civils ltd - consulting engineers

BASEMENT IMPACT ASSESSMENT

Land Adjacent to 1 Ellerdale Road
London NW3 6BA

CLIENT
Mr. Georg Galberg
Flat C, 15 Cleveland Square
London
W2 6DG

Ref: 4555/2.3F
Date: November 2012

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SCHEDULE OF APPENDICES

Issue	Date	Compiled	Checked
First Issue	13 November '12	JP	MR

Report by: **John Pakenham - GTA Civils - (Hydrology) BSc (Hons)**
Jeff Walker - AND Designs – (Structural) C.Eng, MIStructE
Checked by: **Martin Roberts - GTA Civils - (Hydrology) I Eng, CIWEM, MCIHT**

1.0 INTRODUCTION

1.1 This report has been prepared for Mr. G. Galberg in relation to land adjacent to the rear garden of 1 Ellerdale Road, London NW3 6BA. No responsibility is accepted to any third party for all or part of this study in connection with this or any other development.

1.2 GTA Civils Ltd. was appointed by its client to provide a Basement Impact Assessment (BIA) as requested by Camden Council in order to achieve Planning Approval at said property.

1.3 This report has been structured to cover the topics outlined in Camden's policy document DP27, namely the proposed scheme's impact on local drainage and flooding and on the structural stability of neighbouring properties through its effect on groundwater conditions and ground movement.

1.4 Note that the structural 'design' is sufficiently comprehensive for planning purposes only. It is not intended as a fully worked up design to comply with current Building Regulations.

2.0 EXISTING SITE

2.1 The site comprises an area of cleared garden adjacent to the rear garden of 1 Ellerdale Road, which is in the London Borough of Camden.

2.2 The site is 45m southwest of the junction with Fitzjohns Avenue. An existing site location map and photos of the site are shown in Appendix A. The site slopes to the southwest.

2.3 The BGS online geology map indicates this site on the junction between the Bagshot Formation (Sandstone) and the Claygate Member (clay, silt and sand.) This is the highest solid (or bedrock) stratum with no superficial (or 'drift') deposits recorded.

2.4 As there are no particular groundwater issues (e.g. underground rivers nearby) this assessment was not deemed to need input from a chartered professional hydro-geologist.

2.5 The has planning permission to build a single storey flat roofed garden house. The proposal is to extend this scheme by forming and additional basement level – see the proposed scheme drawings in Appendix C and the planning decision notice in Appendix F.

3.0 CPG4 SCREENING FLOWCHARTS

3.1 Subterranean (Groundwater) Flow

1A: Is the site located directly above an aquifer?
Yes, the site is underlain by a minor aquifer, as denoted on the EA's 'Groundwater Vulnerability' and solid bedrock aquifers maps – see excerpts in Appendix D: carry forward to scoping stage.

1B: Will the proposed basement extend beneath the water table surface?
No water was encountered in the boreholes.

2: Is the site within 100m of a watercourse, well (used/disused) or potential spring line?

No, the site is further away than 100m from the nearest watercourse, well or potential spring line.

3: Is the site within the catchment of the pond chains on Hampstead Heath?
No, it is not near this area.

4: Will the proposed basement development result in a change in the proportion of hard surface/paved areas?
Yes the amount of hardstanding areas will increase as the new unit will replace soft landscaped area – carry forward to scoping stage.

5: As part of the site drainage, will more surface water (e.g. rainfall and run-off) than at present be discharged to ground (e.g. via soakaways and/or SUDS)?
No, the use of infiltration methods is not possible due to lack of external space.

6: Is the lowest point of the proposed excavation (allowing for any drainage and foundation space under the basement floor) close to, or lower than, the mean water level of any local pond (not just the chain of ponds in Hampstead Heath) or spring line?
No, the elevation of the site is approximately 100m AOD and there are no ponds or spring lines hydraulically connected to the site.

3.2 Slope Stability

1: Does the existing site include slopes, natural or man-made, greater than 7° (approximately 1 in 8)?
No, the site's gradient is less than 1:8.

2: Will the proposed re-profiling of landscaping at site change slopes at the property boundary to greater than 7° (approximately 1 in 8)?
No, the proposal does not include landscaping that affects the boundaries.

3: Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?
No, the neighbouring sites are at a similar gradient.

4: Is the site within a wider hillside setting in which the general slope is greater than 7° (approximately 1 in 8)?
No, the wider gradient is less than 1:8.

5: Is London Clay the shallowest stratum on the site?
No London Clay was found on this site, just Bagshot Beds and Claygate member deposits.

6: Will any trees be felled as part of the proposed development and/or are there any proposed works within any tree protection zones where trees are to be retained?
No trees are to be felled as part of this proposal.

7: Is there a history of shrink-swell subsidence in the local area and/or evidence of such effects at the site?
There is no such evidence to this or neighbouring properties, which is expected as London Clay is not the major stratum.

8: Is the site within 100m of a watercourse, or spring line?
No, the site is further away than 100m from the nearest watercourse or spring line.

9: Is the site within an area of previously worked ground?
Yes, the borehole records show made ground to 2.9m below ground level – carry forward to scoping stage.

10: Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering will be required during construction?
Yes, the site is within an aquifer - carry forward to scoping stage.

11: Is the site within 50m of the Hampstead Heath ponds?
No, it is significantly further than 50m away from these ponds.

12: is the site within 5m of a public highway or pedestrian right of way?
No, the proposed extension is further than 5m from the nearest highway/pedestrian right of way.

13: Will the proposed basement significantly extend the differential depth of basements relative to neighbouring properties?
Yes, the basement is being formed adjacent to the neighbouring property – carry forward to scoping stage.

14: Is the site over (or within the exclusion zone of) any tunnels, e.g. railway lines?
No, the site is outside all such exclusion zones.

3.3 Surface Flow and Flooding

1: Is the site within the catchment of the pond chains on Hampstead Heath?
No, the site is well removed from these ponds and outside the catchment area.

2: As part of the proposed site drainage, will surface water flows (e.g. volume of rainfall and peak run-off) be materially changed from the existing route?
No, these will be unaffected as the lower ground floor is only 1m lower than ground level.

3: Will the proposed basement development result in a change in the proportion of hard surfaces/paved external areas?
Yes, the amount and proportion of hardstanding areas will increase - carry forward to scoping stage.

4: Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by adjacent properties or downstream watercourses?
No, there will be no surface water flow off-site as a result of this proposal.

5 Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?
No, there will be no surface water flow off-site as a result of this proposal.

6 Is the site in an area known to be at risk from surface water flooding, such as Hampstead Heath, Gospel Oak and King's Cross, or is it at risk from flooding, for example because the proposed basement is below the static water level of a nearby surface water feature?
No, the site is not in an area susceptible to surface water flooding – as per Figure 15 of the Camden Geological, Hydrogeological and Hydrological Study.

4.0 SCOPING STAGE

4.1 Subterranean (Groundwater) Flow

1A: The site overlies a minor aquifer as denoted on the EA's 'Groundwater Vulnerability' and solid bedrock aquifers maps – see excerpt in Appendix D. The proposal does not impact on this, however.

4 & 5: The increase in impermeable area does not impact on the groundwater flow as the water table is considerably lower than 9m below current ground levels.

4.2 Slope Stability

9: The presence of made ground to the depth of 2.9m does not constitute a cause for concern as the depth of the new construction is greater than this: all made ground below formation level will be removed.

13. There will be a sequence of remedial work to underpin the adjacent properties, together with a complex sequence of work involving 'top-down' construction methodology. This is necessary due to the limited access and minimal stable working area within the confines of this site. See Appendix D for the structural method statements, sketches and specifications.

