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CONSTRUCTION METHOD STATEMENT
53 Ferme Park Road, London N4 4EB

This Construction Method Statement is produced for submission to the London Borough of Haringey planning department for planning application only and should not be used for any other purposes, e.g. Party Wall Awards.

SCOPE OF WORKS

The existing basement at the property is to be extended under the footprint of the property plus the proposed side extension, including creating a front lightwell.

DESCRIPTION OF 53 Ferme Park Road AND ADJOINING PROPERTIES

The property is a three storey end of a terrace house. It is of masonry construction with timber floors to all levels and timber rafters to form the roof. The property is in sound condition structurally. The adjoining properties are of similar construction and look to be in sound condition from an external non – intrusive visual examination. Neither the property nor any of the adjoining properties are listed on the Statutory List of Buildings of Special Architectural or Historic Interest.

SOIL CONDITIONS

To support this Construction Method Statement we confirm that we have carried out a soil investigation via two trial holes as can be seen in photos below, the depths we excavated to were equal to this proposal and we can confirm that no groundwater was encountered. Borehole logs from the British Geological Society confirm that ground conditions generally comprise of firm/stiff brown clay (copies of bore hole records atrtached). The new basement will be designed to limit ground – bearing pressure to 100 kN/m2, which we consider to be conservative. Thus, the existing geology at the depth of the proposed basement will be capable of supporting the new imposed loads.

Trial hole 1

Trial hole 2

UNDERPINNING DRAWING

Please refer to drawings no. 217-FR-01, 217-FR-02 & 217-FR-03 in the Appendices for an underpinning sequence to the property and typical sections.

CONSTRUCTION SEQUENCE

1. Excavation will commence from the front of the property, progressing towards the rear.

2. A conveyor belt will be set up through the existing basement to convey the spoil from the excavation to the skip placed on the road for disposal.

3. The existing property will be underpinned in the usual 1 to 5 underpinning sequence. See drawing no. 217-FR-01 for the construction sequence of a typical underpin, and see Underpinning Specification in the Appendices.

4. The underpins to form the new basement may require horizontal propping until completion of the basement slab.

5. As excavation progresses, any existing foundations found will be broken out and removed from site to make way for the new basement construction.

6. The existing walls of the building over will be temporarily propped using steel beam needles at regular centres, as necessary. Temporary concrete pad foundations may be required beneath the props, or the props may be supported on the concrete bases of underpins already constructed, wherever the location allows.

7. New concrete pad foundations and strip foundations will be constructed, where specified on the structural drawings.

8. New steel beams and columns will be installed, as specified on the structural drawings. These will be supported on the underpins or on the new concrete foundations. Steel beams supported by existing masonry walls will bear on concrete padstones, as specified on the structural drawings. The padstones will spread the load on the existing masonry down to acceptable levels.

9. The top of the new steel beams will be dry – packed to the underside of the existing walls above, and the existing walls will be repaired and made good, as required.

10. When all the underpins to the existing property have been completed, bulk excavation to the whole site will be carried out.

11. Horizontal propping across the site, if required by design, will be installed at high level. This will be via a proprietary propping system such as Mabey props or similar.

12. Once the bulk excavation is down to approximately 500mm above the proposed basement level, a second level of horizontal props will be installed, if required by design.

13. Excavation will then be carried out down to formation level.

14. The below – slab drainage for foul & ground water, sumps and pumps will then be installed.

The pumps will discharge the foul / ground water into the existing sewer system to the front of the property.

15. The new basement RC slab (ground – bearing slab) will then be constructed.

16. Once the new basement slab has gained sufficient strength, the horizontal propping across the site will be removed.

17. After the new basement slab has cured, a drained – cavity layer will be laid to the slab and walls.

18. A layer of insulation will be placed on top of the drained – cavity layer on the slab, and in front of the drained – cavity layer on the walls.

19. Finally a layer of screed will be laid to form the finished basement floor.

POTENTIAL IMPACT ON 51 Ferme Park Road AND ADJOINING PROPERTIES

The proposed basement under the existing property will be formed using an underpinning method, constructed in sections each no wider than 1000mm, with no adjacent underpins constructed within a 48 hour period. This method of construction reduces the amount of potential ground movement and so minimises the effects of settlement of the adjacent structures.

Expected settlement is zero provided an experienced contractor is appointed who undertakes the works using good practice in accordance with the structural design and follows all agreed method statements, installing all necessary temporary vertical and lateral supports required. In practice some settlement is possible but this should be no worse than 'aesthetic', according to the BRE's definition. If these conditions are met, any settlement that occurs is likely to be minimal and is likely to be accommodated in the elasticity of the superstructure. This has been borne out in the vast majority of past projects on similar properties.

The design and construction methodology, as described above, deals with the potential risks and ensures that the excavation and construction of the proposed basement will not affect the structural integrity of the property and adjoining properties.

The site is located on ground that is relatively flat and so slope instability can only be initiated in the temporary condition as the basement is being built. This would be via a collapse of the partially formed underpinning. This is highly unlikely due to the construction sequence and implementation of temporary works and is covered by the statement above on the impact on adjoining properties.

POTENTIAL IMPACT ON EXISTING AND SURROUNDING UTILITIES, INFRASTRUCTURE AND MAN – MADE CAVITIES

Any local services on the property's land will be maintained during construction and re – routed if necessary. The exact location of these services will not be known until the works commence. However, the impact will be negligible as these services will be maintained. If it is necessary to relocate or divert any utilities, the Contractor and Design Team will be under a statutory obligation to notify the utility owner prior to any works. This will be so that they can assess the impact of the works and grant or refuse their approval. There are no known man – made cavities (e.g. tunnels) in the vicinity of the proposed basement.

POTENTIAL IMPACT ON DRAINAGE, SEWAGE, SURFACE AND GROUND WATER LEVELS AND FLOWS

All existing drainage and sewage connections will be maintained throughout the construction works so there will be no impact on these existing systems.

Surface water drainage for the external areas affected by the proposed subterranean development would be collected in a new soak – away which will be constructed as part of the site works. The soak – away design will be appropriate for the existing ground conditions and in accordance with BRE Digest 365/SUDS design guidance.

A sustainable, environmentally friendly and responsible approach will be taken in the design of the surface water drainage for the development, which will also be in accordance with the SUDS framework directive.

Borehole logs from the British Geological Survey (attached) confirm that the new basement will be formed above the ground water level, thus there will be no impact on ground water flows and levels.

Mostafa Alakrash BSc. Civil Engineering
Director

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Bore hole records in the area:

GROUND EXPLORATIONS LTD.

BOREHOLE NO. 4

TQ 38 NW 375

Contract Name: The Highlands

Report No. 8170/SJB/IAB

Client: London Borough of Islington,

Address: Architectural Department, Gloucester House, Margery St., London, WC1X 0JL

Site Address: The Highlands, Crouch Hill, London, N.4.

Standing Water Level: -

Method of Boring: Clay cutter

Water Struck: None

Diameter: 150 mm

Ground Level: 62·18 m

Start: 24.10.79. Finish: 24.10.79.

Perforated Casing: -

Remarks

JARS m	CORES m	BULK m
2478 0·6	2479 1·1	
2480 2·1	2481 2·6	
2482 3·7	2483 4·1	
2484 5·2	2485 5·6	
2486 6·7	2487 7·2	
2480 8·2	2490 8·7	
	2491 9·6	

Description	Thickness m	Depth m
MADE GROUND (clay, topsoil, etc.)	0·9	0·9
Firm brown mottled CLAY	4·0	4·9
Firm/stiff dark brown fissured CLAY with crystals	2·1	7·0
Stiff blue fissured CLAY	3·0	10·0
TOTALS	10·0	10·0

Notes 1. Descriptions are in accordance with B.S. Code of Practice C.P. 2001
Clients are requested to compare with samples submitted.
2. Core samples are nominally 102mm diameter and 460mm long.

GROUND EXPLORATIONS LTD.

BOREHOLE NO. 3 TQ 38 NW 374

Contract Name: The Highlands
Report No. 8170/SJB/IAB

Client: London Borough of Islington,
Site Address

Address: Architectural Department,
Gloucester House, Margery St.,
London WC1X 0JL

The Highlands,
Crouch Hill,
London, N.4.

Standing Water Level: -
Method of Boring: Claycutter

Water Struck: None
Diameter: 150 mm

Ground Level: 60•98 m
Start: 23.10.79. Finish: 24.10.79.

Perforated Casing: -

Remarks

JARS m		CORES m		BULK m
2458 0•6	2472 11•3	2459 1•1	2473 11•7	
2460 2•1	2474 12•8	2461 2•4	2475 13•3	
2462 3•7	2476 14•0	2463 4•1	2477 14•5	
2464 5•2		2465 5•6		
2466 6•7		2467 7•2		
2468 8•2		2469 8•7		
2470 9•8		2471 10•2		

Description	Thickness m	Depth m
MADE GROUND (clay, brick, tins, gravel, etc.)	1•8	1•8
Firm/stiff brown mottled CLAY	3•1	4•9
Stiff dark brown fissured CLAY with crystals	3•0	7•9
Stiff/very stiff blue fissured CLAY	7•0	14•9
TOTALS	14•9	14•9

Notes
1. Descriptions are in accordance with B.S. Code of Practice C.P. 2001
Clients are requested to compare with samples submitted.
2. Core samples are nominally 102mm diameter and 460mm long.
Depths shown are to top of sample.

PR11779

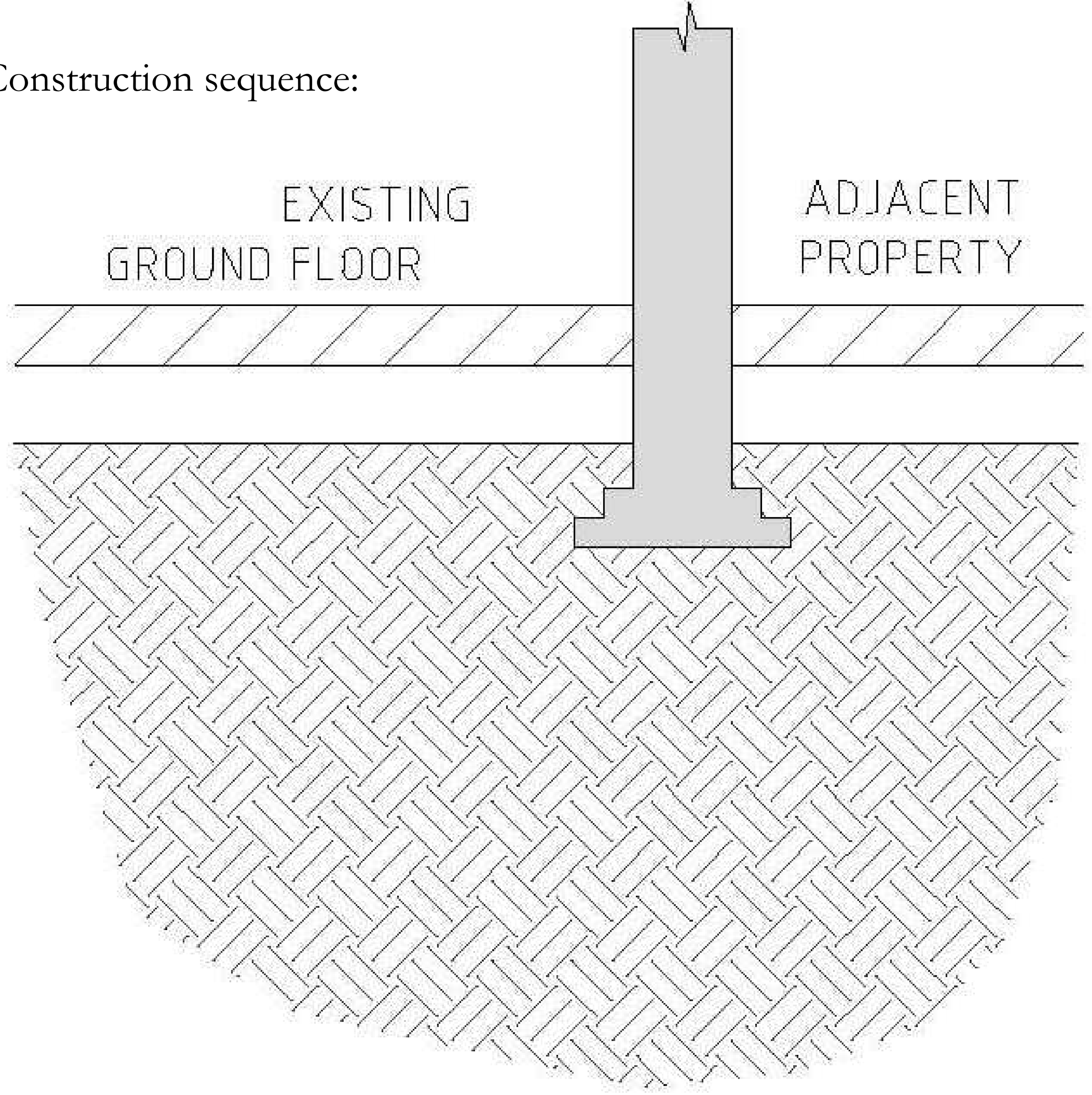
Appendix:

