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The logo for ROSE ASSOCIATES features the word "ROSE" in a large, blue, serif font, with a stylized circular graphic to its left. To the right of "ROSE" is the word "ASSOCIATES" in a smaller, blue, sans-serif font. Below this, the words "CIVIL & STRUCTURAL ENGINEERS" are written in a small, black, sans-serif font.

ROSE ASSOCIATES
CIVIL & STRUCTURAL ENGINEERS

40 COOLHURST ROAD
LONDON
N8 8EU

STRUCTURAL CALCULATIONS

JOB NO: 26023
ISSUED: JUNE 2026

REV A 08/06/2026.
Page 2:01

PROPOSALS

The works are for the replacement of the existing timber fence between the subject property and the adjacent "downhill" house, No 38. There is a stepped difference in ground level between the two properties.

Reference to the British Geological Survey indicate the underlying ground conditions are London Clay formations, clay, silt and sand. Further reference to BGS borehole records provides details for 18 Coolhurst Road reference TQ28NE317, copy attached which details topsoil over a clay bound fill over soft grey clay with peat overlying firm brown clay which is consistent with London Clay.

Hence for the purposes of our design the ground will be taken as a shrinkable clay.

Towards the rear of the proposed works are trees in close proximity to the proposed boundary wall.

Given the works are for a boundary wall it is not cost effective to provide deep foundations beyond the zone of tree root activity so a degree of shrinkage and swelling, i.e. subsidence and heave could be expected. There is also the issue of potential made ground and the influence any deep-seated peat that may also have an effect on the performance of the new garden wall.

If the works were for a domestic extension a detailed site investigation should be undertaken which could lead to a piled foundation solution. For a boundary wall these costs and a piled solution are not economically viable, so a pragmatic solution has been adopted.

So, in an attempt to mitigate potential ground movement, the wall is to be built in discrete panel. Structural crack damage usually occurs as a result of angular distortions so discrete panel lengths can allow a degree of movement between panels where expansion joints between can accommodate a degree of predicted movement.



TERRESEARCH LIMITED

1:02.

BOREHOLE NO. 2

Contract Name Coolhurst Road, Report No. S. 364/11
Client Star (Greater London) Site Address 13, Coolhurst Road,
Address Holdings Ltd., London, N. 16

Standing Water Level 14'0" (see below) Diameter 6"
Water Struck 13'0" Method of Boring Auger
Ground Level 15.9.61 Start 15.9.61 Finish 16.9.61

Remarks:

Seepage of water at 13'0". Borehole dry when completed -
water rose to 14'0" depth in 3 Hours.

Description of Strata	Thickness	Depth	Disturbed Samples	'U' Cores a 'N' P. Tes
Top soil	1'6"	1'6"	J12491 1'6"	U12493 1'6"
			J12493 3'0"	U12494 8'0"
			J12495 9'6"	U12498 12'
Brown clay (made ground)	11'0"	12'6"	J12497 14'0"	U12498 18'
Soft grey clay with peat	2'6"	15'0"	J12499 20'0"	
			W12500	
Firm brown clay	5'0"	20'0"		
TOTALS	20'0"	20'0"		

PROJECT: 40 COOLHURST ROAD
LONDON
N8 8EU

Section: WIND LOADS

DIRECTIONAL FACTOR

Job No: 26023

Calculations:

Date: JUNE 2026

Page No:

REV A. 08/06/26
2 of 01

WIND C 65° Sd=0.73 WIND C 115° Sd=0.73



WIND LOADS

Whilst the bulk of the proposed new brick boundary wall is shielded between the two rear additions the section beyond an alignment with the end walls and to the side of the terrace, which is at its tallest at 3m height, will be subject to wind loads.

Table 3 wind direction factor $S_d = 0.73$.

Figure 6, basic wind speed $V_b = 21 \text{ m/s}$

Distance to sea, approximately 63km

$$S_1 = 1.0 \quad S_p = 1.0 \quad S_a = 1 + 0.001(63) = 1.063 \quad S_s = 1.0$$

Max height of wall 3m.

Table 1. $K_d = 0.5$ hence $C_e = 0.01$.

