M_{Rd}/f_{yd} z   from Eqn (6.22)

Using Grade 500 deformed steel bars with a yield strength, f_{yk}, of 500 N/mm^2 and with γ_M as 1,15 from the UK National Annex

f_{yd} = 500 / 1,15 = 435N/mm^2

A_s required = 97,24 x 0,9 x 10^6 / 435 x 242
A_s required = 860mm^2

Therefore use 2no. deformed 25mm diameter bars, Grade 500, which provide 982 mm^2. If the bars are placed centrally in the pocket the cover is (113/2 - 25/2) = 44mm which is adequate for durability protection when a C35/45 concrete infilling to BS EN 206 Part 1 and BS 8500 is used for exposure situation MX3. (Table NA.9 of the UK National Annex requires a minimum thickness of concrete cover of 30mm using a 20mm aggregate size).

Main tensile steel provides 3,7% of area of infill pocket which is less than 4% limiting given in Clause 8.2.7(4).

Check that the limiting compressive moment of the brickwork section is not exceeded

$M_{Rd} \leq f_d\, b_{ef}\, t_f\, (d - 0{,}5t_f)$ Eqn (6.28)

$M_{Rd} \leq 7{,}27 \times 900 \times 135 \times 10^{-6}\ (270 - 0{,}5 \times 135)$

$M_{Rd} \leq$ 178,87kN.m for a 900mm flange length (198,74kN.m/m run of wall)

This exceeds the design moment applied, M_{Ed}, of 97,24 kN.m/m run of wall and therefore the brickwork compressive section is satisfactory.

Section shear capacity

The design shear force, $V_{Ed,}$ is 71,85kN/m run of wall

From Annex J for enhanced shear capacity of section incorporating vertical reinforcement:
Characteristic shear strength of section, $f_{vd} = (0{,}35 + 17{,}5 \times \rho)/\gamma_M$ from Eqn. J.1

Where $\rho = A_s/bd = (982/900 \times 270) = 0{,}0040$

$f_{vd} = (0{,}35 + 17{,}5 \times 0{,}0040)/2{,}0 = 0{,}21\text{N/mm}^2$

And $f_{vd} \leq 0{,}7/\gamma_M = 0{,}7/2{,}0 = 0{,}35\text{N/mm}^2$ limiting

f_{vd} may be increased by the factor $\chi = (2{,}5 - 0{,}25a_v/d)$

where a_v = 97,24kN.m/m /71,85kN/m = 1,353m

and $a_v/d = 1353/270 = 5{,}01 < 6$ limiting

$\chi = (2{,}5 - 0{,}25 \times 5{,}01) = 1{,}248$

f_{vd} enhanced = $1{,}248 \times 0{,}21\text{N/mm}^2 = 0{,}26\text{N/mm}^2$

And $f_{vd} \leq 1{,}75/\gamma_M = 1{,}75/2{,}0 = 0{,}88\text{N/mm}^2$ limiting

Therefore shear capacity of section without shear reinforcement, V_{Rd1} is:
$V_{Rd1} = f_{vd}\, b\, d = (0{,}26 \times 900 \times 270 \times 10^{-3}) = 63{,}18$kN for a 900 mm flange length (70,20kN/m run of wall)

This very slightly falls short of the design shear applied, V_{Ed}, of 71,85kN/m run of wall (by 2,3%) and hence 6mm diameter Grade 200 austenitic stainless steel formed closed shear links will be provided at pocket positions only at 225mm vertical centres and placed in brickwork bed joints.

Localised protection of 25mm diameter bars against stainless steel bars will be required at link to main vertical steel spot contact points to avoid potential galvanic action between dissimilar steel types.

Therefore using nominal shear reinforcement localised in pocket positions,

$V_{Rd2} = 0{,}9d\, A_{sw}\, f_{yd}\, (1 + \cot\alpha)\sin\alpha / s$ from Eqn. 6.42 considering reinforced pocket alone as vertical beam element with 235mm width pocket dimension

where $f_{yd} = 200/1{,}15 = 174$ N/mm^2 for grade 200 steel.