General Underpinning Specification

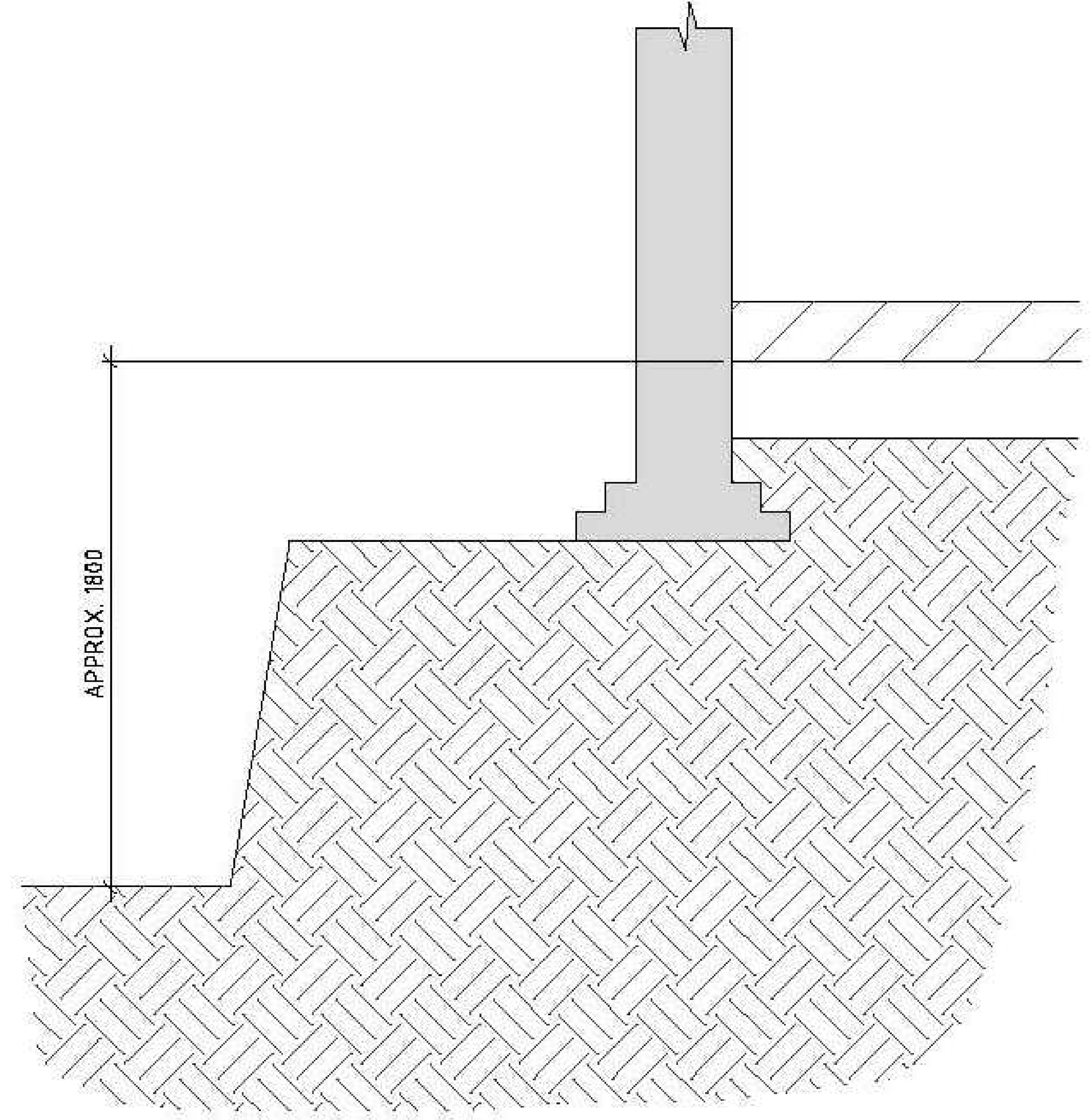
1. The walls to the perimeter of the new basement shall be underpinned in reinforced concrete. The underpins shall take the vertical loads from the walls and horizontal loads from the earth.
2. Underpinning bases shall be excavated in short sections not exceeding 1000mm in width.
3. The sequence of the underpinning shall be such that any given underpin will be completed, drypacked and a minimum period of 48 hours lapsed before an adjacent excavation commenced to form another underpin.
4. In the event that the existing foundations to the wall are found to be unstable, sacrificial steel jacks shall be installed underneath the foundation to prop the bottom few courses of bricks. These steel jacks shall be left in place and shall be incorporated into the concrete stem.
5. In the event that the ground is unstable, lateral propping shall be provided as required to the rear of the excavation and to the sides of the excavated working trench. The front and side faces of the excavation shall be propped using trench sheeting or plywood, timber boards and acrow props as appropriate. Sacrificial back – shutters shall be used to the rear face of the excavation (i.e. underneath the wall) if required. Cementitious grout will be poured behind the back – shutters to fill up the voids behind the back – shutters.
6. Excavation for an underpin section shall be dug in a day, and the concrete to the base shall be poured by the end of the same day.
7. The concrete to the stem of the underpin shall be poured the following day. This shall be poured up to within 50 – 75mm of the underside of the existing wall foundations.
8. On the following day, the gap between the concrete and the underside of the existing foundation shall be drypacked with C35 concrete using 5 – 10mm coarse aggregate and "Combex 100" expanding admixture by Fosroc UK Ltd in accordance with their instructions.
9. Once the drypack has gained sufficient strength, any protrusions of the footings into our site shall be carefully trimmed back using hand tools to avoid causing any damage to the foundation. The protrusions shall be trimmed back to be flush in-line with the face of the wall above.
10. A minimum of 48 hours shall be allowed before adjacent sections are excavated to form a new underpin.
11. Adjacent underpins shall be connected using T12 dowel bars 600mm long, 300mm embedment each side, at 300mm vertical centres.
12. Concrete cover to reinforcement shall be 35mm for cast against shutter or the top surface of the basement slab, 50mm for cast against blinding and 75mm for cast against earth.

13. Grade of concrete shall be C35 with minimum cement content 300kg/m3, maximum free water to cement ratio 0.60, slump 100mm.

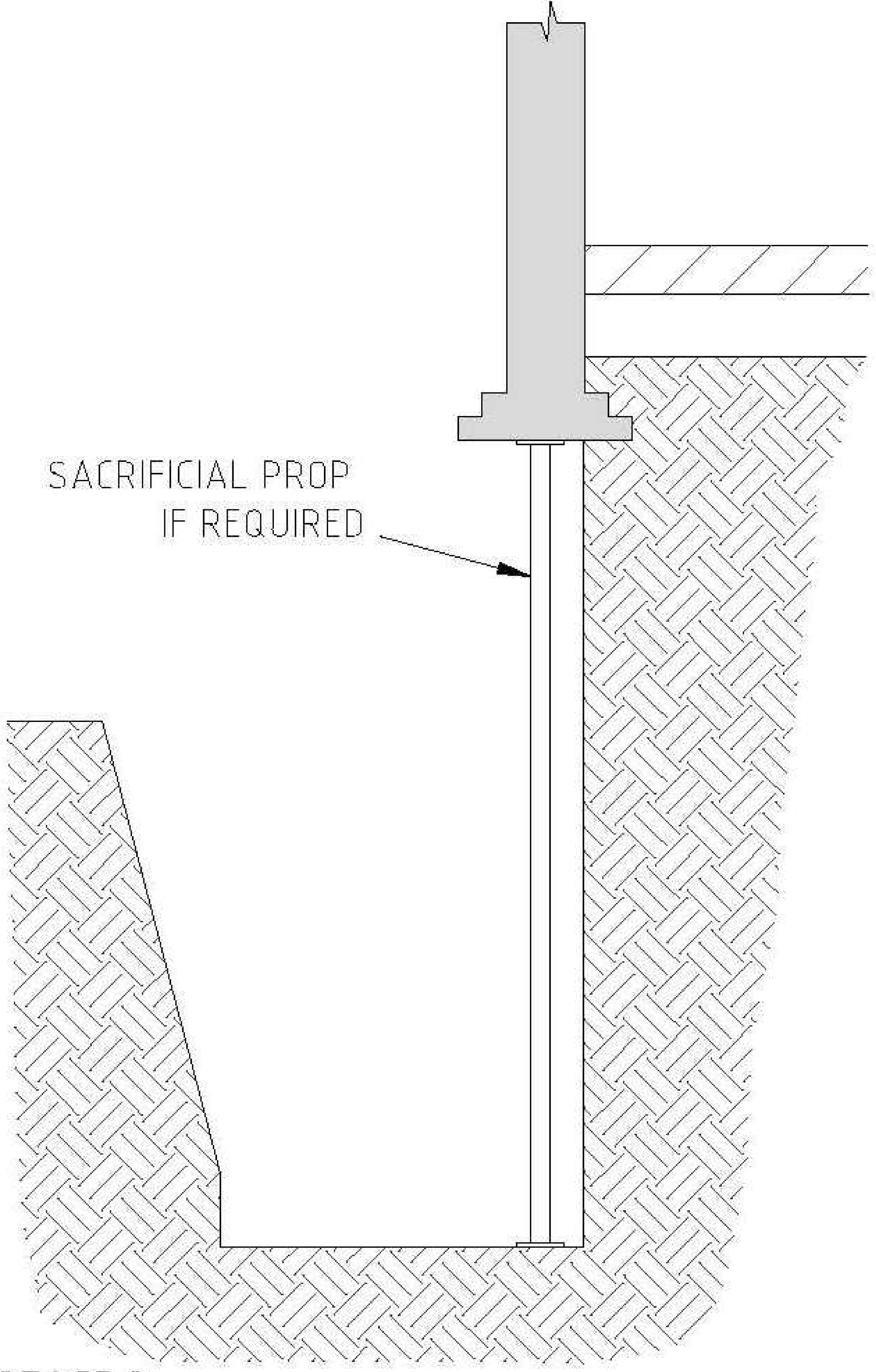
Construction sequence:



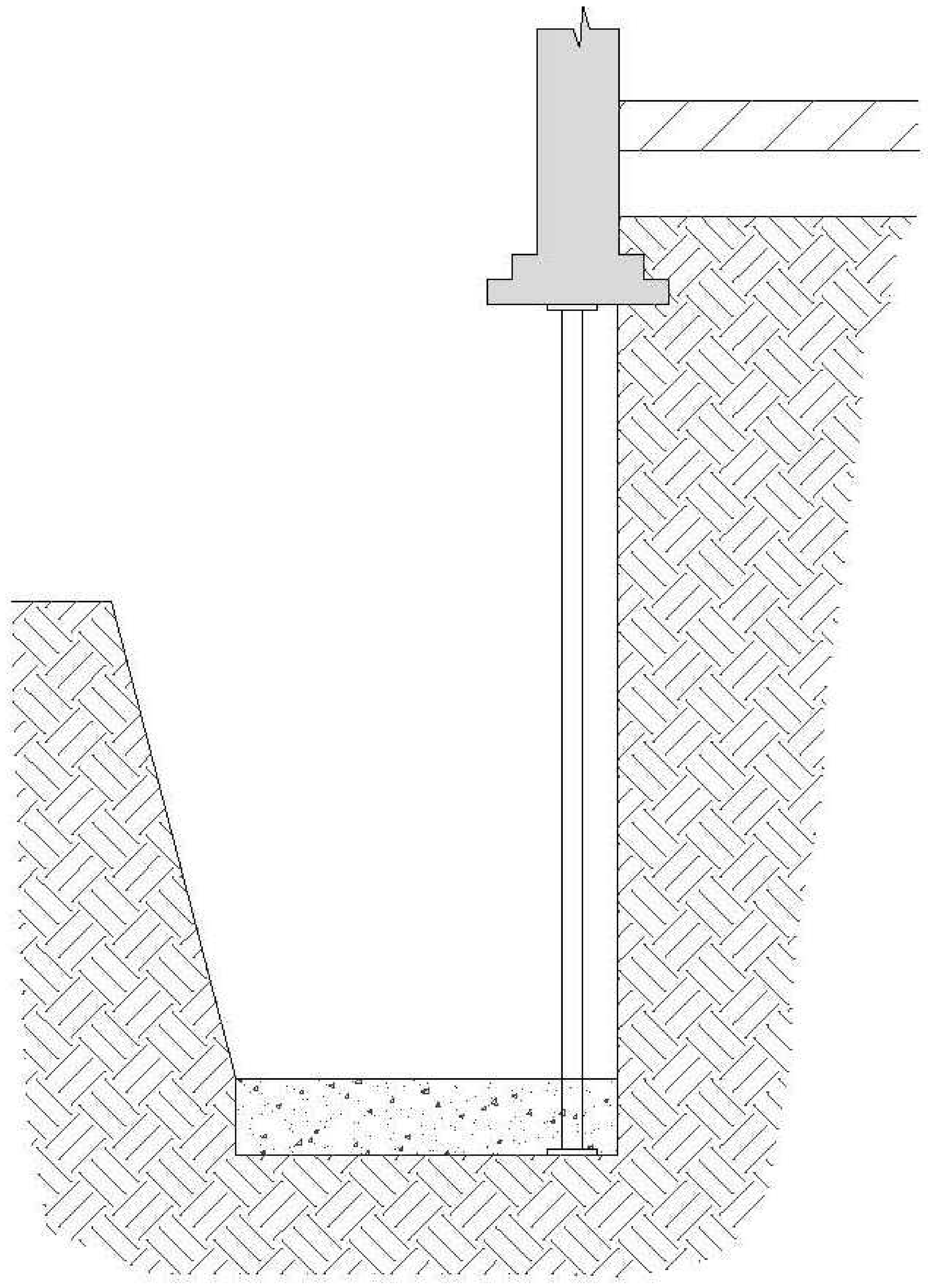
STAGE 0
EXISTING CONDITION



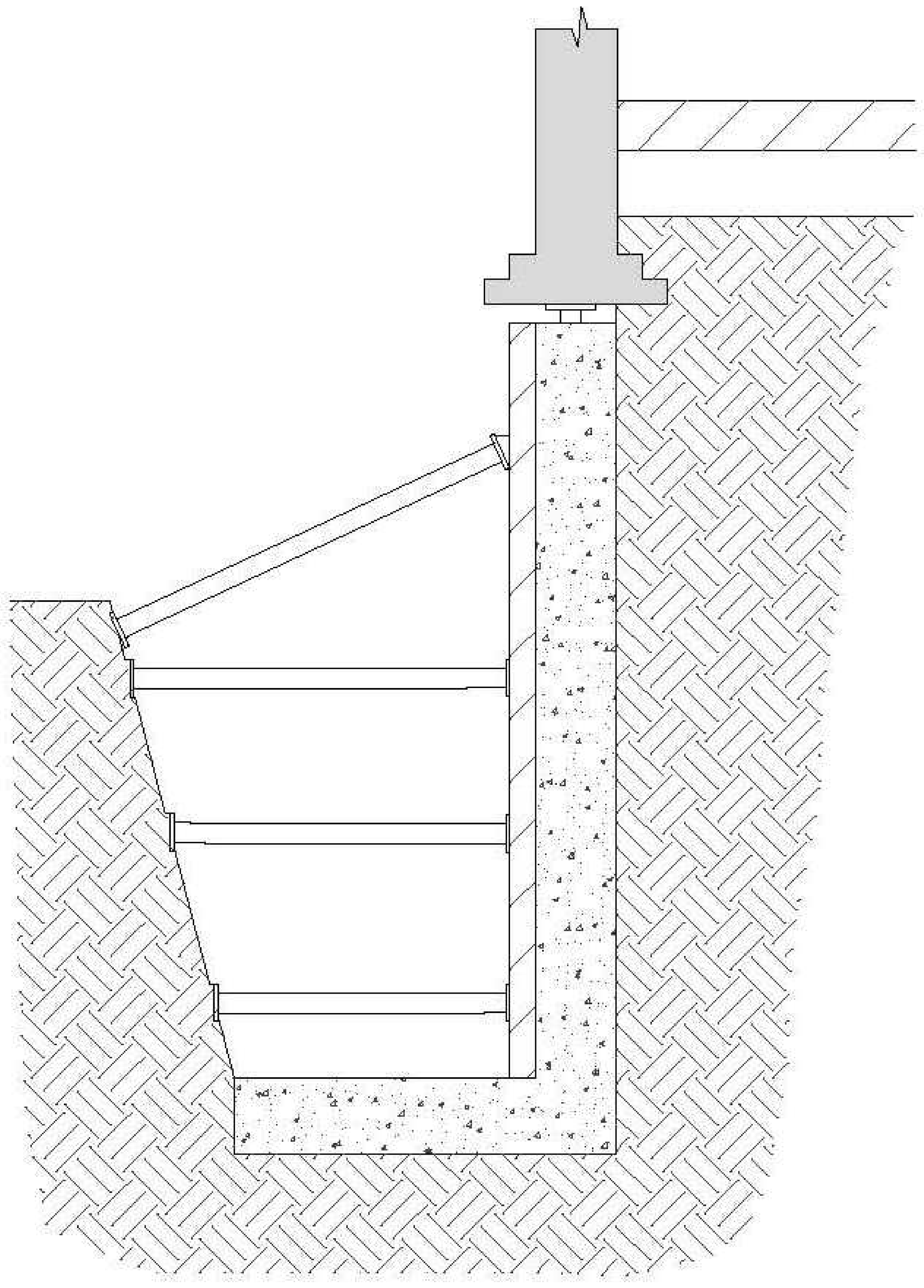
STAGE 1
GENERAL LEVEL REDUCTION



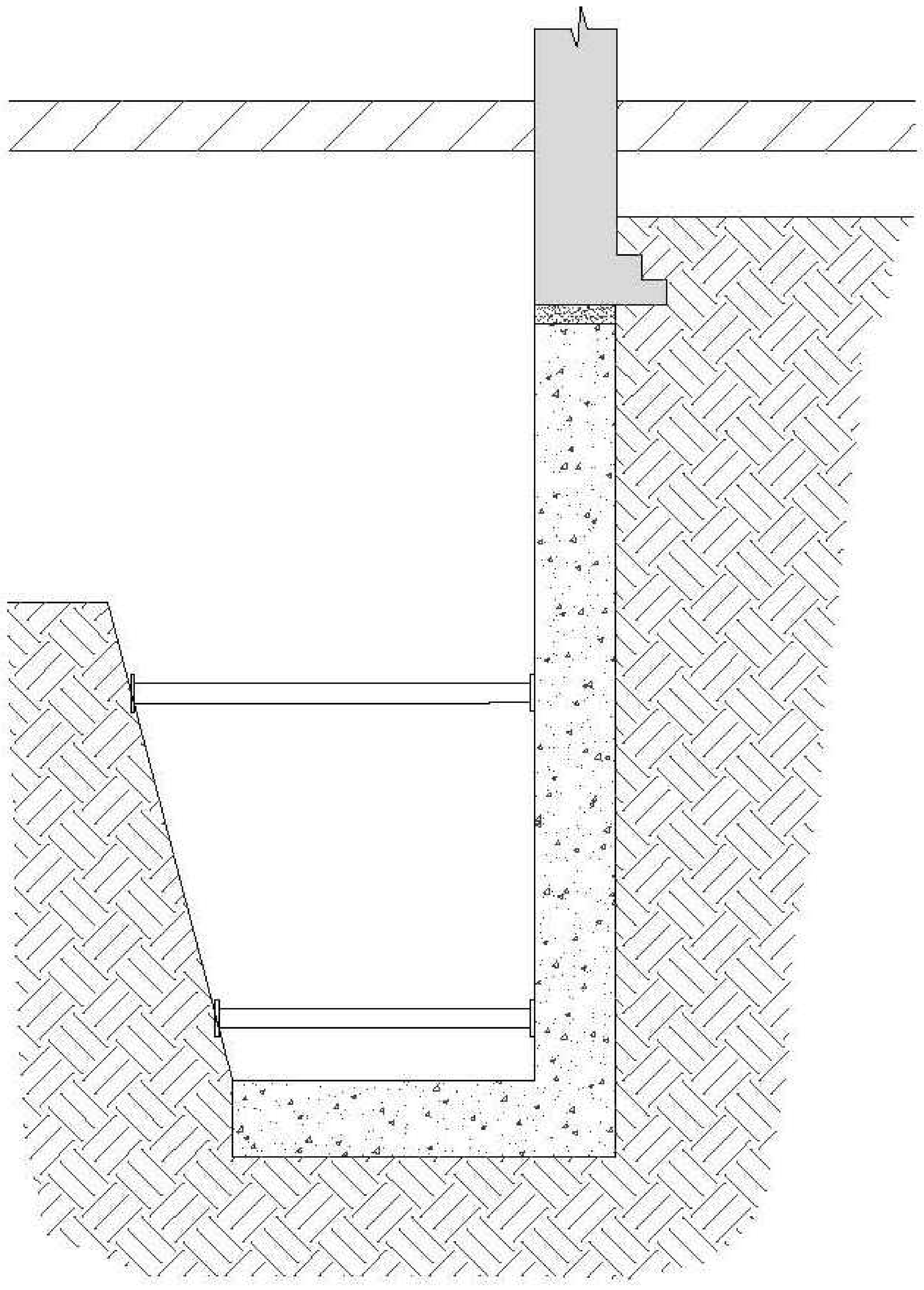
STAGE 2
EXCAVATE TO FORM UNDERPIN



STAGE 3
CONCRETE BASE OF UNDERPIN

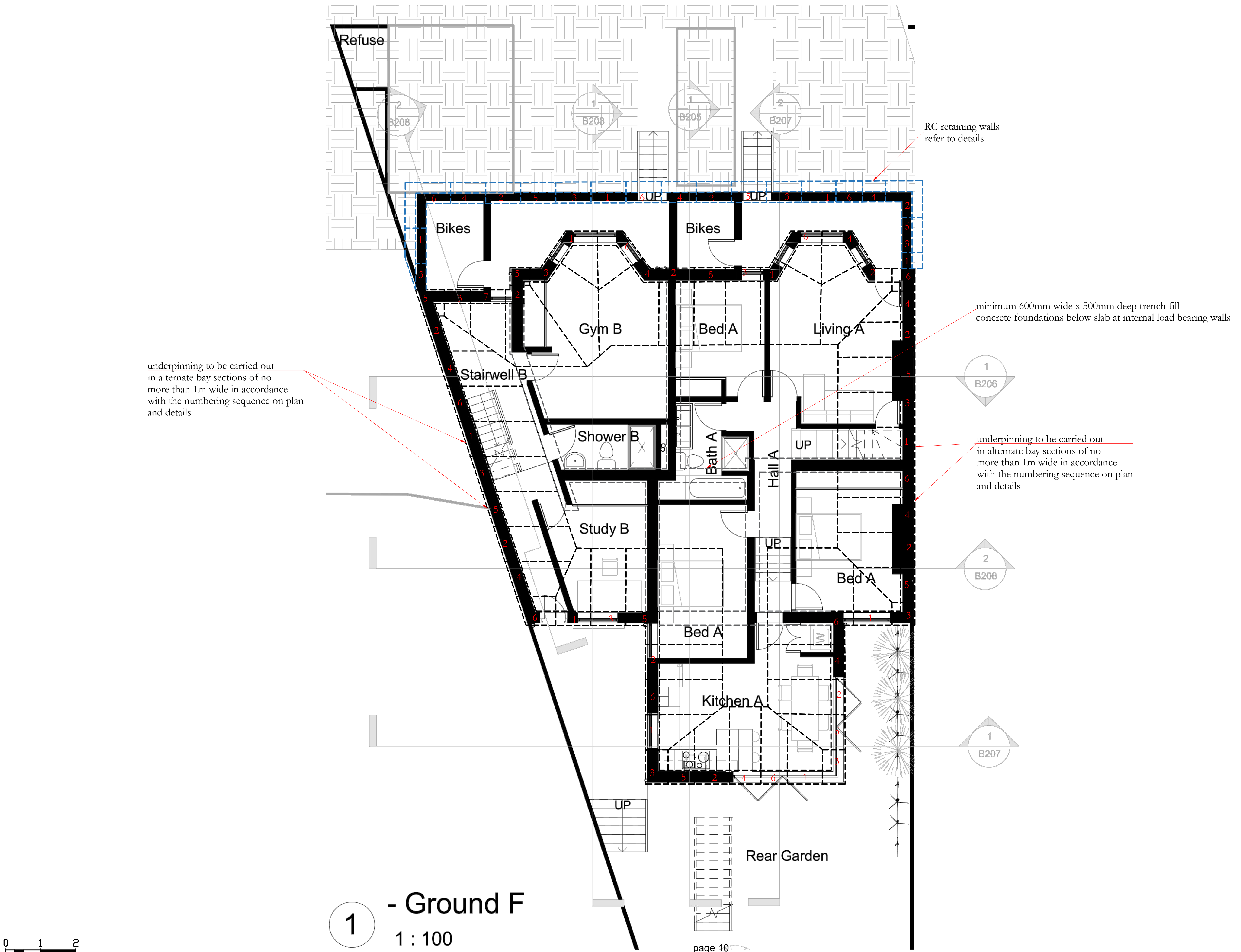


STAGE 4
CONCRETE STEM OF UNDERPIN



STAGE 5
STRIKE SHUTTER WHEN CONCRETE HAS
GAINED SUFFICIENT STRENGTH, DRYPACK,
TRIM – OFF PROJECTING FOOTING, RE-PROP
UNTIL BASEMENT SLAB IS CAST.

Notes:
-To be confirmed by Building Control before work commencing
-All dimensions to be confirmed on site
-Floor and roof joists to be strap tied to walls via steel straps at 600cc
-All floor and roof joists to have noggings at 600cc for lateral support.
-Joists under stud walls, around stairs and around Velux to be doubled.
-Drawing to be read in conjunction with Architects drawings.



1 - Ground F

1 : 100

Showing foundations below

A&S Design
8, 90 Akerman Road London. SW9 6SN
info@as-d.co.uk | m:07588693045

Project:
53 Ferme Park Road
N4 4EB

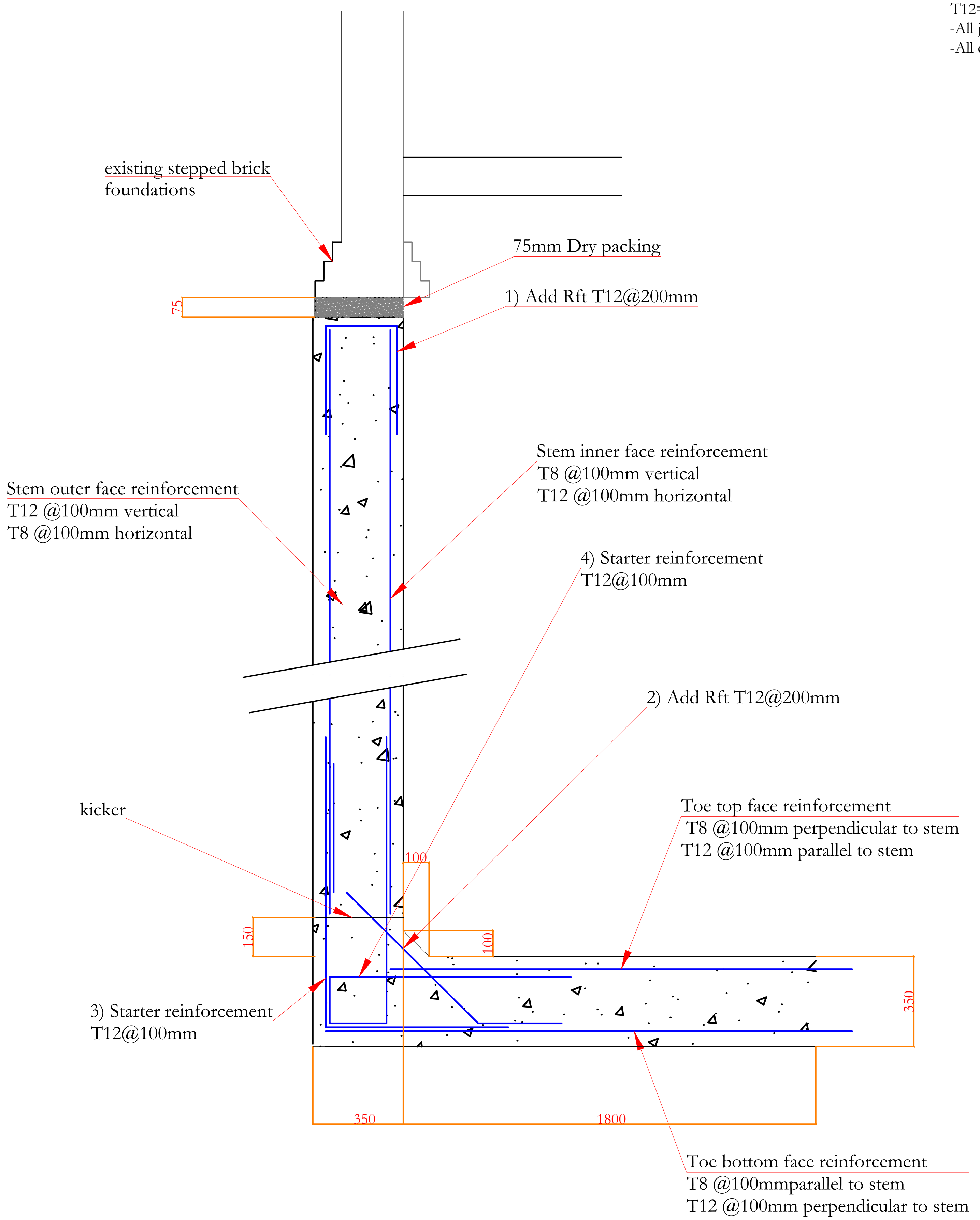
Title:
Structural plans
REV: ()

Scale:
1:100@a3p

DN: 217-FR-01

Underpinning details

Notes:
-All structural drawings to be read in conjunction with Architects drawings and specification.
-Concrete grade C40 (40 N/mm2).
-Minimum cover to be 50mm.
-Reinforcement steel grade T (460 N/mm2)
-Minimum laps to reinforcement to be:
T12=600mm
-Minimum anchorage to be
T12=840mm
-All joints horizontal or vertical to have water stops as per manufacturers guidance.
-All dimensions to be confirmed on site by contractor prior to work commencing.