Site > 2km into town

Table 4.	Eff. ht.	S_b .
	$\leq 2\text{m}$	1.10
	$\leq 5\text{m}$	1.40.

$$V_s = V_b \times S_a \times S_d \times S_s \times S_p$$

$$V_s = 21 \times 1.063 \times 0.73 \times 1.0 \times 1.0 = 16.4 \text{ m/s}$$

$$V_e = V_s \times S_b. \quad q < 0.613 V_e^2.$$

$$\leq 2\text{m} \quad V_e = 16.4 \times 1.1 = 18.04 \quad \therefore q < 0.2 \text{ kN/m}^2.$$

$$\leq 5\text{m} \quad V_e = 16.4 \times 1.4 = 23.0 \quad \therefore q < 0.32 \text{ kN/m}^2.$$

Net pressure coefficients for free standing walls:-

Take $C_{as} = 1.0$ and without return walls.

Zone A = 3.4
Zone B = 2.1
Zone C = 1.7
Zone D = 1.2

Net pressure coefficients for
free standing walls.

$$\frac{C_{pe}}{C_{pi}} = \frac{18000 \text{ max}}{3000 \text{ max}} = 6 \quad \text{Hence reduction factor } K = 0.74$$

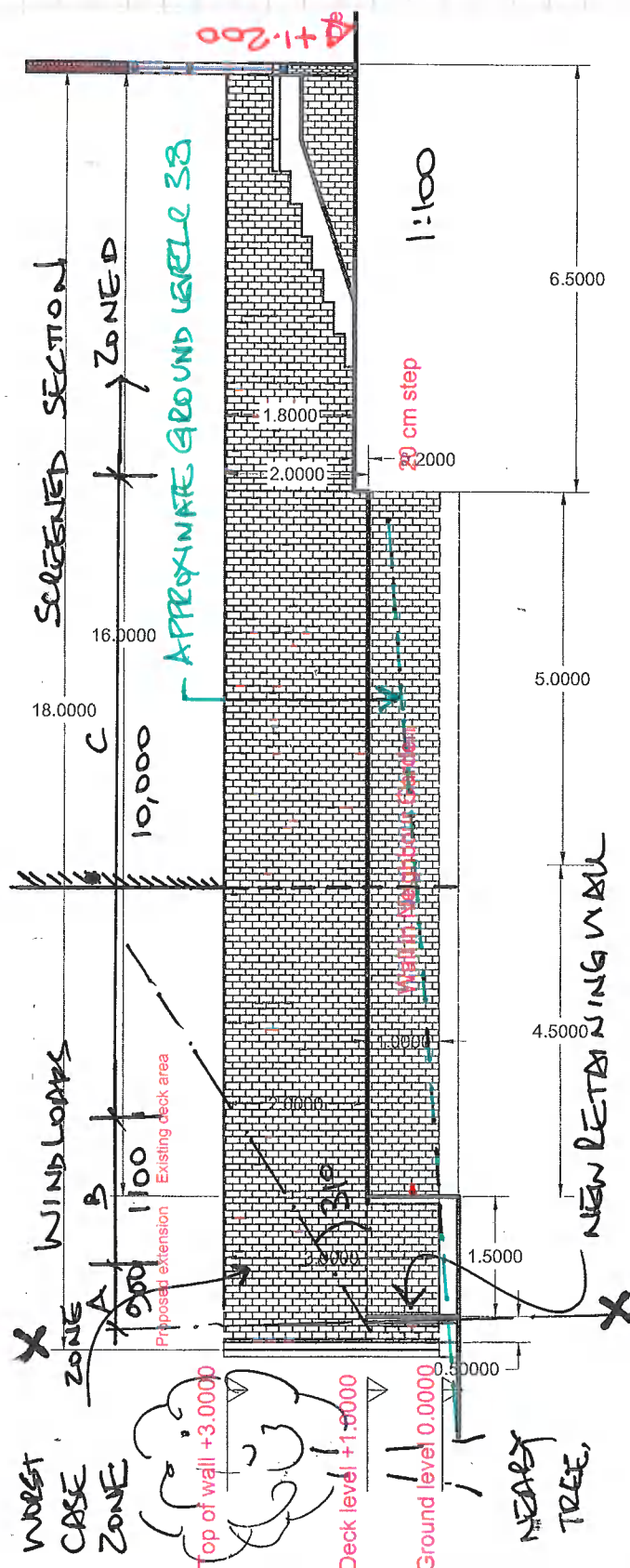
Lengths of zones

$$A = 0.3h = 0.3 \times 3.0 = 0.9 \text{ m}$$

$$B = 2h = 2 \times 3.0 = 6 \text{ m from free edge}$$

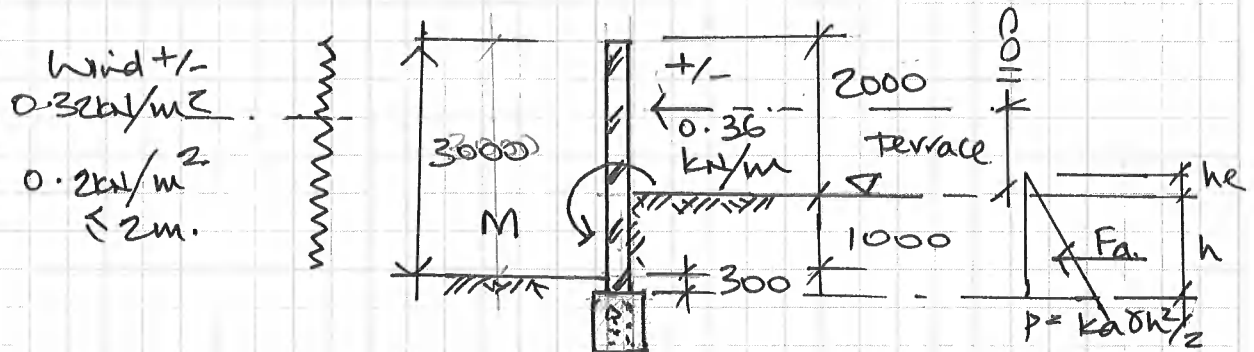
$$C = 4h = 12 \text{ m from free edge}$$

D, beyond 12m.



Note, the extent of the existing terrace is to be extended by 1.5m as shown requiring a new retaining wall across the width of the garden

Consider boundary retaining wall at X-X