4.3 Surface Flow and Flooding

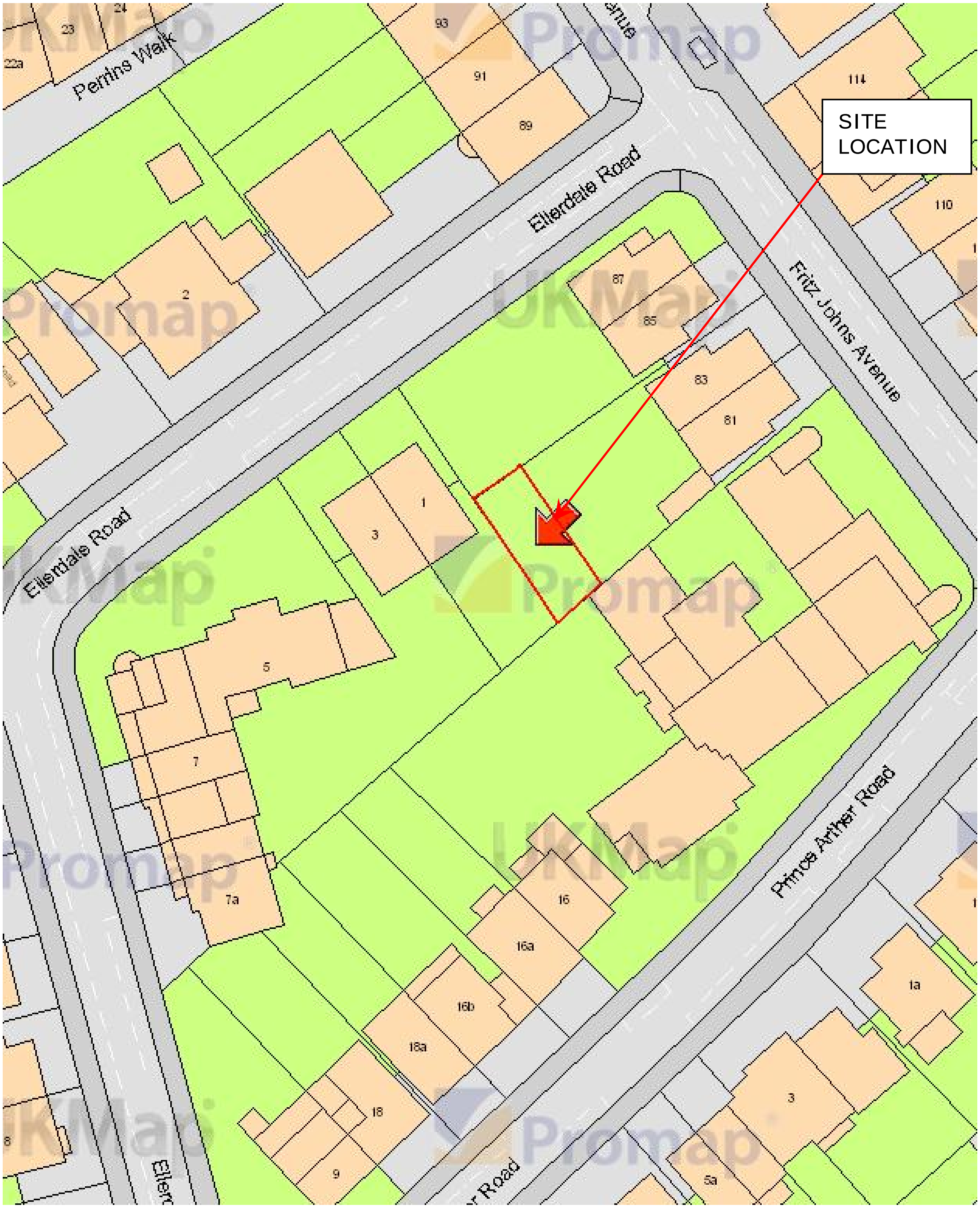
3: the roof's surface water will drain to to the nearest sewer in Ellerdale Road. There is insufficient space for infiltration methods to be used on this site.

It is concluded that this proposal is safe and all of the points requiring to be covered in Section 1.3 have been dealt with conclusively.