$V_{Rd2} = 0{,}9 \times 270 \times 56\text{mm}^2 \times 174 \times 10^{-3} / 225 = 10{,}52$kN (11,69/m run of wall)
where A_{sw} is 56mm^2 for 2no. 6mm diameter bars in formed closed shear link and $(1 + \cot\alpha)\sin\alpha = 1{,}0$ for horizontally placed shear links ($\alpha = 90°$)

$V_{Rd1} + V_{Rd2} = 70{,}20 + 11{,}69 = 81{,}89$kN/m run of wall

This exceeds the design shear applied, V_{Ed}, of 71,85kN/m run of wall

$(V_{Rd1} + V_{Rd2})/ t\, l = (81{,}89 \times 0{,}9 \times 10^3) / (328 \times 900) = 0{,}25$N/mm^2

And $(V_{Rd1} + V_{Rd2})/ t\, l \leq 2{,}0$N/mm^2 limiting from Eqn. 6.37

And $V_{Rd1} + V_{Rd2} \leq 0{,}25 f_d\, bd$ limiting from Eqn. 6.43

$0{,}25 f_d\, bd = 0{,}25 \times 7{,}27 \times 235 \times 270 \times 10^{-3} = 115{,}32$kN (128,13kN/m run of wall)

And 128,13kN/m is greater than 81,89/m run of wall.

Note that spacing of shear links at 225mm vertical centres slightly exceeds 0,75d (203mm) or 300mm, whichever is lesser, as given by Clause 8.2.7(6), but as any shear steel requirement is very marginal here this is considered practically acceptable in this particular design case.

Reinforced brickwork wall shear capacity is adequate.

Reinforcement lapping

Consider now whether the main tensile reinforcement can be curtailed

Use 20mm diameter reinforcement to lap on. In order to curtail the 25mm diameter bars in the tension zone, Clause 8.2.5.4 must be satisfied. The moment condition is appropriate and therefore the design moment capacity of the 20mm diameter bars must be at least twice the design moment due to applied lateral loads for curtailment.
Design moment capacity of 2 no. 20mm diameter bars per pocket,
$A_s = 628\text{mm}^2$, assume $c = 0{,}905$

M_{Rd} = $(628 \times 0{,}905 \times 270 \times 435 \times 10^{-6})$ = 66,75kN.m per 900mm pocket spacing
$Q = M/bd^2 = (66{,}75 \times 10^6) / (900 \times 270^2) = 1{,}017$ and f_d = 5,82N/mm^2 as before.
c = 0,905 is the correct assumption substituting into and solving the interaction equation for c from above

Thus 25mm diameter bars may be stopped where the design moment due to applied lateral loads, M_{Ed} = (66,75 x 0,5/0,9) = 37,08kN.m/m run of wall.

From a consideration of the design lateral loads upon the wall stem the applied design moment of 37,08kN.m/m run of wall is achieved at 2,56m from the top of the wall (from retained earth level).
Check that 25mm diameter bars will extend at least an effective depth or 12 bar diameters, whichever is greater, beyond the point where they are no longer needed. The 20mm diameter bars will resist a design a moment of: 67,86/0,9 = 75,40kN.m/m run of wall

From a consideration of the design lateral loads this will occur at 3,32m from the top of the wall. Therefore curtailment of 25mm diameter bars at 2,56m below top of wall is satisfactory because:

a) An effective depth above this level is (3,32 – 0,270) = 3,05m, therefore curtailment of 25mm diameter bars at 2,56m from top of wall is satisfactory,

b) And 12 diameters above this level (3,32 - 12 x 0,025) = 3,02m, therefore curtailment of 25mm diameter bars at 2,56m from top of wall is also satisfactory

Check lap length for 20mm diameter Grade 500 bars
Characteristic bond strength, f_{bok} = 3,4N/mm^2 from Table 3.6
And design bond strength, $f_{bod} = f_{bok}/\gamma_M$ = 3,4/1,5 = 2,26N/mm^2 where γ_M is 1,5 from the UK National Annex.