check stability of wall, worst case with retained soil.

Take surcharge load on terrace, domestic load only

Live load = 1.5 kN/m, take as h_e

Backfill to raise terrace, granular type material.

Take $\gamma = 1900 \text{ kg/m}^3$

$\theta = 30^\circ$

Ignore passive pressure for stability on first 300mm depth

Bending at base of wall.

Case (1) 1.4gk + 1.6qk

Earth pressures. $h_e = \frac{1.5(1.6/1.4)}{19} = 0.09 \text{ m}$.

Pressure at base of wall $p = \gamma h^2 / 2 K_a$

Where $K_a = \frac{1 - \sin \theta}{1 + \sin \theta} = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = 0.33$

Hence $p = 0.33 \times 19 \times 1.0 = 6.3 \text{ kN/m}^2$

$F_a = 6.3 \times 1.0 / 2 = 3.15 \text{ kN/m}$

$M = 3.15 \times 0.33 = 1.04 \text{ kNm}$

} Service loads

Bending due to lateral pressure from "banstrade"
load

$$M_b = 0.36 \times 2.4 = 0.86 \text{ kNm}$$

$$M_T = 1.04 [1.4] + 0.86 [1.6]$$

$$= 1.9 \text{ kNm service} \text{ \& } 2.83 \text{ kNm UH}$$

Case (ii) $1.2 \text{ gk} + 1.2 \text{ qk} + 1.2 \text{ wk}$.

At exposed corner retaining wall restrained by return
retaining wall across width of garden, hence zone A
loads over first 900mm not critical.

Take zone B factors as worst case

Hence bending due to wind;