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Project:
53 Ferme Park Road
N4 4EB

Title:
Details
REV: ()

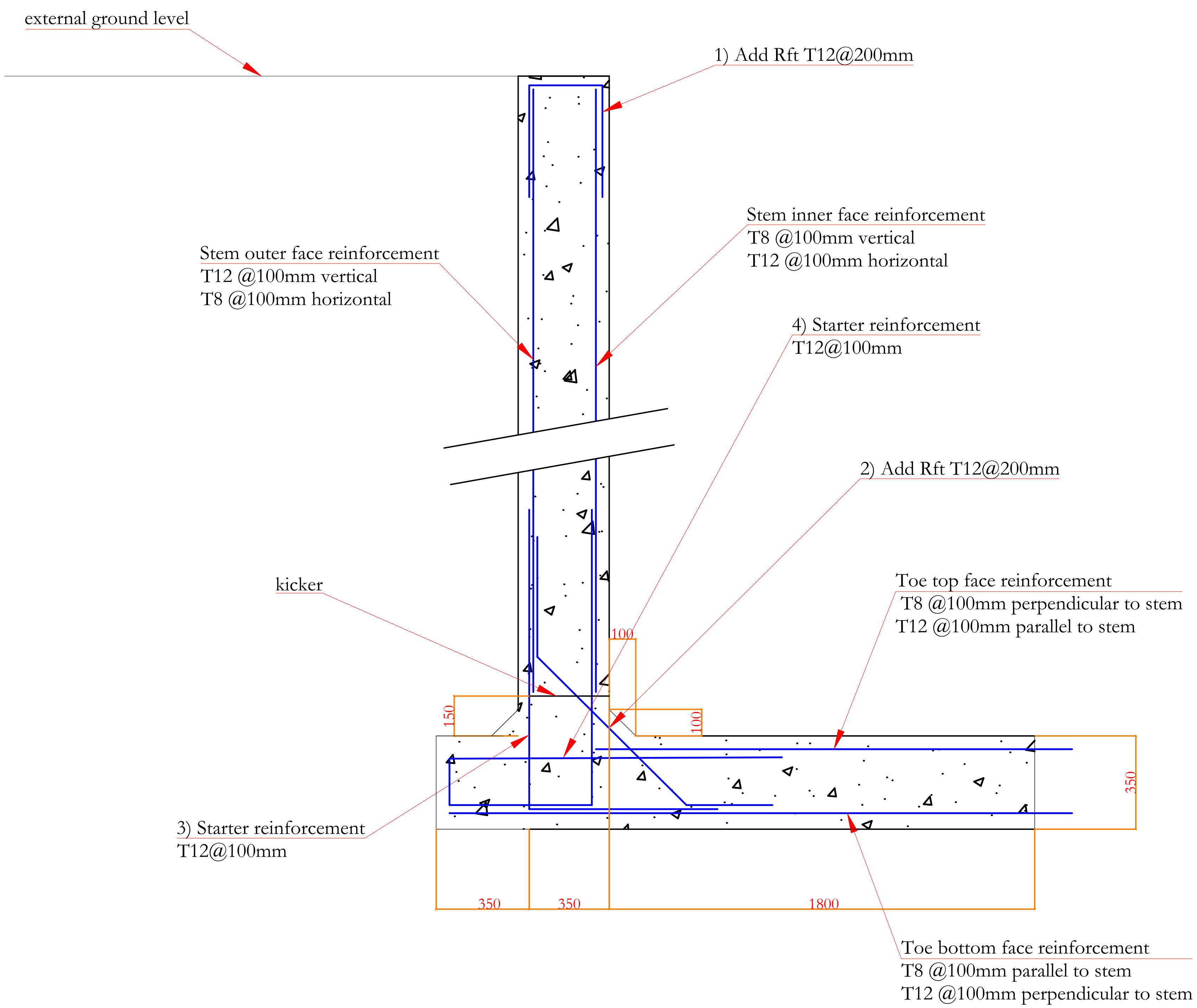
Scale:
1:20@a3p

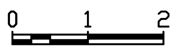
DN: 217-FR-02

0 1 2

RC retaining wall details

Notes:
-All structural drawings to be read in conjunction with Architects drawings and specification.
-Concrete grade C40 (40 N/mm2).
-Minimum cover to be 50mm.
-Reinforcement steel grade T (460 N/mm2)
-Minimum laps to reinforcement to be:
T12=600mm
-Minimum anchorage to be
T12=840mm
-All joints horizontal or vertical to have water stops as per manufacturers guidance.
-All dimensions to be confirmed on site by contractor prior to work commencing.





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info@as-d.co.uk | m:07588693045

Project:
53 Ferme Park Road
N4 4EB

Title:
Details
REV: ()

Scale:
1:20@a3p

DN: 217-FR-03

Structural calculations:

<u>project: 53 Ferme Park Road</u>

loads on foundations:

worst case party wall side

<u>i- LOADS</u>

a) Uniform Loads

1- wall		4.5	kn/m2
	x	10.65	m
	=	**47.93**	**kn/m**
2- floors above (dead+live)		2	kn/m2
	x	4.70	m
	x	3	floors
	=	**28.20**	**kn/m**
4- roof (dead+live)		1.85	kn/m2
	x	4.70	m
	=	**8.70**	**kn/m**

<u>TOTAL U.L=</u> <u>84.82</u> **Kn/m**

UNDERPINNING DESIGN CHECK LAOD CASE 1 (NO GROUND WATER):

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **2250** mm
Stem thickness;	t_{stem} = **350** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1800** mm
Base thickness;	t_{base} = **350** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **1900** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **350** mm
Depth of excavation;	d_{exc} = **350** mm

Retained soil properties

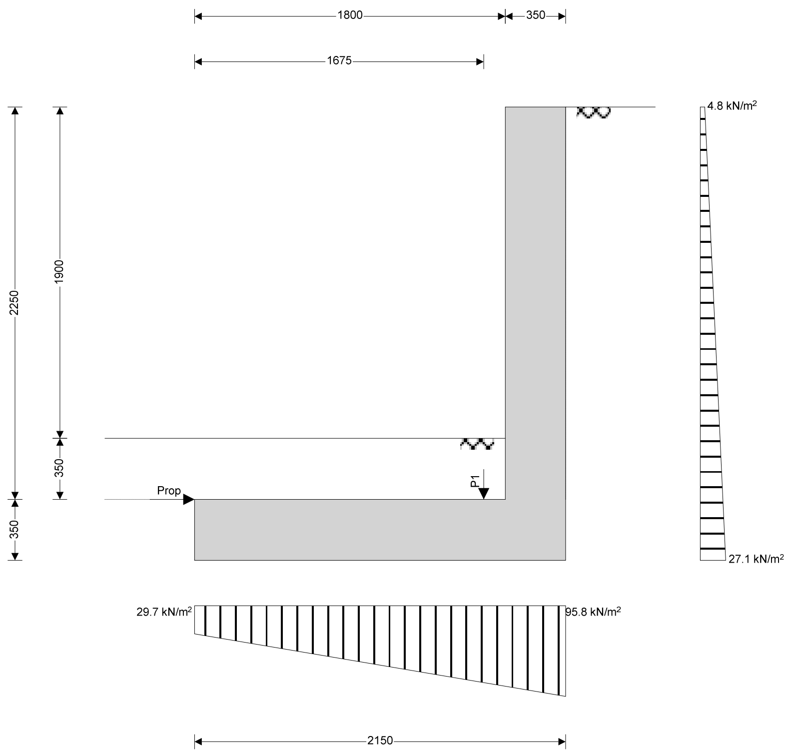
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1675 mm;	P_{G1} = **85** kN/m



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} = \mathbf{2150}$ mm
Moist soil height;	$h_{moist} = h_{soil} = \mathbf{2250}$ mm
Length of surcharge load;	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2150}$ mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2600}$ mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1300}$ mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.788}$ m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1975}$ mm
Area of wall base;	$A_{base} = l_{base} \times t_{base} = \mathbf{0.753}$ m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2 = \mathbf{1075}$ mm
Area of base soil;	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{0.63}$ m^2
- Distance to vertical component;	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{900}$ mm
- Distance to horizontal component;	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{233}$ mm

Using Coulomb theory

Active pressure coefficient;	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[}\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2) = \mathbf{0.483}$
Passive pressure coefficient;	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[}\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem}$ = **19.7** kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base}$ = **18.8** kN/m

Line loads; $F_{P\_v} = P_{G1}$ = **85** kN/m

Base soil; $F_{pass\_v} = A_{pass} \times \gamma_b'$ = **11.3** kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{pass\_v} + F_{P\_v}$ = **134.8** kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff}$ = **12.4** kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times h_{eff}^2 / 2$ = **29** kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2$ = **-10.3** kN/m

Total; $F_{total\_h} = F_{moist\_h} + F_{pass\_h} + F_{sur\_h}$ = **31.2** kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem}$ = **38.9** kNm/m

Wall base; $M_{base} = F_{base} \times x_{base}$ = **20.2** kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = **-16.1** kNm/m

Line loads; $M_P = P_{G1} \times p_1$ = **142.4** kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h}$ = **-25.2** kNm/m

Base soil; $M_{pass} = F_{pass\_v} \times x_{pass\_v}$ = **10.2** kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur} + M_P$ = **170.4** kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h}$ = **31.2** kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v}$ = **1264** mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2$ = **189** mm

Loaded length of base; $l_{load} = l_{base}$ = **2150** mm

Bearing pressure at toe; $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **29.7** kN/m^2

Bearing pressure at heel; $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **95.8** kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.044**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.6.08

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class; C35/45

Characteristic compressive cylinder strength; f_{ck} = **35** N/mm^2

Characteristic compressive cube strength; $f_{ck,cube}$ = **45** N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = **43** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **3.2** N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.2** N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **34077** N/mm^2

Partial factor for concrete - Table 2.1N; γ_C = **1.50**

Compressive strength coefficient - cl.3.1.6(1); α_{cc} = **0.85**

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **19.8** N/mm^2

Maximum aggregate size; h_{agg} = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement; f_{yk} = **500** N/mm^2

Modulus of elasticity of reinforcement; E_s = **200000** N/mm^2

Partial factor for reinforcing steel - Table 2.1N; γ_S = **1.15**

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

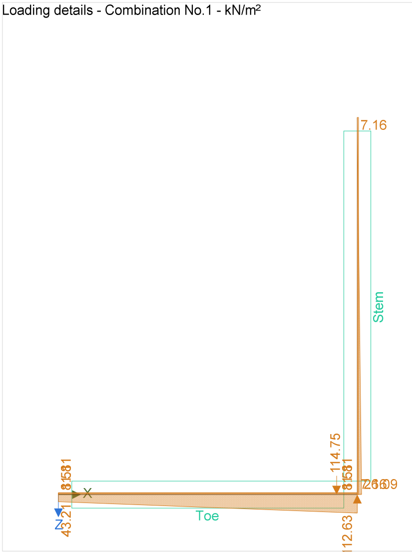
Cover to reinforcement

Front face of stem; c_{sf} = **50** mm

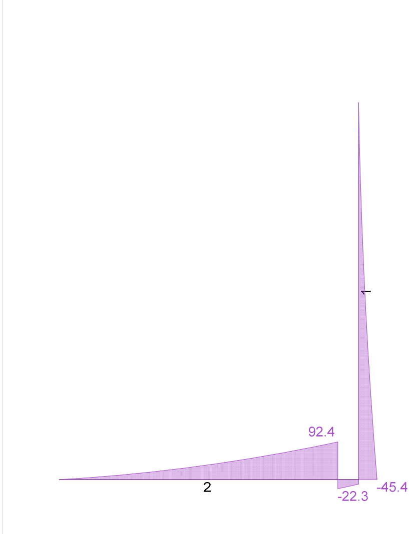
Rear face of stem; c_{sr} = **50** mm

Top face of base; c_{bt} = **50** mm

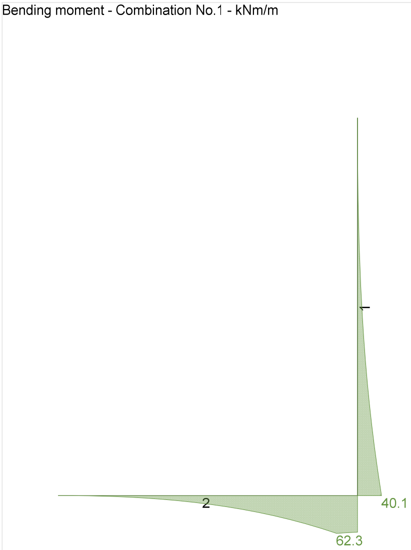
Bottom face of base; c_{bb} = **50** mm



Loading details - Combination No.1 - kN/m²



Shear force - Combination No.1 - kN/m



Bending moment - Combination No.1 - kNm/m

Check stem design at base of stem

Depth of section;	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1;	M = **40.1** kNm/m
Depth to tension reinforcement;	$d = h - c_{sr} - \phi_{sr} / 2 =$ **295** mm
	$K = M / (d^2 \times f_{ck}) =$ **0.013**
	K' = **0.207**
	K' > K - No compression reinforcement is required
Lever arm;	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d =$ **280** mm
Depth of neutral axis;	$x = 2.5 \times (d – z) =$ **37** mm
Area of tension reinforcement required;	$A_{sr.req} = M / (f_{yd} \times z) =$ **329** mm²/m
Tension reinforcement provided;	10 dia.bars @ 100 c/c
Area of tension reinforcement provided;	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) =$ **785** mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d =$ **492** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h =$ **14000** mm^2/m