- End of Report -

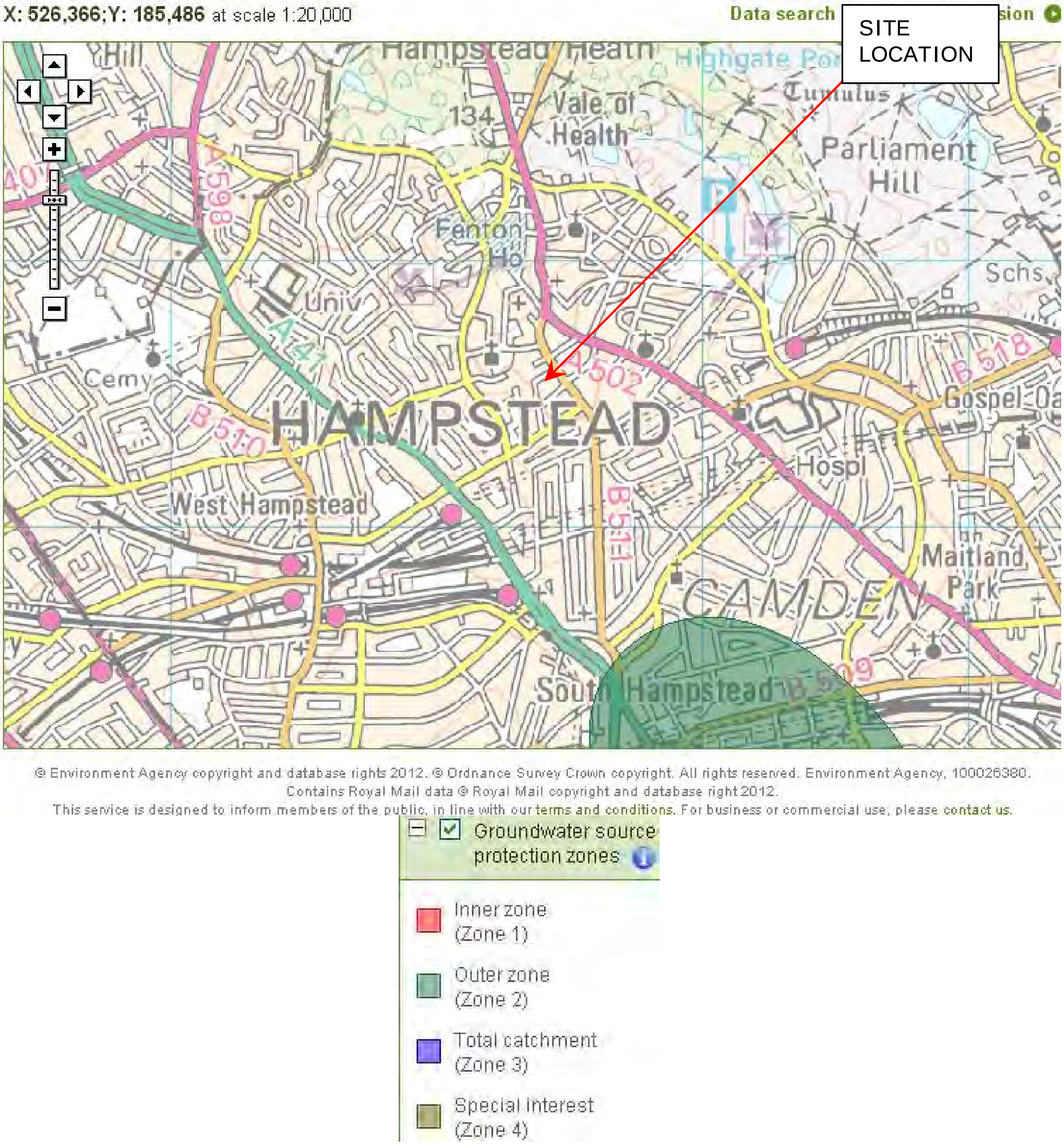
APPENDIX A

Site Location Map & Photos of the Site



APPENDIX B

Environment Agency's Goundwater Maps



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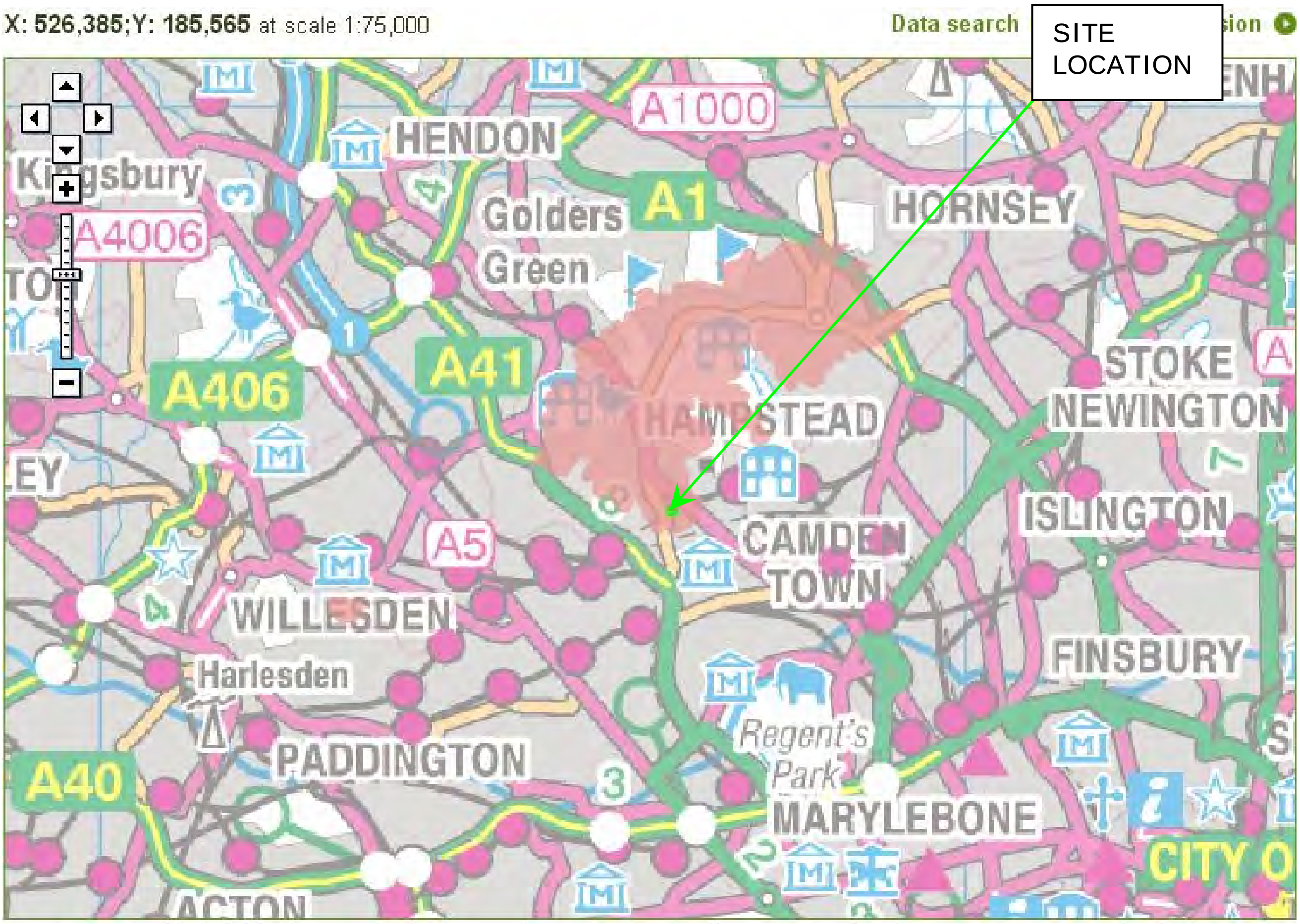
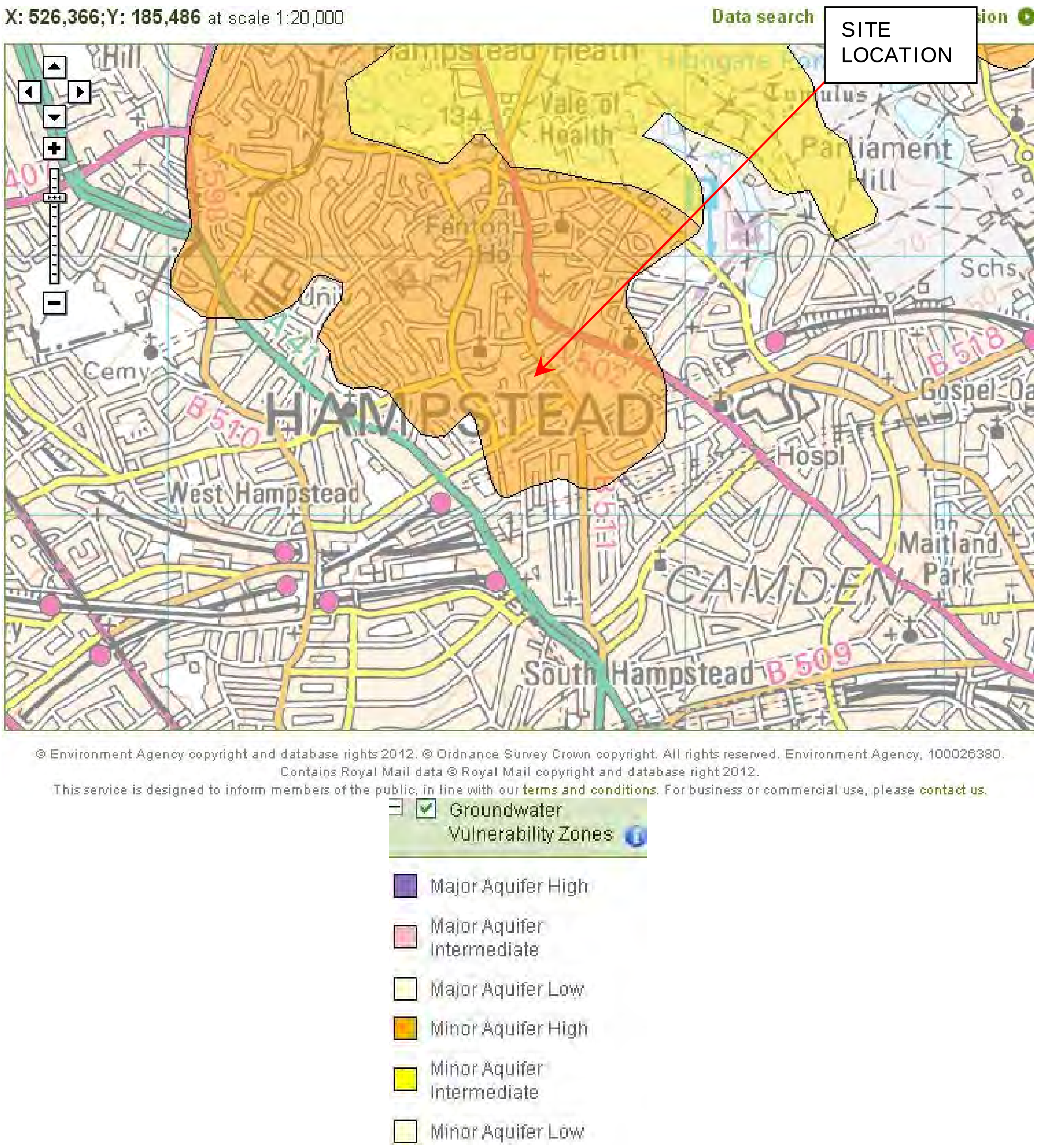