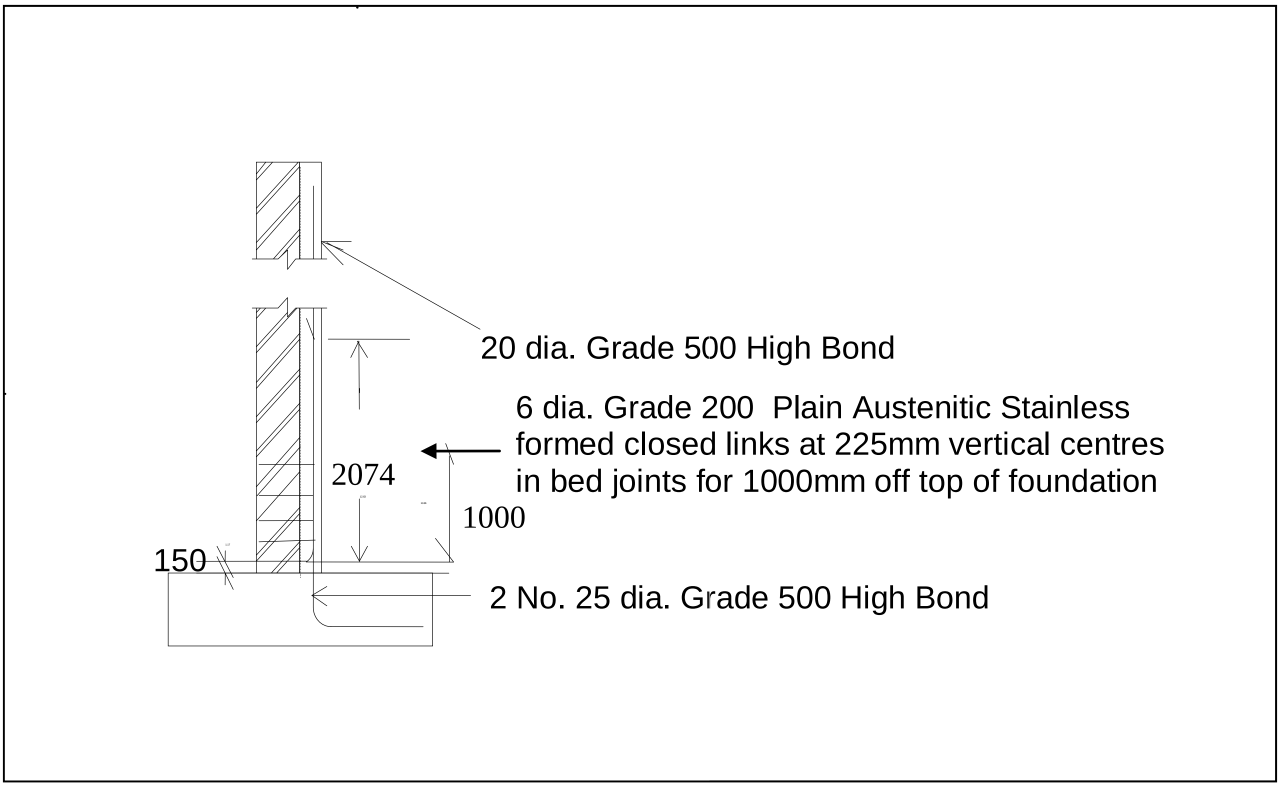

Figure 2: Arrangement of reinforcing steel

Therefore lap length required to achieve full anchorage bond, l_b is:

$l_b = \phi f_{yd} / 4 f_{bod}$ from Eqn. 8.1

$l_b = (20 \times 435) / (4 \times 2{,}26) = 962\text{mm}$

And ($2 l_b$) = 1924mm and therefore governs the lap from Clause 8.2.5.2(3)

Therefore curtail 25mm diameter bars 2074mm above foundation base and continue 20mm diameter bars down to within 150mm of foundation base. This provides a lap length for 20mm diameter bars of 1924mm which is satisfactory.

It may be noted that at 2074mm above the spread foundation base the applied design shear load is 17,36kN/m run of wall from a consideration of the lateral loads acting on the wall stem and the corresponding design shear resistance is 54,00kN/m run of wall taking f_{vd} as 0,20N/mm^2 minimum for 20mm diameter continuing bars with no shear span enhancement.

Consider the necessary extent up the wall stem for the 6mm diameter shear links.

If shear links are provided from the base of the wall stem (from the top of the spread foundation level) to 1000mm above this position only then from a consideration of the design lateral loads upon the wall stem the design shear force applied at this level,

V_{Ed}, is 41,09kN/m run of wall

And minimum design shear resistance of section at this level ignoring any shear span enhancement and without shear link contribution is:

$V_{Rd1} = f_{vd}\, b\, d = (0{,}21 \times 900 \times 270 \times 10^{-3}) = 51{,}03$kN for a 900 mm flange length (56,70kN/m run of wall)

And 56,70kN/m exceeds 41,09kN/m run of wall

Therefore 6mm diameter formed closed shear links at 225mm vertical centres should be provided for a distance of 1,0m up from the base of the wall stem (top of spread foundation) only to each reinforced pocket position along the wall length.