$$wk = 0.2 (2.1 \times 0.74) 2 \times 3.3 = 2.05 \text{ kNm service}$$

Hence ultimate bending at base of wall, -

$$1.04 [1.2] + 0.86 [1.2] + 2.05 [1.2]$$

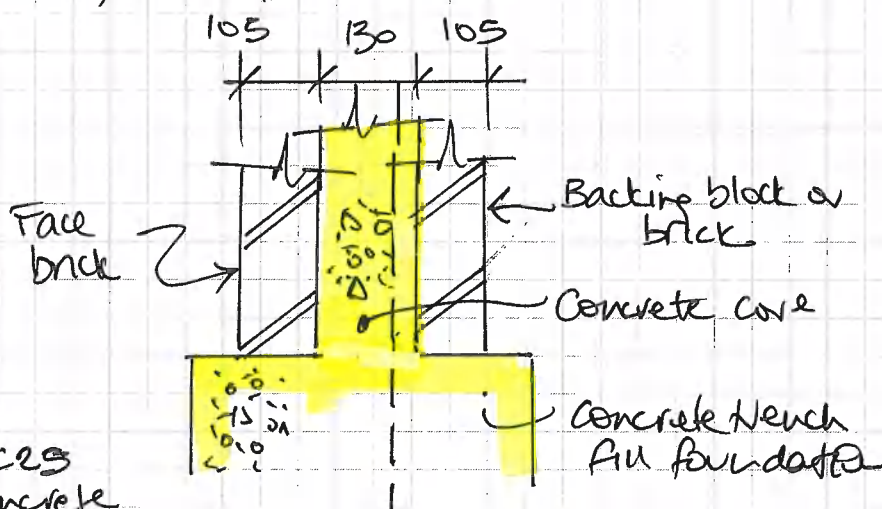
$$= 1.25 + 1.03 + 2.46$$

$$= 4.74 \text{ kNm}$$

Hence case (ii) loads critical.

Try partially reinforced masonry wall
to deal with retained section of wall

Try face brick, cavity, face brick



Try grade C25
site mix concrete

$$\begin{aligned} H &= 130 \text{ core width} \\ \text{cover} &- 40 \\ 1/2\phi &- 5 \\ \hline d &= 85\text{mm} \end{aligned}$$

Check bending :-

$$\frac{M}{F_{m,k} b d^2} = \frac{4.74 \times 10^6}{25 \times 10^3 \times 85^2} = 0.026 \therefore z = 0.95$$

$$A_s = \frac{4.74 \times 10^6}{0.87 \times 500 \times 0.95 \times 85} = 135\text{mm}^2$$

Hence use A252 fabric reinforcement

check bending at new terrace floor level, ie
free standing wall at 2m height

CHECK BENDING AT NEW TERRACE LEVEL.

Again using case(ii) loads as worst case

$$M = 0.36 \times 1.1 \times 1.2 + 0.2 \times \frac{2^2}{2} \times 1.2 (2.1 \times 0.74) \\ = 0.48 + 0.62 = 1.1 \text{ kNm ul}$$

Check solid 340 brickwork

$$Z = \frac{10^3 \times 340^2}{6} = 19.356 \text{ mm}^3$$

$$M_{1/2} = \frac{1.1 \times 10^6}{19.3 \times 10^6} = 0.06 \text{ N/mm}^2 \text{ ultimate +/-}$$

Check flexural strength in bed joint parallel to joints.

Table. 3, bricks with absorption between 7% & 12%

flex parallel to bed joints $\geq 0.4 \text{ N/mm}^2$ in M4(iii) mortar

Take $\gamma_m = 3.1$ since brick manufacturers quality assured

Hence excluding compression due to self wt of wall

$$\text{Allowable flexural strength} < \frac{0.4}{3.1} = 0.129 \text{ N/mm}^2 \text{ ul}$$

Since $0.129 > 0.06 \text{ N/mm}^2$ solid 340 brickwork OK
check solid 220 thick brick capacity

$$Z = \frac{10^3 \times 220^2}{6} = 8.07 \text{ E}6 \text{ mm}^3$$

Case(ii) bending c 1.1 kNm

$$M_{1/2} = \frac{1.1 \times 10^6}{8.07 \times 10^6} = 0.136 \text{ N/mm}^2 (+/-). \\ < 0.129 \text{ N/mm}^2$$