Aquifer Maps - Bedrock Designation

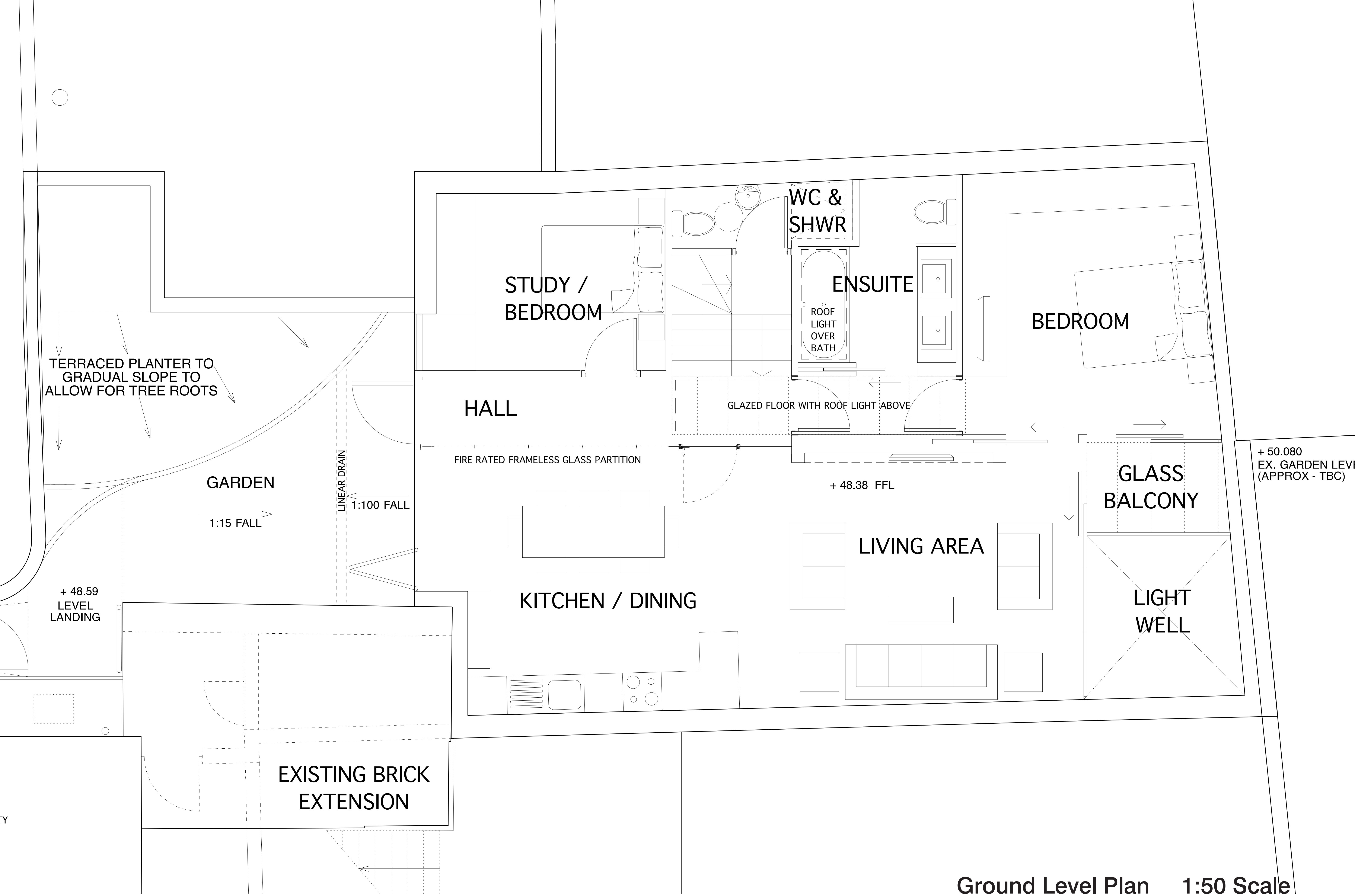
- Principal
- Secondary A
- Secondary B
- Secondary (undifferentiated)



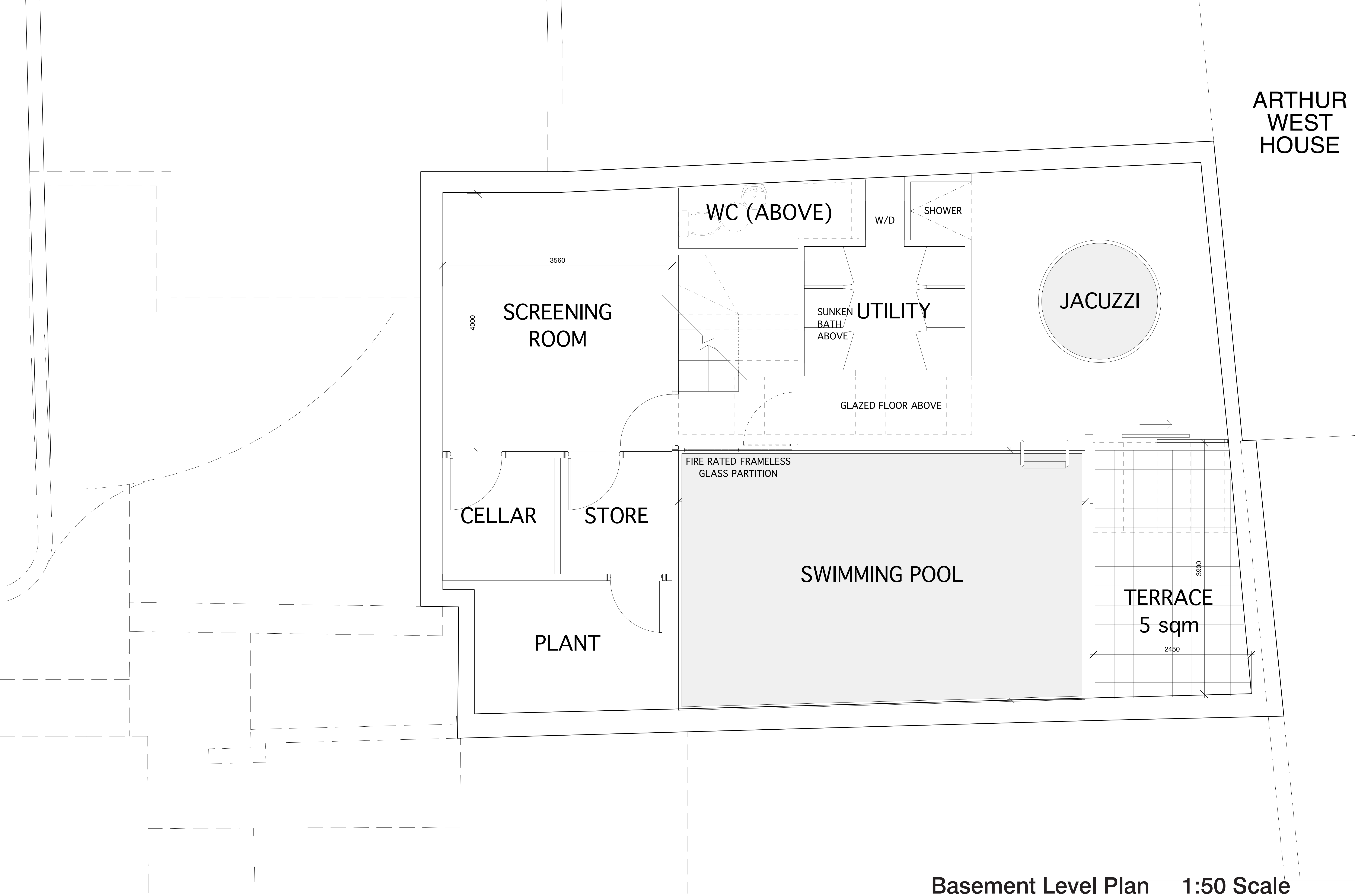
W:\Projects\4555 BIA, Burrell Foley Fisher, 1 Ellerdale Road, London NW3 6BA\2.3 Specifications & Reports\F. BIA	Date	Job No.
	November 2012	4555/2.3F