$max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} =$ **0.627**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) / 1000 =$ **0.006**

Required tension reinforcement ratio; $\rho = A_{sr.req} / d =$ **0.001**

Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 =$ **0.000**

Structural system factor - Table 7.4N; $K_b =$ **0.4**

Reinforcement factor - exp.7.17; $K_s = min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) =$ **1.5**

Limiting span to depth ratio - exp.7.16.a; $K_s \times K_b \times [11 + 1.5 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times \rho_0 / \rho + 3.2 \times \sqrt{}(f_{ck} / 1 \text{ N/mm}^2) \times (\rho_0 / \rho - 1)^{3/2}] =$ **136.1**

Actual span to depth ratio; $h_{stem} / d =$ **7.6**

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} =$ **0.3** mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 =$ **0.6**

Serviceability bending moment; $M_{sls} =$ **23.5** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z) =$ **107** N/mm^2

Load duration; Long term

Load duration factor; $k_t =$ **0.4**

Effective area of concrete in tension; $A_{c.eff} = min(2.5 \times (h - d), (h – x) / 3, h / 2) =$ **104375** mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} =$ **3.2** N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} =$ **0.008**

Modular ratio; $\alpha_e = E_s / E_{cm} =$ **5.869**

Bond property coefficient; $k_1 =$ **0.8**

Strain distribution coefficient; $k_2 =$ **0.5**

$k_3 =$ **3.4**

$k_4 =$ **0.425**

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} =$ **396** mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k =$ **0.127** mm

$w_k / w_{max} =$ **0.424**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **45.4** kN/m

$C_{Rd,c} = 0.18 / \gamma_C =$ **0.120**

$k = min(1 + \sqrt{}(200 \text{ mm} / d), 2) =$ **1.823**

Longitudinal reinforcement ratio; $\rho_l = min(A_{sr.prov} / d, 0.02) =$ **0.003**

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} =$ **0.510** N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} =$ **150.4** kN/m

$V / V_{Rd.c} =$ **0.302**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1);	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **350** mm²/m
Maximum spacing of reinforcement – cl.9.6.3(2);	s_{sx_max} = **400** mm
Transverse reinforcement provided;	10 dia.bars @ 100 c/c
Area of transverse reinforcement provided;	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **785** mm²/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section;	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1;	M = **62.3** kNm/m
Depth to tension reinforcement;	$d = h - c_{bb} - \phi_{bb} / 2$ = **295** mm
	$K = M / (d^2 \times f_{ck})$ = **0.020**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm;	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **280** mm
Depth of neutral axis;	$x = 2.5 \times (d – z)$ = **37** mm
Area of tension reinforcement required;	$A_{bb.req} = M / (f_{yd} \times z)$ = **511** mm²/m
Tension reinforcement provided;	10 dia.bars @ 100 c/c
Area of tension reinforcement provided;	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **785** mm²/m
Minimum area of reinforcement - exp.9.1N;	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **492** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3);	$A_{bb.max} = 0.04 \times h$ = **14000** mm²/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.651**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Limiting crack width;	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1;	ψ_2 = **0.6**
Serviceability bending moment;	M_{sls} = **44.6** kNm/m
Tensile stress in reinforcement;	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **202.6** N/mm²
Load duration;	Long term
Load duration factor;	k_t = **0.4**
Effective area of concrete in tension;	$A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2)$ = **104375** mm²/m
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm}$ = **3.2** N/mm²
Reinforcement ratio;	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.008**
Modular ratio;	$\alpha_e = E_s / E_{cm}$ = **5.869**
Bond property coefficient;	k_1 = **0.8**
Strain distribution coefficient;	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11;	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **396** mm
Maximum crack width - exp.7.8;	$w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.241** mm
	w_k / w_{max} = **0.802**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V = \mathbf{92.4}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.823}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.003}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}$ N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{150.4}$ kN/m

$V / V_{Rd.c} = \mathbf{0.615}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

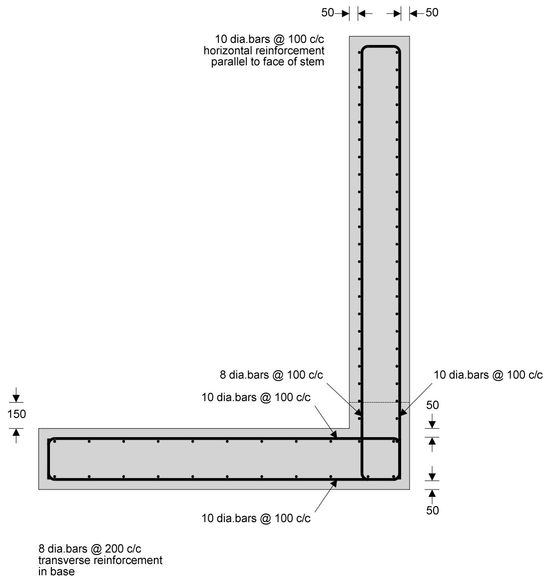
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{157}$ mm^2/m

Maximum spacing of reinforcement – cl.9.3.1.1(3); $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided; 8 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{251}$ mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

UNDERPINNING DESIGN CHECK LAOD CASE 2 (GROUND WATER PRESENT):

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **2250** mm
Stem thickness;	t_{stem} = **350** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1800** mm
Base thickness;	t_{base} = **350** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **1900** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **350** mm
Depth of excavation;	d_{exc} = **350** mm
Height of water;	h_{water} = **1900** mm
Water density;	γ_w = **9.8** kN/m^3

Retained soil properties

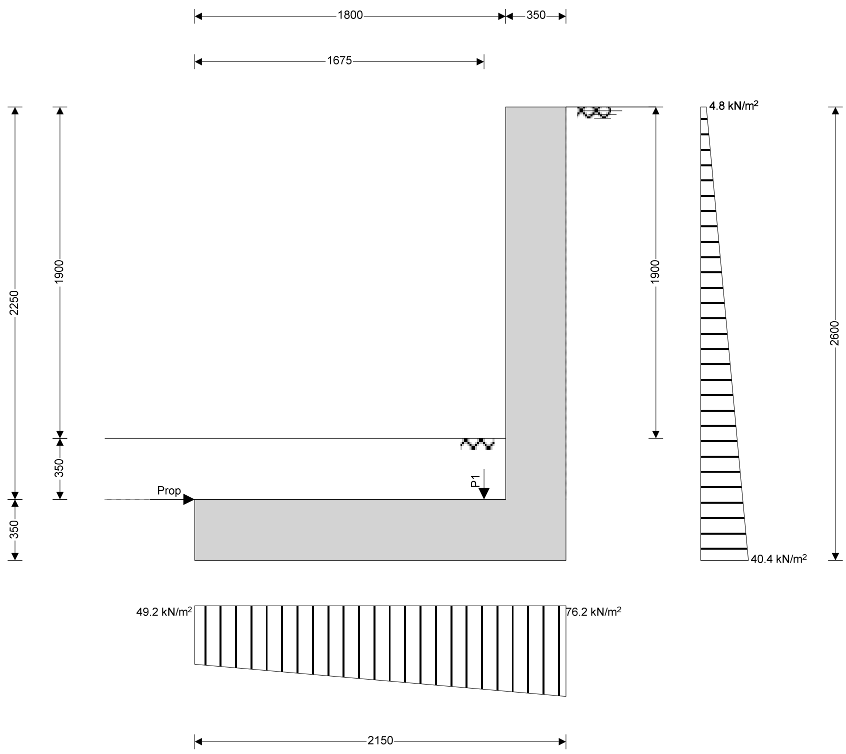
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2
Vertical line load at 1675 mm;	P_{G1} = **85** kN/m



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem}$ = **2150** mm
Saturated soil height;	$h_{sat} = h_{water} + d_{cover}$ = **2250** mm
Moist soil height;	$h_{moist} = h_{ret} - h_{water}$ = **0** mm
Length of surcharge load;	$l_{sur} = l_{heel}$ = **0** mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **2150** mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **2600** mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2$ = **1300** mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem}$ = **0.788** m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2$ = **1975** mm
Area of wall base;	$A_{base} = l_{base} \times t_{base}$ = **0.753** m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2$ = **1075** mm
Area of base soil;	$A_{pass} = d_{cover} \times l_{toe}$ = **0.63** m^2
- Distance to vertical component;	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass}$ = **900** mm
- Distance to horizontal component;	$x_{pass\_h} = (d_{cover} + h_{base}) / 3$ = **233** mm

Using Coulomb theory

Active pressure coefficient;	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[}\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]]^2)$ = **0.483**
Passive pressure coefficient;	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[}\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]]^2)$ = **2.359**

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem}$ = **19.7** kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base}$ = **18.8** kN/m

Line loads; $F_{P\_v} = P_{G1}$ = **85** kN/m

Base soil; $F_{pass\_v} = A_{pass} \times \gamma_b'$ = **11.3** kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{pass\_v} + F_{water\_v} + F_{P\_v}$ = **134.8** kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff}$ = **12.4** kN/m

Saturated retained soil; $F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2$ = **13.2** kN/m

Water; $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2$ = **33.2** kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base}))$ = **0** kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2$ = **-10.3** kN/m

Total; $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h}$ = **48.5** kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem}$ = **38.9** kNm/m

Wall base; $M_{base} = F_{base} \times x_{base}$ = **20.2** kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = **-16.1** kNm/m

Line loads; $M_P = P_{G1} \times p_1$ = **142.4** kNm/m

Saturated retained soil; $M_{sat} = -F_{sat\_h} \times x_{sat\_h}$ = **-11.4** kNm/m

Water; $M_{water} = -F_{water\_h} \times x_{water\_h}$ = **-28.7** kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h}$ = **0** kNm/m

Base soil; $M_{pass} = F_{pass\_v} \times x_{pass\_v}$ = **10.2** kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{pass} + M_{water} + M_{sur} + M_P$ = **155.4** kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h}$ = **48.5** kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v}$ = **1152** mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2$ = **77** mm

Loaded length of base; $l_{load} = l_{base}$ = **2150** mm

Bearing pressure at toe; $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base})$ = **49.2** kN/m^2

Bearing pressure at heel; $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base})$ = **76.2** kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.312**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.6.08

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

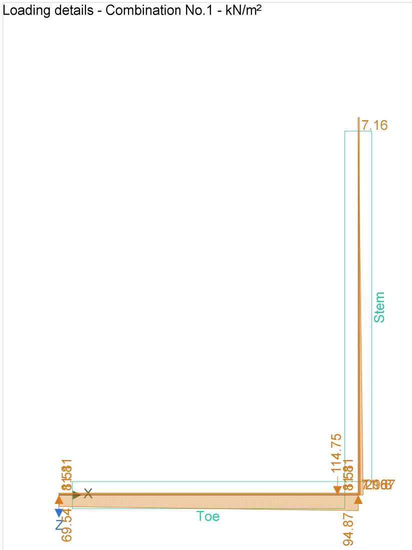
Concrete strength class;	C35/45
Characteristic compressive cylinder strength;	f_{ck} = **35** N/mm^2
Characteristic compressive cube strength;	$f_{ck,cube}$ = **45** N/mm^2
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck} + 8$ N/mm^2 = **43** N/mm^2
Mean value of axial tensile strength;	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}$ / 1 N/mm$^2)^{2/3}$ = **3.2** N/mm^2
5% fractile of axial tensile strength;	$f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.2** N/mm^2
Secant modulus of elasticity of concrete;	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm}$ / 10 N/mm$^2)^{0.3}$ = **34077** N/mm^2
Partial factor for concrete - Table 2.1N;	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1);	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15;	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **19.8** N/mm^2
Maximum aggregate size;	h_{agg} = **20** mm