$$0.9 \text{ wk} = 0.9 \times 0.22 \times 2.0 \times 19 = 7.52 \text{ kN/m ul}$$

Hence precompression in wall

$$\sigma_c = \frac{7.52 \times 10^3}{220 \times 10^3} = 0.036 \text{ N/mm}^2$$

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Combined stresses become:-

$$0.036 \pm 0.136 \text{ N/mm}^2$$

$$\text{ie } 0.172 \text{ N/mm}^2 \text{ max compression}$$

$$0.1 \text{ N/mm}^2 \text{ max tension} < 0.120 \text{ N/mm}^2$$

Hence flexural tension ok

Summary for wall:-

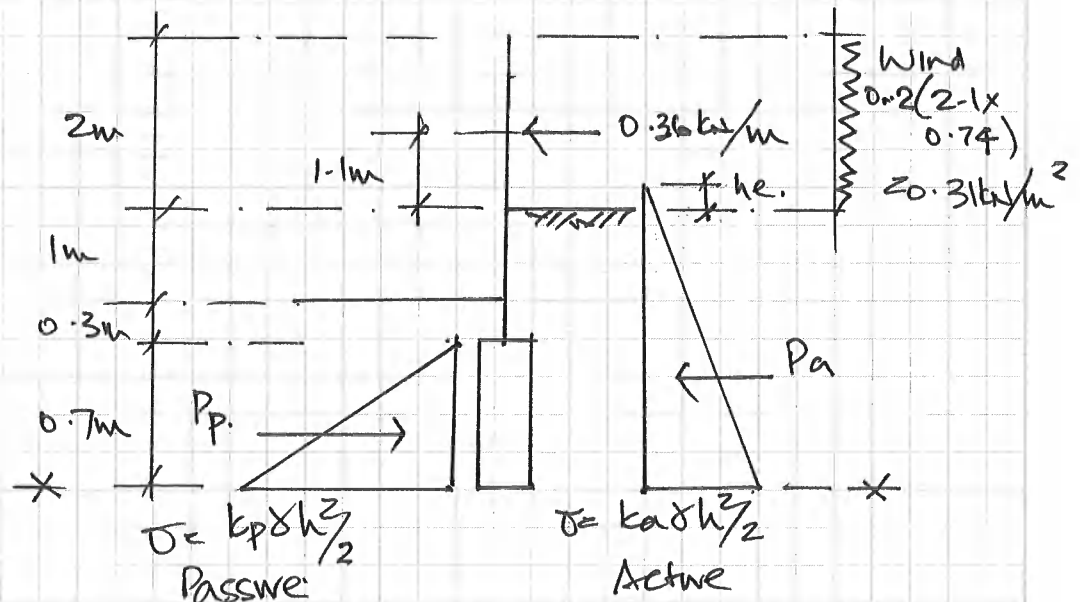
Provide reinforced masonry as retaining wall

340 overall thickness reducing to 220 once
wall once above new kerb level.

FOUNDATIONS

check foundation requirements

Try 1m deep foundations, overall depth measured
from neighbouring lower ground level, cast to
within 300mm of existing garden level.



Consider earth pressures

Active pressure at base of foundation

$$\sigma_a = 0.33 \times 19 \times 2.08 = 13.04 \text{ kN/m}^2 \text{ service}$$

$$P_a = 13.04 \times 2.08 / 2 = 13.6 \text{ kN/m service}$$

$$M_{xx} = 13.6 \times 2.08 / 3 = 12.7 \text{ kNm}$$

Add wind + lateral load bending about x-x.

$$\leftarrow 0.36 \text{ kN/m load } M = 0.36 \times 2.0 = 1.2 \text{ kNm}$$

$$\text{wind } 0.31 \times 2.0 \times 3 = 1.9 \text{ kNm}$$

$$M_{T. \text{ about } x-x} = 12.7 + 1.2 + 1.9 = 15.8 \text{ kNm}$$

$$\text{And lateral load } F_T = 13.6 + 0.36 + (0.31 \times 2.0) \\ = 14.6 \text{ kN/m}$$

These forces to be resisted by passive pressures

Passive pressure at base of foundation

$$\sigma_p = \frac{1}{0.33} \times 19 \times 1.0 = 57.6 \text{ kN/m}^2 \text{ service}$$

$$P_p = 57.6 \times 0.7 / 2 = 20.2 \text{ kN/m}$$

Factor of safety against sliding

$$20.2 / 14.6 = 1.38 < 1.5, \text{ additional depth reqd}$$

$$\text{And stability moment } 20.2 \times 0.7 / 3 = 4.71 \text{ kNm} \\ < 15.8 \text{ kNm}$$

Pressures at base of mass concrete foundation

$$\text{Active } 0.33 \times 19(2 + 0.08) = 13.0 \text{ kN/m}^2$$

$$\text{Hence } P_a = 13.0 \times 2.08 / 2 = 13.5 \text{ kN/m}$$

$$M_{xx} = 13.5 \times 2.08 / 3 = 9.4 \text{ kNm}$$

Passive pressures.