APPENDIX C

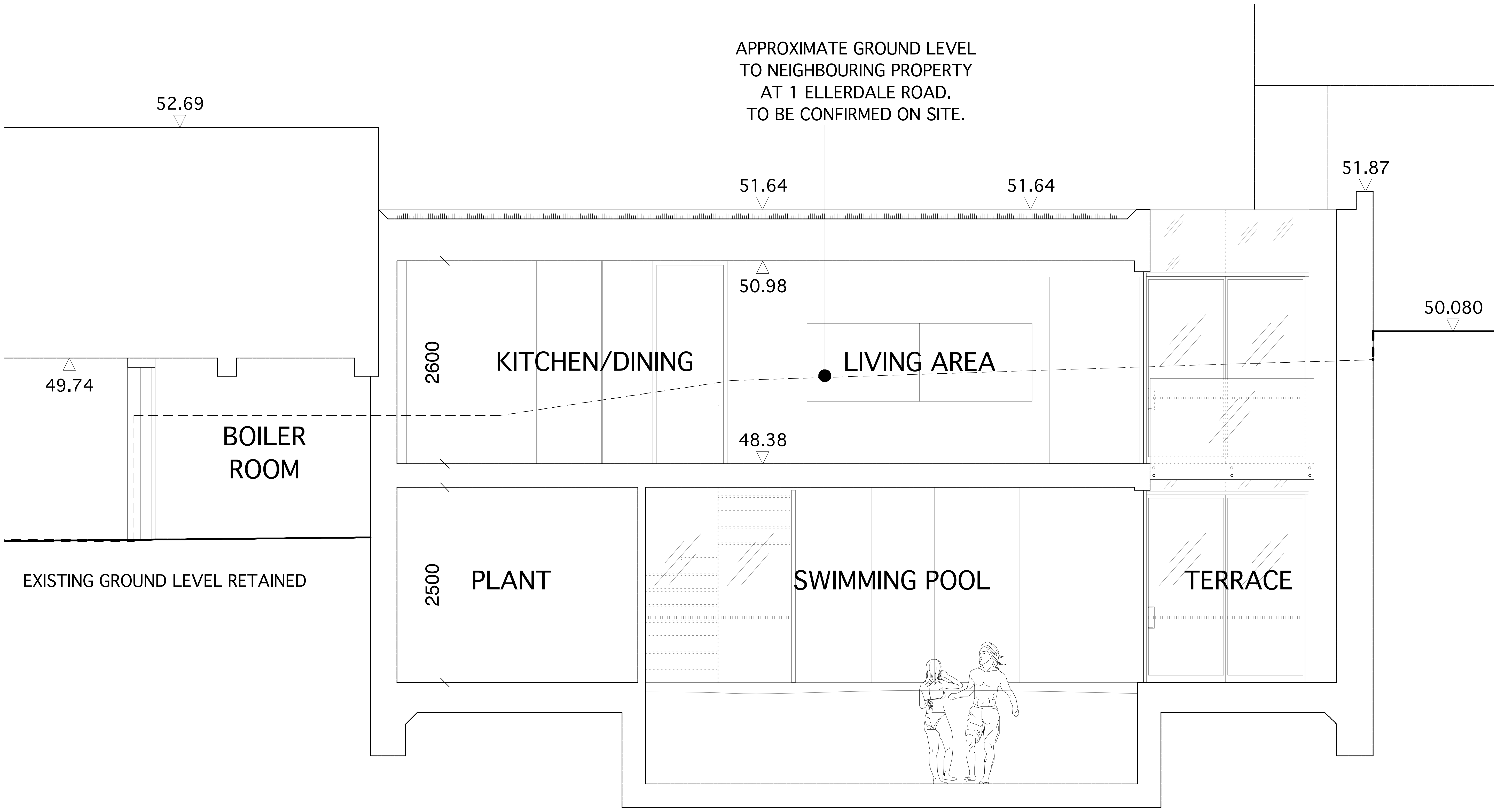
Architect's Scheme Drawings



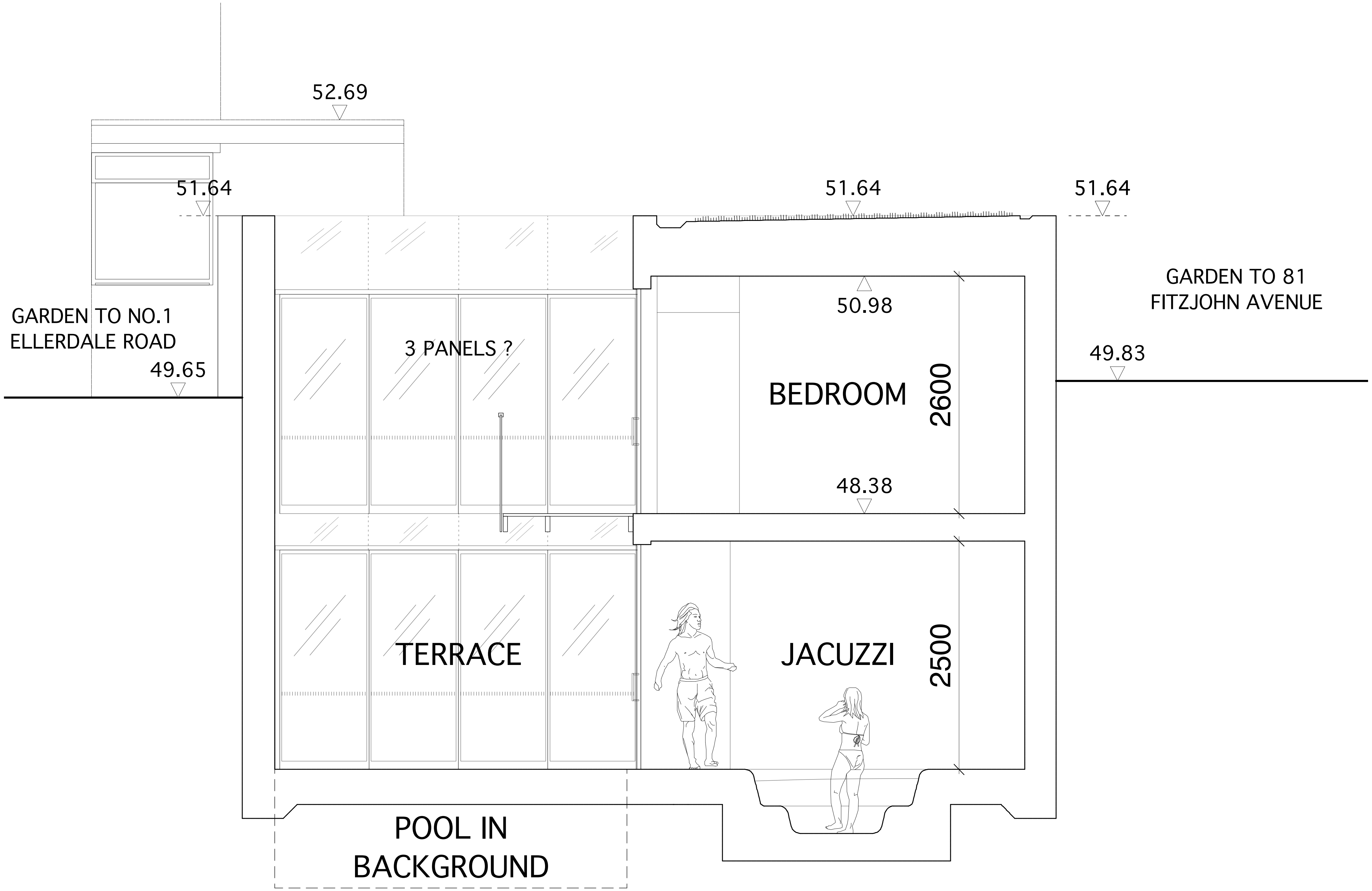
Ground Level Plan 1:50 Scale



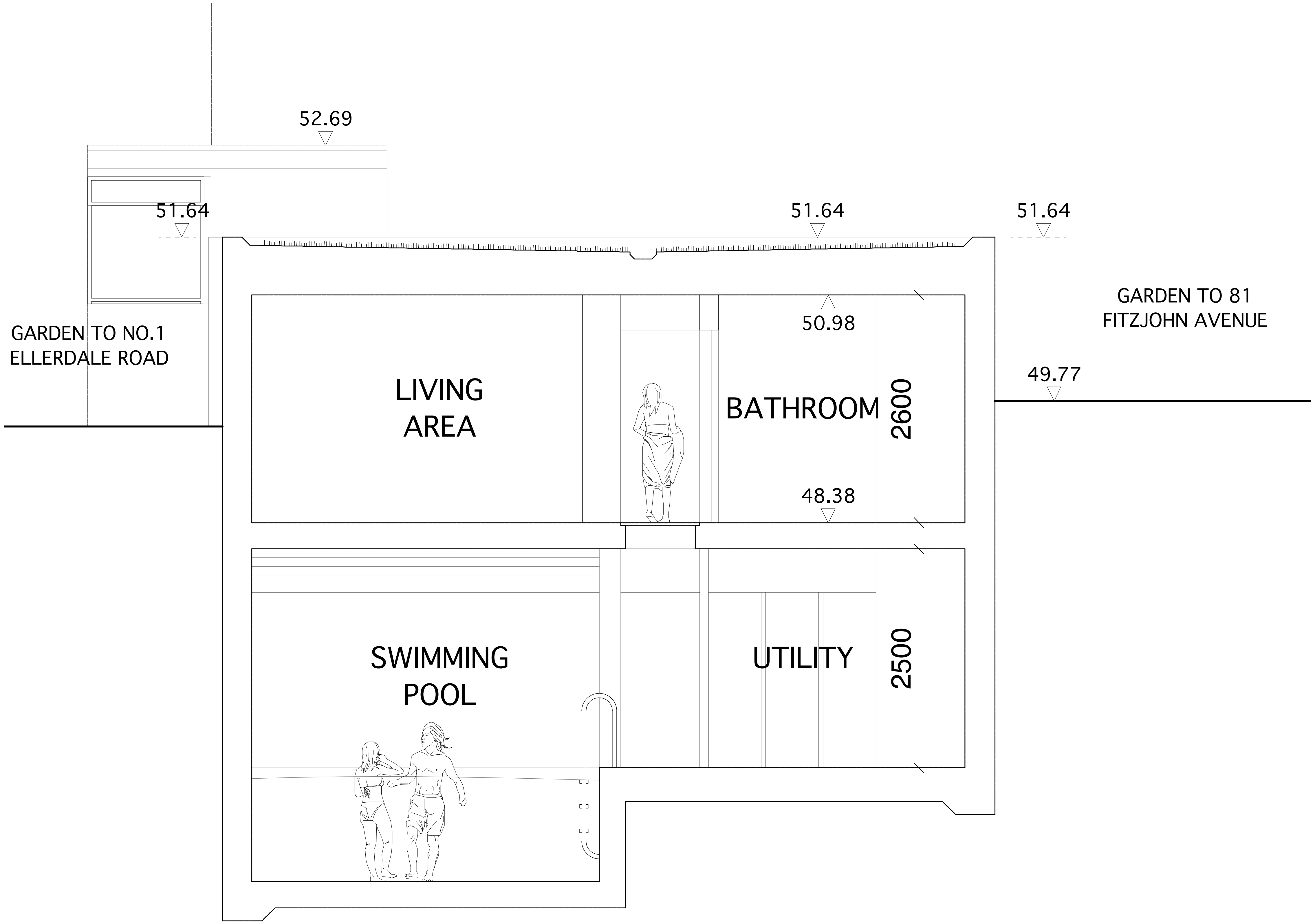
Basement Level Plan 1:50 Scale



Section A-A 1:50 Scale



Section B-B 1:50 Scale



Section C-C 1:50 Scale

APPENDIX D

Structural Method Statement, Specifications & Sketches

AND DESIGNS LTD 90 Meadrow, Godalming, Surrey GU7 3HY Tel: 01483 418 140 Fax: 01483 421 304 e-mail: info@anddesigns.co.uk	Project 1 ELLERDALE ROAD HAMPSTEAD NW3			Job Ref. 12 .195	
	Part of Structure PROPOSED NEW BASEMENT			Sheet No./rev. Design principles	
	Calc. by JW	Date OCT 12	Chck'd by	Date	App'd by	Date

Ref.	Calculations	Output

DESIGN PRINCIPLES (FEASIBILITY DESIGN)

The existing site is surrounded on four boundaries and it is intended to design for a two story basement to the rear of the property access is limited and therefore underpinning is required to obtain the required depths of the basement.

The Geological report indicates fill to 2.9m below the existing ground level but should be capable of supporting a ground slab during the course of the works. The ground below the fill is Bagshot beds and has an allowable bearing Pressure of 100 Kpa