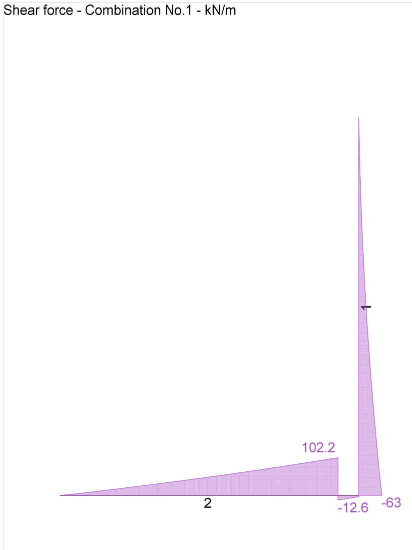
Reinforcement details

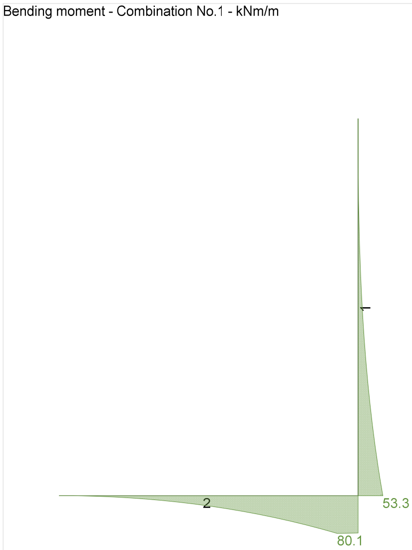
Characteristic yield strength of reinforcement;	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement;	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N;	γ_S = **1.15**
Design yield strength of reinforcement;	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Cover to reinforcement

Front face of stem;	c_{sf} = **50** mm
Rear face of stem;	c_{sr} = **50** mm
Top face of base;	c_{bt} = **50** mm
Bottom face of base;	c_{bb} = **50** mm







Check stem design at base of stem

Depth of section; $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{53.3}$ kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{294}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.018}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{279}$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{37}$ mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{439}$ mm^2/m

Tension reinforcement provided; 12 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{1131}$ mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{491}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h = \mathbf{14000}$ mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.434}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = \mathbf{0.001}$

Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N; $K_b = \mathbf{0.4}$

Reinforcement factor - exp.7.17; $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a; $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{85.7}$

Actual span to depth ratio; $h_{stem} / d = \mathbf{7.7}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = \mathbf{0.6}$

Serviceability bending moment; $M_{sls} = \mathbf{33.3}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{105.4}$ N/mm^2

Load duration; Long term

Load duration factor; $k_t = \mathbf{0.4}$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2) = \mathbf{104417}$ mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = \mathbf{3.2}$ N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.011}$

Modular ratio; $\alpha_e = E_s / E_{cm} = \mathbf{5.869}$

Bond property coefficient; $k_1 = \mathbf{0.8}$

Strain distribution coefficient; $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{358}$ mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.113}$ mm

$w_k / w_{max} = \mathbf{0.378}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V = \mathbf{63}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = \mathbf{1.825}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.004}$

$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}$ N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{153.2}$ kN/m

$V / V_{Rd.c} = \mathbf{0.411}$

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1);	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **350** mm²/m
Maximum spacing of reinforcement – cl.9.6.3(2);	s_{sx_max} = **400** mm
Transverse reinforcement provided;	12 dia.bars @ 100 c/c
Area of transverse reinforcement provided;	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **1131** mm²/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section;	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1;	M = **80.1** kNm/m
Depth to tension reinforcement;	$d = h - c_{bb} - \phi_{bb} / 2$ = **294** mm
	$K = M / (d^2 \times f_{ck})$ = **0.026**
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm;	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **279** mm
Depth of neutral axis;	$x = 2.5 \times (d - z)$ = **37** mm
Area of tension reinforcement required;	$A_{bb.req} = M / (f_{yd} \times z)$ = **659** mm²/m
Tension reinforcement provided;	12 dia.bars @ 100 c/c
Area of tension reinforcement provided;	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1131** mm²/m
Minimum area of reinforcement - exp.9.1N;	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **491** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3);	$A_{bb.max} = 0.04 \times h$ = **14000** mm²/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.583**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Limiting crack width;	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1;	ψ_2 = **0.6**
Serviceability bending moment;	M_{sls} = **57.7** kNm/m
Tensile stress in reinforcement;	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **182.8** N/mm²
Load duration;	Long term
Load duration factor;	k_t = **0.4**
Effective area of concrete in tension;	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **104417** mm²/m
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm}$ = **3.2** N/mm²
Reinforcement ratio;	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.011**
Modular ratio;	$\alpha_e = E_s / E_{cm}$ = **5.869**
Bond property coefficient;	k_1 = **0.8**
Strain distribution coefficient;	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11;	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **358** mm
Maximum crack width - exp.7.8;	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.197** mm
	w_k / w_{max} = **0.655**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V = \mathbf{102.2}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200\text{ mm} / d)}, 2) = \mathbf{1.825}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.004}$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}$ N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{153.2}$ kN/m

$V / V_{Rd.c} = \mathbf{0.667}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

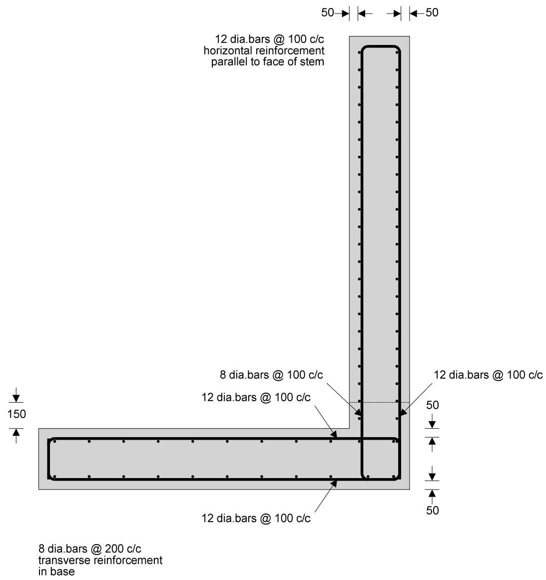
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{226}$ mm^2/m

Maximum spacing of reinforcement – cl.9.3.1.1(3); $s_{bx\_max} = \mathbf{450}$ mm

Transverse reinforcement provided; 8 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{251}$ mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

RETAINING WALL DESIGN CHECK LOAD CASE 1 (NO GROUND WATER):

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **2250** mm
Stem thickness;	t_{stem} = **350** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1800** mm
Base thickness;	t_{base} = **350** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **1900** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **350** mm
Depth of excavation;	d_{exc} = **350** mm

Retained soil properties

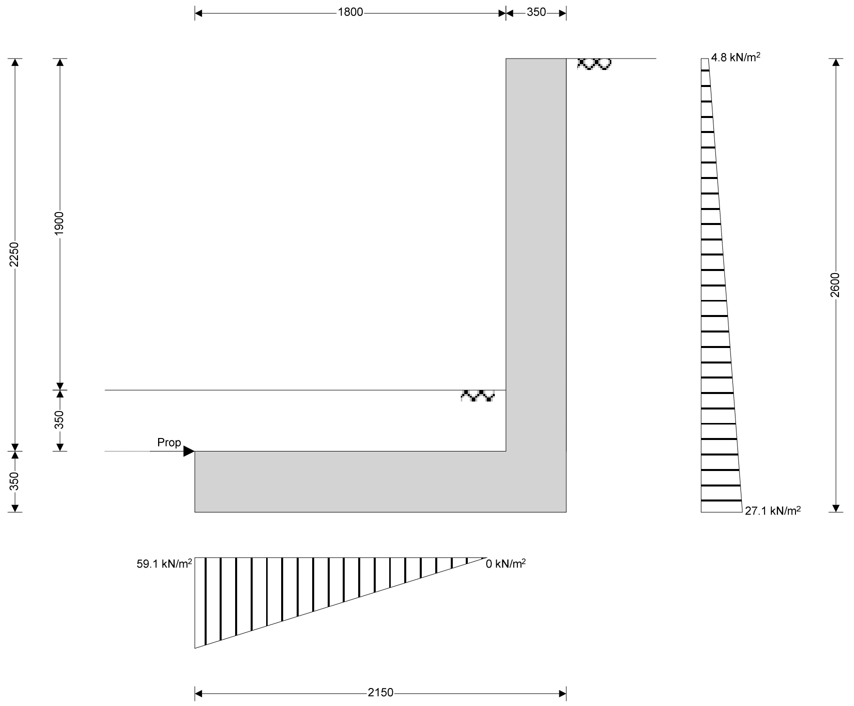
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} = \mathbf{2150}$ mm
Moist soil height;	$h_{moist} = h_{soil} = \mathbf{2250}$ mm
Length of surcharge load;	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2150}$ mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2600}$ mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1300}$ mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.788}$ m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1975}$ mm
Area of wall base;	$A_{base} = l_{base} \times t_{base} = \mathbf{0.753}$ m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2 = \mathbf{1075}$ mm
Area of base soil;	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{0.63}$ m^2
- Distance to vertical component;	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{900}$ mm
- Distance to horizontal component;	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{233}$ mm

Using Coulomb theory

Active pressure coefficient;	$K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.483}$
Passive pressure coefficient;	$K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem}$ = **19.7** kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base}$ = **18.8** kN/m

Base soil; $F_{pass\_v} = A_{pass} \times \gamma_b'$ = **11.3** kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{pass\_v}$ = **49.8** kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff}$ = **12.4** kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times h_{eff}^2 / 2$ = **29** kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2$ = **-10.3** kN/m

Total; $F_{total\_h} = F_{moist\_h} + F_{pass\_h} + F_{sur\_h}$ = **31.2** kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem}$ = **38.9** kNm/m

Wall base; $M_{base} = F_{base} \times x_{base}$ = **20.2** kNm/m

Surcharge load; $M_{sur} = -F_{sur\_h} \times x_{sur\_h}$ = **-16.1** kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h}$ = **-25.2** kNm/m

Base soil; $M_{pass} = F_{pass\_v} \times x_{pass\_v}$ = **10.2** kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{moist} + M_{pass} + M_{sur}$ = **28** kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h}$ = **31.2** kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v}$ = **562** mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2$ = **-513** mm

Loaded length of base; $l_{load} = 3 \times \bar{x}$ = **1687** mm

Bearing pressure at toe; $q_{toe} = 2 \times F_{total\_v} / l_{load}$ = **59.1** kN/m^2

Bearing pressure at heel; q_{heel} = **0** kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel})$ = **1.693**

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

<u>RETAINING WALL DESIGN</u>

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.6.08

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

Concrete strength class; C35/45

Characteristic compressive cylinder strength; f_{ck} = **35** N/mm^2

Characteristic compressive cube strength; $f_{ck,cube}$ = **45** N/mm^2

Mean value of compressive cylinder strength; $f_{cm} = f_{ck} + 8$ N/mm^2 = **43** N/mm^2

Mean value of axial tensile strength; $f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3}$ = **3.2** N/mm^2

5% fractile of axial tensile strength; $f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.2** N/mm^2

Secant modulus of elasticity of concrete; $E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3}$ = **34077** N/mm^2

Partial factor for concrete - Table 2.1N; γ_C = **1.50**

Compressive strength coefficient - cl.3.1.6(1); α_{cc} = **0.85**

Design compressive concrete strength - exp.3.15; $f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **19.8** N/mm^2

Maximum aggregate size; h_{agg} = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement; f_{yk} = **500** N/mm^2

Modulus of elasticity of reinforcement; E_s = **200000** N/mm^2

Partial factor for reinforcing steel - Table 2.1N; γ_S = **1.15**

Design yield strength of reinforcement; $f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Cover to reinforcement

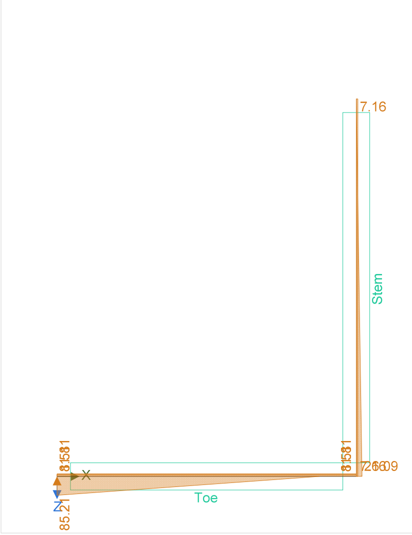
Front face of stem; c_{sf} = **50** mm

Rear face of stem; c_{sr} = **50** mm

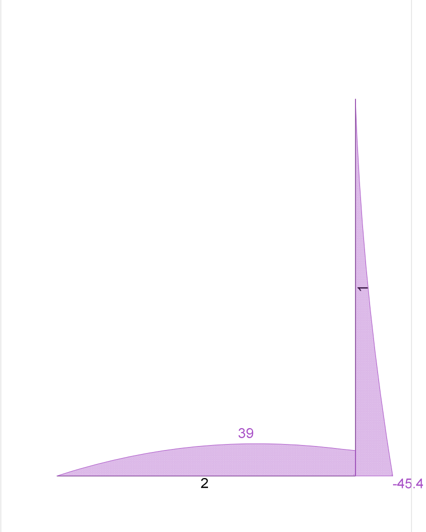
Top face of base; c_{bt} = **50** mm

Bottom face of base; c_{bb} = **50** mm

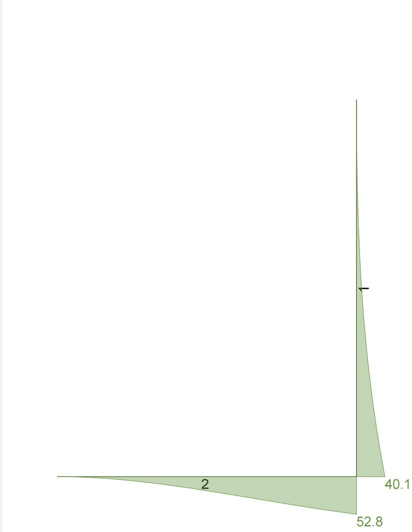
Loading details - Combination No.1 - kN/m²



Shear force - Combination No.1 - kN/m



Bending moment - Combination No.1 - kNm/m



Check stem design at base of stem

Depth of section; $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{40.1}$ kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{295}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.013}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{280}$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{37}$ mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{329}$ mm^2/m

Tension reinforcement provided; 10 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{785}$ mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{492}$ mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h = \mathbf{14000}$ mm^2/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.627}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = \mathbf{0.001}$

Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N; $K_b = \mathbf{0.4}$

Reinforcement factor - exp.7.17; $K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a; $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{136.1}$

Actual span to depth ratio; $h_{stem} / d = \mathbf{7.6}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = \mathbf{0.6}$

Serviceability bending moment; $M_{sls} = \mathbf{23.5}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{107}$ N/mm^2

Load duration; Long term

Load duration factor; $k_t = \mathbf{0.4}$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{104375}$ mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = \mathbf{3.2}$ N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.008}$

Modular ratio; $\alpha_e = E_s / E_{cm} = \mathbf{5.869}$

Bond property coefficient; $k_1 = \mathbf{0.8}$

Strain distribution coefficient; $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{396}$ mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.127** mm

w_k / w_{max} = **0.424**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **45.4** kN/m

$C_{Rd,c}$ = 0.18 / γ_C = **0.120**

k = min(1 + √(200 mm / d), 2) = **1.823**

Longitudinal reinforcement ratio; ρ_l = min($A_{sr.prov}$ / d, 0.02) = **0.003**

v_{min} = 0.035 $N^{1/2}$/mm × $k^{3/2}$ × $f_{ck}^{0.5}$ = **0.510** N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c}$ = max($C_{Rd.c}$ × k × (100 N^2/mm^4 × ρ_l × f_{ck})$^{1/3}$, v_{min}) × d

$V_{Rd.c}$ = **150.4** kN/m

V / $V_{Rd.c}$ = **0.302**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1); $A_{sx.req}$ = max(0.25 × $A_{sr.prov}$, 0.001 × t_{stem}) = **350** mm^2/m

Maximum spacing of reinforcement – cl.9.6.3(2); s_{sx_max} = **400** mm

Transverse reinforcement provided; 10 dia.bars @ 100 c/c

Area of transverse reinforcement provided; $A_{sx.prov}$ = π × ϕ_{sx}^2 / (4 × s_{sx}) = **785** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section; h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **52.8** kNm/m

Depth to tension reinforcement; d = h - c_{bb} - ϕ_{bb} / 2 = **295** mm

K = M / (d^2 × f_{ck}) = **0.017**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; z = min(0.5 + 0.5 × (1 - 3.53 × K)$^{0.5}$, 0.95) × d = **280** mm

Depth of neutral axis; x = 2.5 × (d – z) = **37** mm

Area of tension reinforcement required; $A_{bb.req}$ = M / (f_{yd} × z) = **433** mm^2/m

Tension reinforcement provided; 10 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{bb.prov}$ = π × ϕ_{bb}^2 / (4 × s_{bb}) = **785** mm^2/m

Minimum area of reinforcement - exp.9.1N; $A_{bb.min}$ = max(0.26 × f_{ctm} / f_{yk}, 0.0013) × d = **492** mm^2/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max}$ = 0.04 × h = **14000** mm^2/m

max($A_{bb.req}$, $A_{bb.min}$) / $A_{bb.prov}$ = **0.627**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Limiting crack width; w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1; ψ_2 = **0.6**

Serviceability bending moment; M_{sls} = **37.3** kNm/m

Tensile stress in reinforcement; σ_s = M_{sls} / ($A_{bb.prov}$ × z) = **169.5** N/mm^2

Load duration; Long term

Load duration factor; k_t = **0.4**

Effective area of concrete in tension; $A_{c.eff}$ = min(2.5 × (h - d), (h – x) / 3, h / 2) = **104375** mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff}$ = f_{ctm} = **3.2** N/mm^2

Reinforcement ratio;	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff} = \mathbf{0.008}$
Modular ratio;	$\alpha_e = E_s / E_{cm} = \mathbf{5.869}$
Bond property coefficient;	$k_1 = \mathbf{0.8}$
Strain distribution coefficient;	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11;	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff} = \mathbf{396}$ mm
Maximum crack width - exp.7.8;	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0.201}$ mm
	$w_k / w_{max} = \mathbf{0.671}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

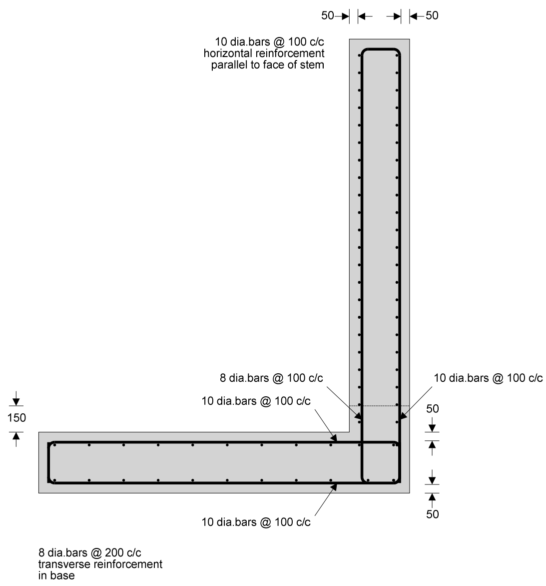
Design shear force;	$V = \mathbf{39}$ kN/m
	$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$
	$k = \min(1 + \sqrt{}(200\text{ mm} / d), 2) = \mathbf{1.823}$
Longitudinal reinforcement ratio;	$\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.003}$
	$v_{min} = 0.035\text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}\text{ N/mm}^2$
Design shear resistance - exp.6.2a & 6.2b;	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} = \mathbf{150.4}$ kN/m
	$V / V_{Rd.c} = \mathbf{0.259}$

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2);	$A_{bx.req} = 0.2 \times A_{bb.prov} = \mathbf{157}\text{ mm}^2/\text{m}$
Maximum spacing of reinforcement – cl.9.3.1.1(3);	$s_{bx\_max} = \mathbf{450}$ mm
Transverse reinforcement provided;	8 dia.bars @ 200 c/c
Area of transverse reinforcement provided;	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = \mathbf{251}\text{ mm}^2/\text{m}$

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details

RETAINING WALL DESIGN CHECK LOAD CASE 2 (GROUND WATER PRESENT):

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Retaining wall details

Stem type;	Cantilever
Stem height;	h_{stem} = **2250** mm
Stem thickness;	t_{stem} = **350** mm
Angle to rear face of stem;	α = **90** deg
Stem density;	γ_{stem} = **25** kN/m^3
Toe length;	l_{toe} = **1800** mm
Heel length;	l_{heel} = **350** mm
Base thickness;	t_{base} = **350** mm
Base density;	γ_{base} = **25** kN/m^3
Height of retained soil;	h_{ret} = **1900** mm
Angle of soil surface;	β = **0** deg
Depth of cover;	d_{cover} = **350** mm
Depth of excavation;	d_{exc} = **350** mm
Height of water;	h_{water} = **1900** mm
Water density;	γ_w = **9.8** kN/m^3

Retained soil properties

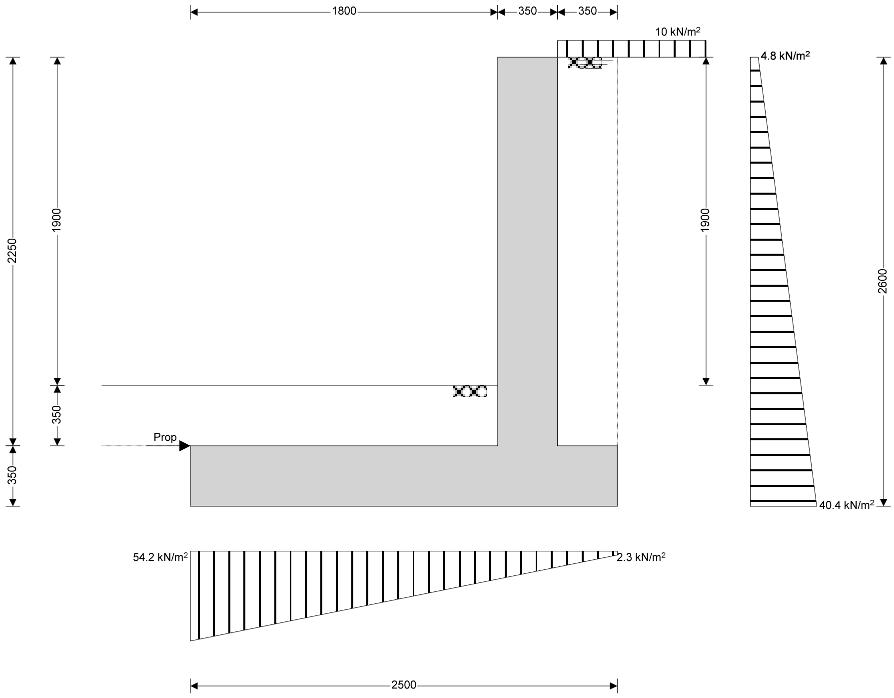
Soil type;	Firm clay
Moist density;	γ_{mr} = **18** kN/m^3
Saturated density;	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type;	Firm clay
Soil density;	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle;	$\phi'_{b.k}$ = **18** deg
Characteristic wall friction angle;	$\delta_{b.k}$ = **9** deg
Characteristic base friction angle;	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity;	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load;	$Surcharge_Q$ = **10** kN/m^2



General arrangement

Calculate retaining wall geometry

Base length;	$l_{base} = l_{toe} + t_{stem} + l_{heel} = \mathbf{2500}$ mm
Saturated soil height;	$h_{sat} = h_{water} + d_{cover} = \mathbf{2250}$ mm
Moist soil height;	$h_{moist} = h_{ret} - h_{water} = \mathbf{0}$ mm
Length of surcharge load;	$l_{sur} = l_{heel} = \mathbf{350}$ mm
- Distance to vertical component;	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{2325}$ mm
Effective height of wall;	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2600}$ mm
- Distance to horizontal component;	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1300}$ mm
Area of wall stem;	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.788}$ m^2
- Distance to vertical component;	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{1975}$ mm
Area of wall base;	$A_{base} = l_{base} \times t_{base} = \mathbf{0.875}$ m^2
- Distance to vertical component;	$x_{base} = l_{base} / 2 = \mathbf{1250}$ mm
Area of saturated soil;	$A_{sat} = h_{sat} \times l_{heel} = \mathbf{0.788}$ m^2
- Distance to vertical component;	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2325}$ mm
- Distance to horizontal component;	$x_{sat\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{867}$ mm
Area of water;	$A_{water} = h_{sat} \times l_{heel} = \mathbf{0.788}$ m^2
- Distance to vertical component;	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat} = \mathbf{2325}$ mm
- Distance to horizontal component;	$x_{water\_h} = (h_{sat} + h_{base}) / 3 = \mathbf{867}$ mm
Area of base soil;	$A_{pass} = d_{cover} \times l_{toe} = \mathbf{0.63}$ m^2
- Distance to vertical component;	$x_{pass\_v} = l_{base} - (d_{cover} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{pass} = \mathbf{900}$ mm
- Distance to horizontal component;	$x_{pass\_h} = (d_{cover} + h_{base}) / 3 = \mathbf{233}$ mm

Using Coulomb theory

Active pressure coefficient; $K_A = \sin(\alpha + \phi'_{r.k})^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta_{r.k}) \times [1 + \sqrt{[\sin(\phi'_{r.k} + \delta_{r.k}) \times \sin(\phi'_{r.k} - \beta) / (\sin(\alpha - \delta_{r.k}) \times \sin(\alpha + \beta))]}]^2) = \mathbf{0.483}$

Passive pressure coefficient; $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{2.359}$

Bearing pressure check

Vertical forces on wall

Wall stem; $F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{19.7}$ kN/m

Wall base; $F_{base} = A_{base} \times \gamma_{base} = \mathbf{21.9}$ kN/m

Surcharge load; $F_{sur\_v} = Surcharge_Q \times l_{heel} = \mathbf{3.5}$ kN/m

Saturated retained soil; $F_{sat\_v} = A_{sat} \times (\gamma_{sr}' - \gamma_w') = \mathbf{6.4}$ kN/m

Water; $F_{water\_v} = A_{water} \times \gamma_w' = \mathbf{7.7}$ kN/m

Base soil; $F_{pass\_v} = A_{pass} \times \gamma_b' = \mathbf{11.3}$ kN/m

Total; $F_{total\_v} = F_{stem} + F_{base} + F_{sat\_v} + F_{pass\_v} + F_{water\_v} + F_{sur\_v} = \mathbf{70.6}$ kN/m

Horizontal forces on wall

Surcharge load; $F_{sur\_h} = K_A \times \cos(\delta_{r.d}) \times Surcharge_Q \times h_{eff} = \mathbf{12.4}$ kN/m

Saturated retained soil; $F_{sat\_h} = K_A \times \cos(\delta_{r.d}) \times (\gamma_{sr}' - \gamma_w') \times (h_{sat} + h_{base})^2 / 2 = \mathbf{13.2}$ kN/m

Water; $F_{water\_h} = \gamma_w' \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{33.2}$ kN/m

Moist retained soil; $F_{moist\_h} = K_A \times \cos(\delta_{r.d}) \times \gamma_{mr}' \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{0}$ kN/m

Base soil; $F_{pass\_h} = -K_P \times \cos(\delta_{b.d}) \times \gamma_b' \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-10.3}$ kN/m

Total; $F_{total\_h} = F_{sat\_h} + F_{moist\_h} + F_{pass\_h} + F_{water\_h} + F_{sur\_h} = \mathbf{48.5}$ kN/m

Moments on wall

Wall stem; $M_{stem} = F_{stem} \times x_{stem} = \mathbf{38.9}$ kNm/m

Wall base; $M_{base} = F_{base} \times x_{base} = \mathbf{27.3}$ kNm/m

Surcharge load; $M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-8}$ kNm/m

Saturated retained soil; $M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h} = \mathbf{3.5}$ kNm/m

Water; $M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h} = \mathbf{-10.8}$ kNm/m

Moist retained soil; $M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{0}$ kNm/m