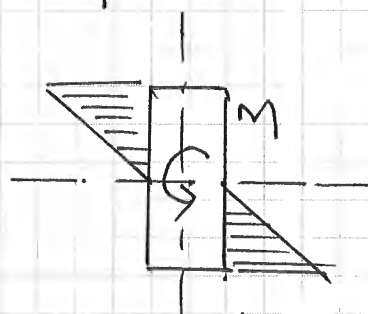
$$\sigma_p = \frac{1}{0.33} \times 19(1.0) = 57.6 \text{ kN/m}^2$$

$$P_p = 57.6 \times 0.7 / 2 = 20.2 \text{ kN/m}$$

$$M_p = 20.2 \times 0.7 / 3 = 4.7 \text{ kNm}$$

$$\text{Sliding F.O.S} = \frac{20.2}{13.5} = 1.496 \geq 1.5. \text{ just ok}$$

Consider overturning :- Overturning moment $\approx 9.4 \text{ kNm}$



Z face in contact with ground

$$1 \times 0.7^2 / 6 = 0.082 \text{ m}^3$$

$$\text{Net overturning } M_{\text{net}} = 15.8 - 9.4 = 6.4 \text{ kNm}$$

$$M/Z = \frac{6.4 \times 10^6}{0.082 \text{ m}^3} = 78 \text{ kN/m}^2 \leftarrow \text{assay/k}^2$$

Bearing pressures on face of concrete OK

But additional depth required to prevent sliding

Try 1200 deep foundation, ie concrete 900 deep

Active pressure at base of foundation

$$\sigma_a = 0.33 \times 19 \times 2.28 = 14.3 \text{ kN/m}^2$$

$$P_a = 14.3 \times 2.28 / 2 = 16.3 \text{ kN/m}$$

$$M_a = 16.3 \times 2.28 / 3 = 12.4 \text{ kNm}$$

Passive pressure at base of foundation

$$\sigma_p = \frac{1}{0.33} \times 19 \times 1.2 = 69.1 \text{ kN/m}^2$$

$$P_p = 69.1 \times 0.9 / 2 = 31.1 \text{ kN/m}$$

F.O.S against sliding $\frac{31.1}{16.3} = 1.91 > 1.5$ sliding OK

$$M_p = 31.1 \times 0.9 / 3 = 9.33 \text{ kNm} \quad Z_{\text{found}} = \frac{1 \times 0.9^2}{6} = 0.135 \text{ m}^3$$

Bearing pressures on face of foundation

$$B.P. = \frac{(12.4 - 9.33) \times 10^6}{0.135 \times 10^6} = 23 \text{ kN/m}^2 < 100 \text{ kN/m}^2 \text{ bearing OK}$$

LOAD AT BASE OF WALL

S.W.T 220 section

$$0.22 \times 19 \times 2.0 \text{ ht} = 8.4$$

340 section

$$0.22 \times 19 \times 1.3 \text{ ht} = 5.4$$

concrete core

$$0.13 \times 24 \times 1.3 \text{ ht} = 4.1$$

Trench fill conc.

$$0.5 \times 23 \times 0.9 \text{ ht} = 10.4$$

kN/m service

$$28.3 \text{ kN/m}$$

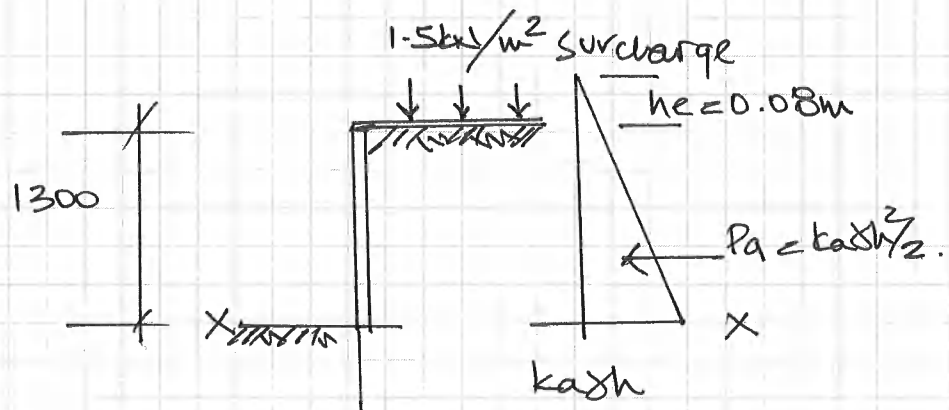
Check 600 width

$$B.P. = \frac{28.3}{0.6} = 47 \text{ kN/m}^2 < 100 \text{ kN/m}^2$$

Hence 600 wide trench for foundations OK

TERRACE RETAINING WALL

This wall is required to support the terrace only up to a maximum retained height of 1.3m.