METHOD STATEMENT.(In Principle)

1. Cast the retaining wall to the proposed ground floor level allowing for all loading conditions on the retaining wall cast the toe of the slab with reinforcement to allow for the design of the slab or retaining wall.

2. Cast the ground floor slab in the temporary condition on the existing ground with 150mm of hardcore and blinding reinforcement to suit span conditions and all relevant beam strips and loadings.

3. Allow for a central trench under the centre of the building and forming the line of the new support prop and strut to allow for the construction of the pins install the columns and temporary bases/props to allow for the excavation of the lower underpin sections from ground to basement.

4. Excavate sequence as indicated on the sketches and cast basement slab onto the bagshot beds

5. Cast remaining central slab and design for all hydrostatic loads

6. Allow slab to cure and install waterproof system to the manufacturer's details

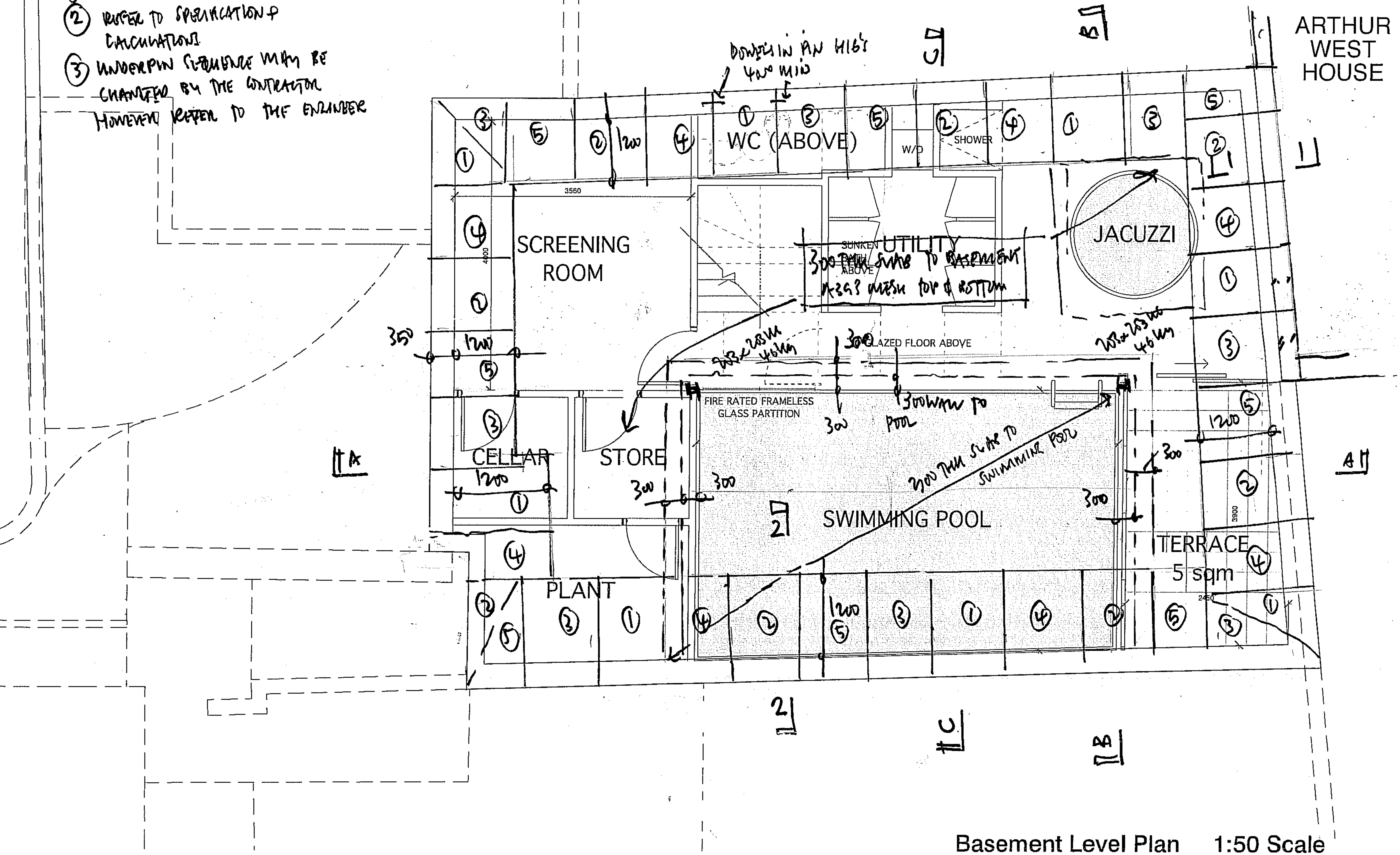
7. Refer to method statement for the general underpinning sequence for the pins