Base soil; $M_{pass} = F_{pass\_v} \times x_{pass\_v} = \mathbf{10.2}$ kNm/m

Total; $M_{total} = M_{stem} + M_{base} + M_{sat} + M_{moist} + M_{pass} + M_{water} + M_{sur} = \mathbf{61.2}$ kNm/m

Check bearing pressure

Propping force; $F_{prop\_base} = F_{total\_h} = \mathbf{48.5}$ kN/m

Distance to reaction; $\bar{x} = M_{total} / F_{total\_v} = \mathbf{867}$ mm

Eccentricity of reaction; $e = \bar{x} - l_{base} / 2 = \mathbf{-383}$ mm

Loaded length of base; $l_{load} = l_{base} = \mathbf{2500}$ mm

Bearing pressure at toe; $q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{54.2}$ kN/m^2

Bearing pressure at heel; $q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{2.3}$ kN/m^2

Factor of safety; $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.847}$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.6.08

Concrete details - Table 3.1 - Strength and deformation characteristics for concrete

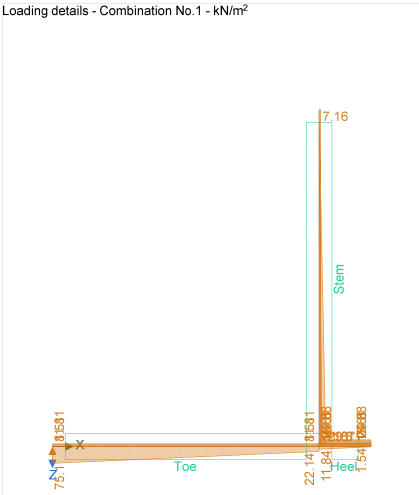
Concrete strength class;	C35/45
Characteristic compressive cylinder strength;	f_{ck} = **35** N/mm^2
Characteristic compressive cube strength;	$f_{ck,cube}$ = **45** N/mm^2
Mean value of compressive cylinder strength;	$f_{cm} = f_{ck} + 8$ N/mm^2 = **43** N/mm^2
Mean value of axial tensile strength;	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck}$ / 1 N/mm$^2)^{2/3}$ = **3.2** N/mm^2
5% fractile of axial tensile strength;	$f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.2** N/mm^2
Secant modulus of elasticity of concrete;	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm}$ / 10 N/mm$^2)^{0.3}$ = **34077** N/mm^2
Partial factor for concrete - Table 2.1N;	γ_C = **1.50**
Compressive strength coefficient - cl.3.1.6(1);	α_{cc} = **0.85**
Design compressive concrete strength - exp.3.15;	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C$ = **19.8** N/mm^2
Maximum aggregate size;	h_{agg} = **20** mm

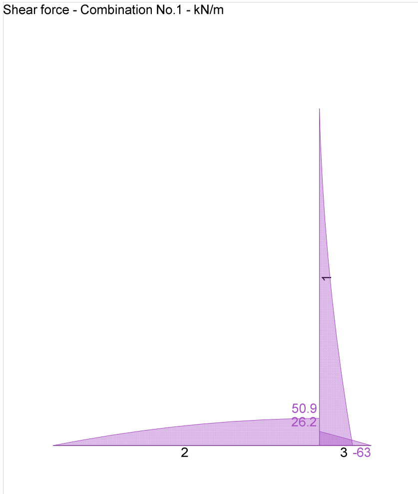
Reinforcement details

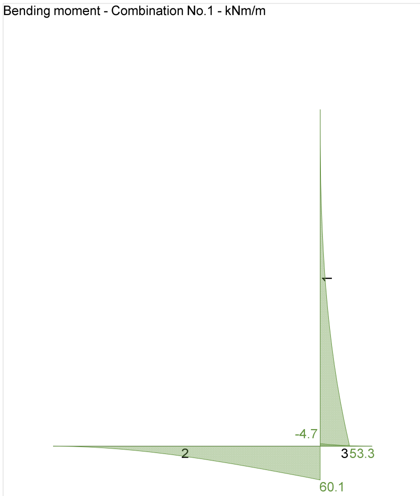
Characteristic yield strength of reinforcement;	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement;	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N;	γ_S = **1.15**
Design yield strength of reinforcement;	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

Cover to reinforcement

Front face of stem;	c_{sf} = **50** mm
Rear face of stem;	c_{sr} = **50** mm
Top face of base;	c_{bt} = **50** mm
Bottom face of base;	c_{bb} = **50** mm







Check stem design at base of stem

Depth of section; $h = \mathbf{350}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; $M = \mathbf{53.3}$ kNm/m

Depth to tension reinforcement; $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{294}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.018}$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{279}$ mm

Depth of neutral axis; $x = 2.5 \times (d - z) = \mathbf{37}$ mm

Area of tension reinforcement required; $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{439}$ mm²/m

Tension reinforcement provided; 12 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{1131}$ mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{491}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{sr.max} = 0.04 \times h = \mathbf{14000}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.434}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Deflection control - Section 7.4

Reference reinforcement ratio; $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.006}$

Required tension reinforcement ratio; $\rho = A_{sr.req} / d = \mathbf{0.001}$

Required compression reinforcement ratio; $\rho' = A_{sr.2.req} / d_2 = \mathbf{0.000}$

Structural system factor - Table 7.4N; $K_b = \mathbf{0.4}$

Reinforcement factor - exp.7.17; $K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5) = \mathbf{1.5}$

Limiting span to depth ratio - exp.7.16.a; $K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}] = \mathbf{85.7}$

Actual span to depth ratio; $h_{stem} / d = \mathbf{7.7}$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width; $w_{max} = \mathbf{0.3}$ mm

Variable load factor - EN1990 – Table A1.1; $\psi_2 = \mathbf{0.6}$

Serviceability bending moment; $M_{sls} = \mathbf{33.3}$ kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{105.4}$ N/mm^2

Load duration; Long term

Load duration factor; $k_t = \mathbf{0.4}$

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h – x) / 3, h / 2) = \mathbf{104417}$ mm^2/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm} = \mathbf{3.2}$ N/mm^2

Reinforcement ratio; $\rho_{p.eff} = A_{sr.prov} / A_{c.eff} = \mathbf{0.011}$

Modular ratio; $\alpha_e = E_s / E_{cm} = \mathbf{5.869}$

Bond property coefficient; $k_1 = \mathbf{0.8}$

Strain distribution coefficient; $k_2 = \mathbf{0.5}$

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{358}$ mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s – k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

$w_k = \mathbf{0.113}$ mm

$w_k / w_{max} = \mathbf{0.378}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; $V = \mathbf{63}$ kN/m

$C_{Rd,c} = 0.18 / \gamma_C = \mathbf{0.120}$

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \mathbf{1.825}$

Longitudinal reinforcement ratio; $\rho_l = \min(A_{sr.prov} / d, 0.02) = \mathbf{0.004}$

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}$ N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c} = \mathbf{153.2}$ kN/m

$V / V_{Rd.c} = \mathbf{0.411}$

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1); $A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem}) = \mathbf{350}$ mm^2/m

Maximum spacing of reinforcement – cl.9.6.3(2); $s_{sx\_max} = \mathbf{400}$ mm

Transverse reinforcement provided; 12 dia.bars @ 100 c/c

Area of transverse reinforcement provided; $A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **1131** mm²/m

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section; h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1; M = **60.1** kNm/m

Depth to tension reinforcement; $d = h - c_{bb} - \phi_{bb} / 2$ = **294** mm

$K = M / (d^2 \times f_{ck})$ = **0.020**

K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm; $z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d$ = **279** mm

Depth of neutral axis; $x = 2.5 \times (d - z)$ = **37** mm

Area of tension reinforcement required; $A_{bb.req} = M / (f_{yd} \times z)$ = **495** mm²/m

Tension reinforcement provided; 12 dia.bars @ 100 c/c

Area of tension reinforcement provided; $A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1131** mm²/m

Minimum area of reinforcement - exp.9.1N; $A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **491** mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3); $A_{bb.max} = 0.04 \times h$ = **14000** mm²/m

$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.438**

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Limiting crack width; w_{max} = **0.3** mm

Variable load factor - EN1990 – Table A1.1; ψ_2 = **0.6**

Serviceability bending moment; M_{sls} = **43.2** kNm/m

Tensile stress in reinforcement; $\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **136.7** N/mm²

Load duration; Long term

Load duration factor; k_t = **0.4**

Effective area of concrete in tension; $A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ = **104417** mm²/m

Mean value of concrete tensile strength; $f_{ct.eff} = f_{ctm}$ = **3.2** N/mm²

Reinforcement ratio; $\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.011**

Modular ratio; $\alpha_e = E_s / E_{cm}$ = **5.869**

Bond property coefficient; k_1 = **0.8**

Strain distribution coefficient; k_2 = **0.5**

k_3 = **3.4**

k_4 = **0.425**

Maximum crack spacing - exp.7.11; $s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **358** mm

Maximum crack width - exp.7.8; $w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$

w_k = **0.147** mm

w_k / w_{max} = **0.49**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **50.9** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

	$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \mathbf{1.825}$
Longitudinal reinforcement ratio;	$\rho_l = \min(A_{bb.prov} / d, 0.02) = \mathbf{0.004}$
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.510}\ N/mm^2$
Design shear resistance - exp.6.2a & 6.2b;	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c} = \mathbf{153.2}$ kN/m
	$V / V_{Rd.c} = \mathbf{0.333}$

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section;	h = **350** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1;	M = **4.7** kNm/m
Depth to tension reinforcement;	$d = h - c_{bt} - \phi_{bt} / 2 = \mathbf{294}$ mm
	$K = M / (d^2 \times f_{ck}) = \mathbf{0.002}$
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm;	$z = \min(0.5 + 0.5 \times (1 - 3.53 \times K)^{0.5}, 0.95) \times d = \mathbf{279}$ mm
Depth of neutral axis;	$x = 2.5 \times (d - z) = \mathbf{37}$ mm
Area of tension reinforcement required;	$A_{bt.req} = M / (f_{yd} \times z) = \mathbf{39}\ mm^2/m$
Tension reinforcement provided;	12 dia.bars @ 100 c/c
Area of tension reinforcement provided;	$A_{bt.prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = \mathbf{1131}\ mm^2/m$
Minimum area of reinforcement - exp.9.1N;	$A_{bt.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{491}\ mm^2/m$
Maximum area of reinforcement - cl.9.2.1.1(3);	$A_{bt.max} = 0.04 \times h = \mathbf{14000}\ mm^2/m$
	$\max(A_{bt.req}, A_{bt.min}) / A_{bt.prov} = \mathbf{0.434}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Crack control - Section 7.3

Limiting crack width;	$w_{max} = \mathbf{0.3}$ mm
Variable load factor - EN1990 – Table A1.1;	$\psi_2 = \mathbf{0.6}$
Serviceability bending moment;	$M_{sls} = \mathbf{3.1}$ kNm/m
Tensile stress in reinforcement;	$\sigma_s = M_{sls} / (A_{bt.prov} \times z) = \mathbf{9.8}\ N/mm^2$
Load duration;	Long term
Load duration factor;	$k_t = \mathbf{0.4}$
Effective area of concrete in tension;	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2) = \mathbf{104417}\ mm^2/m$
Mean value of concrete tensile strength;	$f_{ct.eff} = f_{ctm} = \mathbf{3.2}\ N/mm^2$
Reinforcement ratio;	$\rho_{p.eff} = A_{bt.prov} / A_{c.eff} = \mathbf{0.011}$
Modular ratio;	$\alpha_e = E_s / E_{cm} = \mathbf{5.869}$
Bond property coefficient;	$k_1 = \mathbf{0.8}$
Strain distribution coefficient;	$k_2 = \mathbf{0.5}$
	$k_3 = \mathbf{3.4}$
	$k_4 = \mathbf{0.425}$
Maximum crack spacing - exp.7.11;	$s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff} = \mathbf{358}$ mm
Maximum crack width - exp.7.8;	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	$w_k = \mathbf{0.011}$ mm
	$w_k / w_{max} = \mathbf{0.035}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force; V = **26.2** kN/m

$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**

$k = \min(1 + \sqrt{(200\ mm / d)}, 2)$ = **1.825**

Longitudinal reinforcement ratio; $\rho_l = \min(A_{bt.prov} / d, 0.02)$ = **0.004**

$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.510** N/mm^2

Design shear resistance - exp.6.2a & 6.2b; $V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$

$V_{Rd.c}$ = **153.2** kN/m

$V / V_{Rd.c}$ = **0.171**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

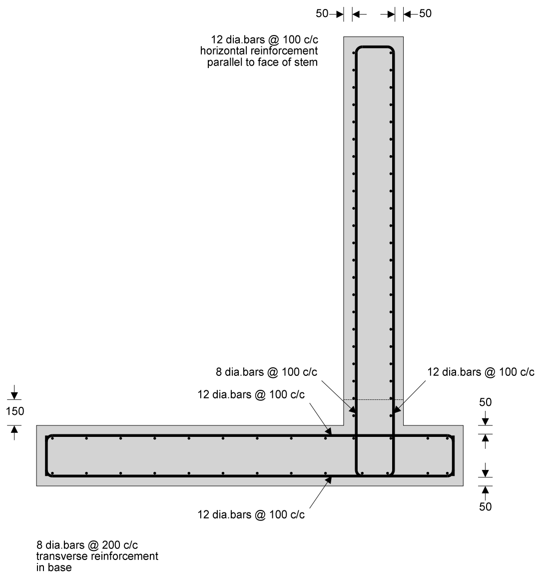
Minimum area of reinforcement – cl.9.3.1.1(2); $A_{bx.req} = 0.2 \times A_{bb.prov}$ = **226** mm^2/m

Maximum spacing of reinforcement – cl.9.3.1.1(3); s_{bx_max} = **450** mm

Transverse reinforcement provided; 8 dia.bars @ 200 c/c

Area of transverse reinforcement provided; $A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **251** mm^2/m

PASS - Area of reinforcement provided is greater than area of reinforcement required



Reinforcement details