$$k_a \gamma h = 0.33 \times 19 \times 1.38 = 8.7 \text{ kN/m}^2$$

$$P_a = 8.7 \times 1.38 / 2 = 6 \text{ kN/m}$$

$$M_{ax} = 6 \times 1.38 / 3 = 2.8 \text{ kNm service.}$$

$$\text{OR } 2.8 \times 1.4 = 3.9 \text{ kNm}$$

220 thick brickwork, semi-engineering bricks

with a water absorption of less than 7% in M4 mortar.

Flexural strength f_{tx} (TB 3) = 0.5

$$Z_{220 \text{ wall}} = \frac{10^3 \times 220^3}{6} = 8.07 \text{ Ebmm}^3$$

Bending stress at base of wall

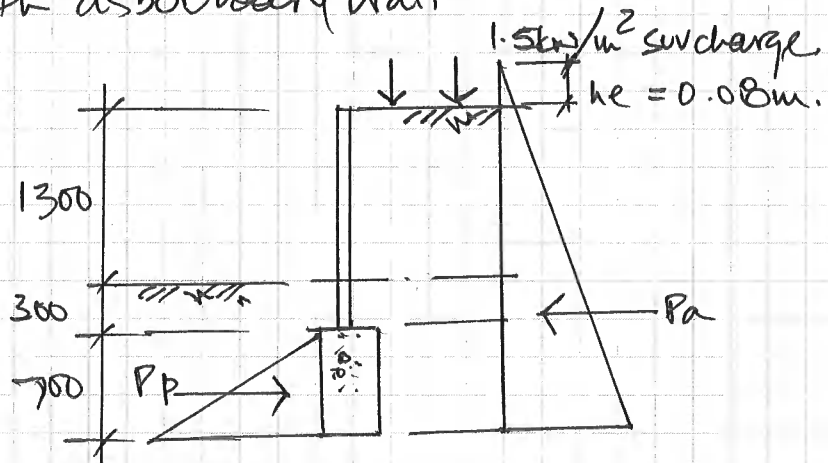
$$M_{\frac{1}{2}} = \frac{3.9 \times 10^6}{8.07 \times 10^6} = 0.48 \text{ N/mm}^2$$

$$> \frac{0.5}{3.1} = 0.16 \text{ N/mm}^2$$

Use concrete core wall as boundary wall.

Foundations to terrace retaining wall

Try 200 depth as boundary wall



$$O.T.M = 0.33 \times 19 \times 2.38^3 / 6 = 14.1 \text{ kNm service}$$

$$E.M = \frac{1}{0.33} \times 19 \times 1.0^3 / 6 = 9.6 \text{ kNm service}$$

$$\text{Net over turning} = 14.1 - 9.6 = 4.5 \text{ kNm}$$

Bearing pressures as before

$$\frac{M}{Z} = \frac{4.5}{0.082} = 55 \text{ N/m}^2 < 100 \text{ kN/m}^2 \text{ Bearing OK}$$

Consider balustrade attached to top of wall

Additional overturning moment using 0.74 kN/m service load

$$O.T.M \text{ about base of foundation } 0.74 \times 3.4 = 2.52 \text{ kNm}$$

Hence net overturning of system is 4.5 + 2.52 = 7.02 kNm

$$\text{Bearing pressures:- } \frac{7.02}{0.082} = 86 \text{ kNm} < 100 \text{ kNm}$$

System still OK.

CHECK SLIDING

$$P_a = 0.33 \times 19 \times \frac{2.08^2}{2} = 13.6 \text{ kN/m service}$$

$$P_p = \frac{1}{0.33} \times 19 \times \frac{1^2}{2} = 28.8 \text{ kN/m service}$$

$$\text{F.O.S against sliding} = \frac{28.8}{13.6} = 2.1 > 1.5$$

Hence sliding ok

use 1m deep mass concrete foundation