1 ELLERDALE ROAD HAMPSTEAD NW3

12.195/SK/01

NOTES

1. DO NOT SCALE
2. REFER TO SPECIFICATION & CALCULATIONS
3. UNDERPIN SEQUENCE MAY BE CHANGED BY THE CONTRACTOR HOWEVER REFER TO THE ENGINEER



Basement Level Plan 1:50 Scale

METHOD STATEMENT FOR STRUCTURAL BASEMENT WORKS TO 1 ELLERDALE ROAD, HAMPSTEAD

1. Carefully excavate the external area directly in front of the existing front bay window. Construct form work insert mesh and pour concrete base to front garden storage. Insert trench sheets behind the slab position (if ground conditions dictate this), construct mesh for vertical concrete walls and construct formwork and pour vertical sides. Construct formwork for slab over inserting mesh and re-bar as per structural engineer's details. Prop beneath and pour concrete. Leave for 14 days before removing props.

2. The concrete box will be used as a loading platform for excavated spoil before being loaded into skips on the road.

3. Each pin is excavated and is no more than 1m in width and poured strictly in line with the sequence and the structural engineer's details with props installed, 4 for each pin. We will use our best endeavours to ensure that the thickness of the underpin matches the thickness of the party wall unless shown otherwise. As the excavation for each underpin progresses, the thickness and depth of the pin will be carefully monitored, ensuring a vertical and where possible smooth shuttering face against the substrate soil. Each pin will be poured in 4 Stages as follows:

   a. The strip foundations will be excavated and cast first and these will be done so in 1m lengths. The excavation below the neighbour's properties will be carried out carefully and assuming that the soil is self supporting enough to allow a plywood box to be inserted to act as formwork and support the soil above. This will then enable the strip foundation (mass concrete) to be poured. Plywood will be secured against the face of the excavation and propped off the central mass while the excavation is open.
   b. The re-enforcement will be installed in the toe section – refer to Structural Engineers details. The Toe will be poured with shuttering propped at high and low level at the back of the pin if necessary (ground conditions will dictate this) to stop spoil falling into the excavation.
   c. 24 Hours after the toe has been poured, mesh will be installed to the vertical section (refer to Structural Engineers details). Shuttering will be installed on the nearside of the pin and propped using acro props at high and low level (4 or 6 in total). The vertical section of the pin will be poured.
   d. 48 Hours after the pin has been poured, the props and shuttering can be removed before the next pin in the sequence (not adjacent) is started. Finally 75mm of drypacking is rammed in to the gap between the top of the pin and the underside of the existing foundation with the exposed section of the footing on the nearside being removed. Temporary propping (Acro Props) is installed to support the pin once the shuttering has been removed, with 2 across at high level and 2 across at low level per pin.

4. The Retaining walls will be created in a similar methodology to the underpins, but there is no need for the strip foundation to be excavated and cast.

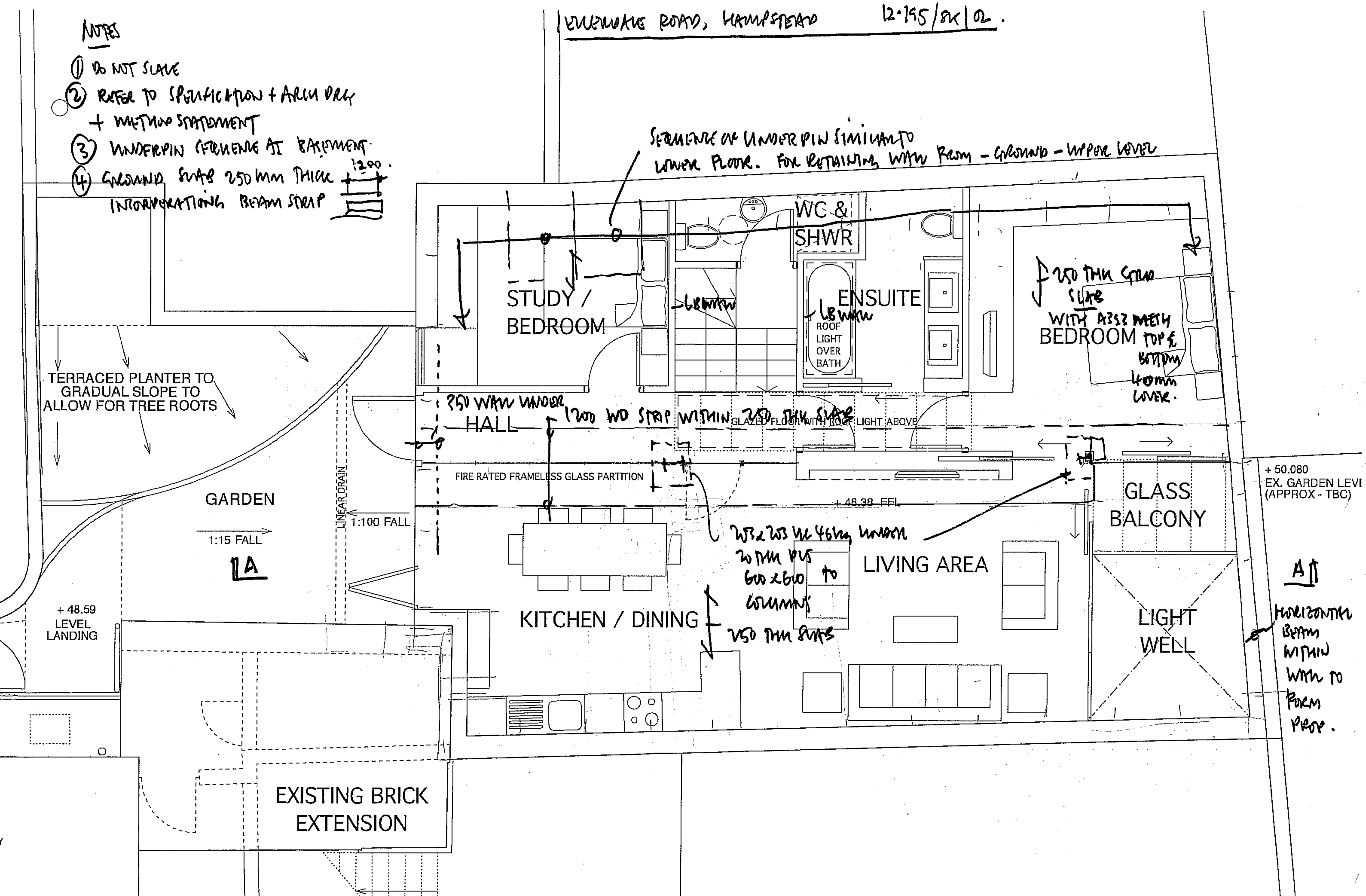
5. The maximum width of any pin will be 1M. Each underpin shall be dug with both mechanical and hand digging and when hand digging is taking place, a trench box will be installed to protect the operative.

6. The central area of excavation shall not be carried out until the perimeter underpinning and retaining walls have been completed.

7. The central section will now be excavated and this will be done in 3 sections to avoid any slippage. Lateral Mabey bracing struts or similar will be installed to counter this. The Re-inforced mesh will be prepared and laid and will be overlapped to ensure integrity. The basement slab is cast as detailed by the structural engineer and will be cast in 3 sections, with only 1 section being excavated and poured at a time.

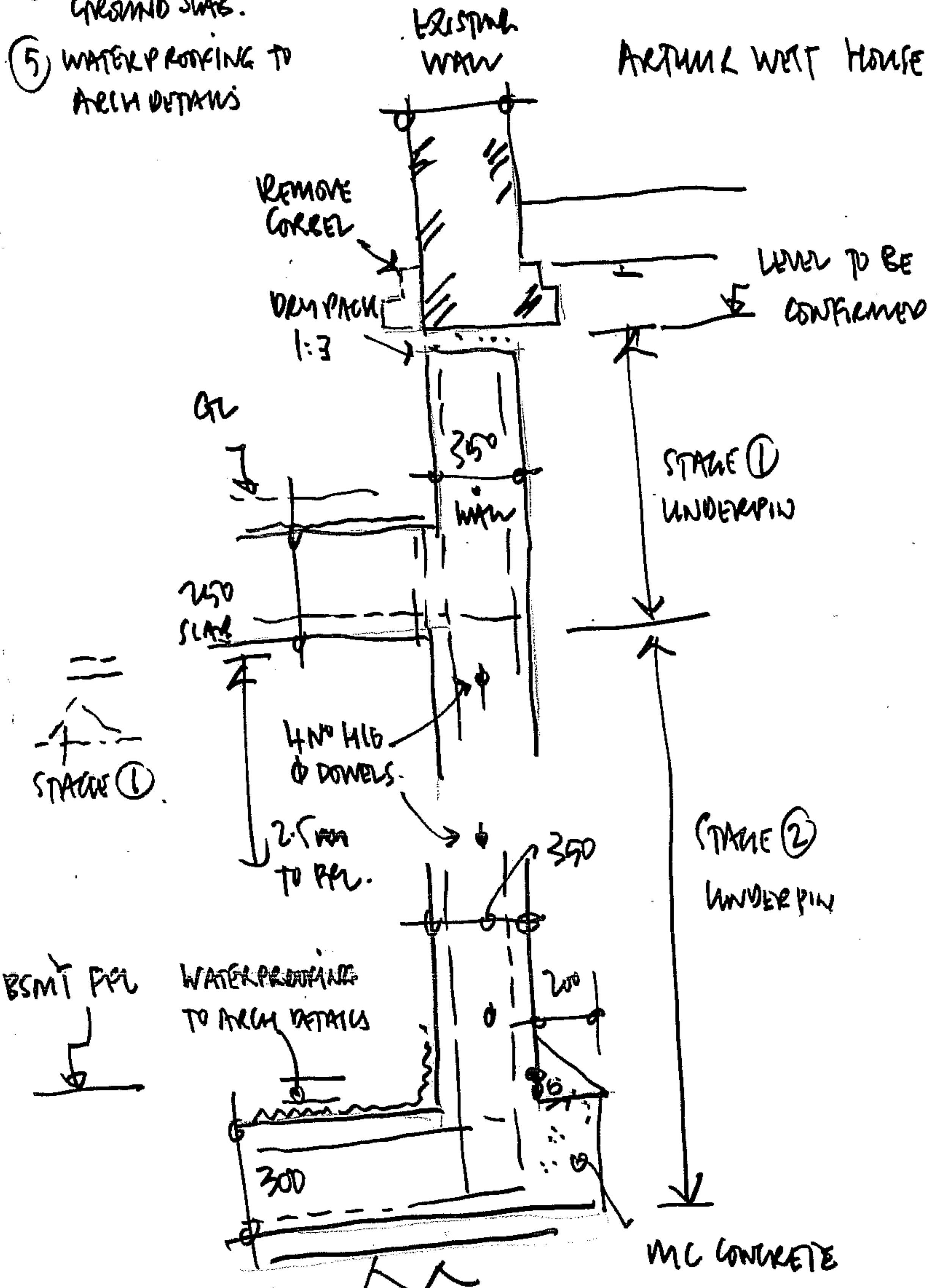
8. Once the concrete slab has been poured, the struts will be removed 48 hours after the pour and the sequence is followed again for the next section.

9. END.



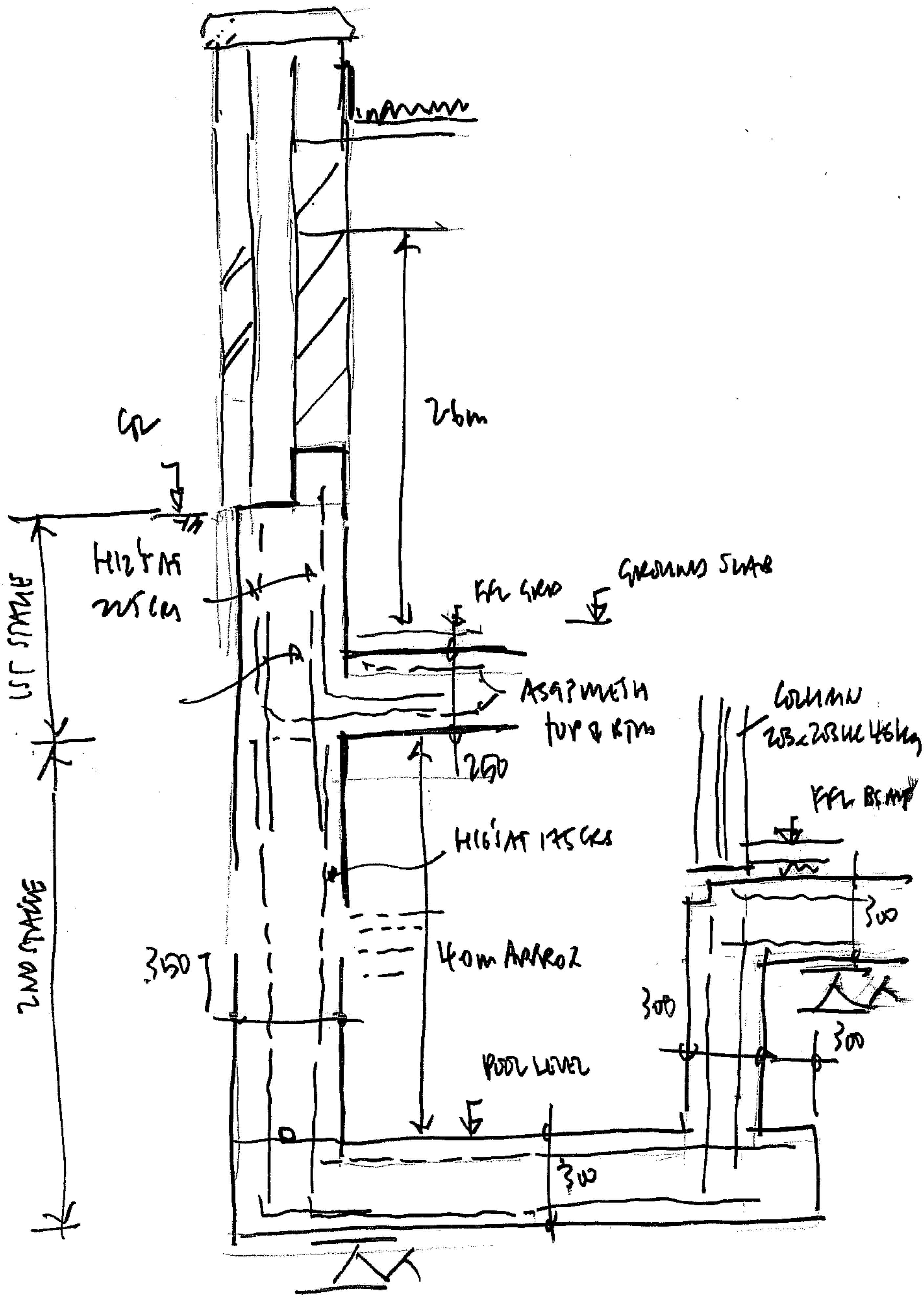
Ground Level Plan 1:50 Scale

1. DO NOT SCALE
2. REFER TO ARCHITECTS/SPECIFICATION
3. LEVELS TO ARCH DETAILS
4. STAGE UNDERPIN TO FORM GROUND SLAB.
5. WATERPROOFING TO ARCH DETAILS



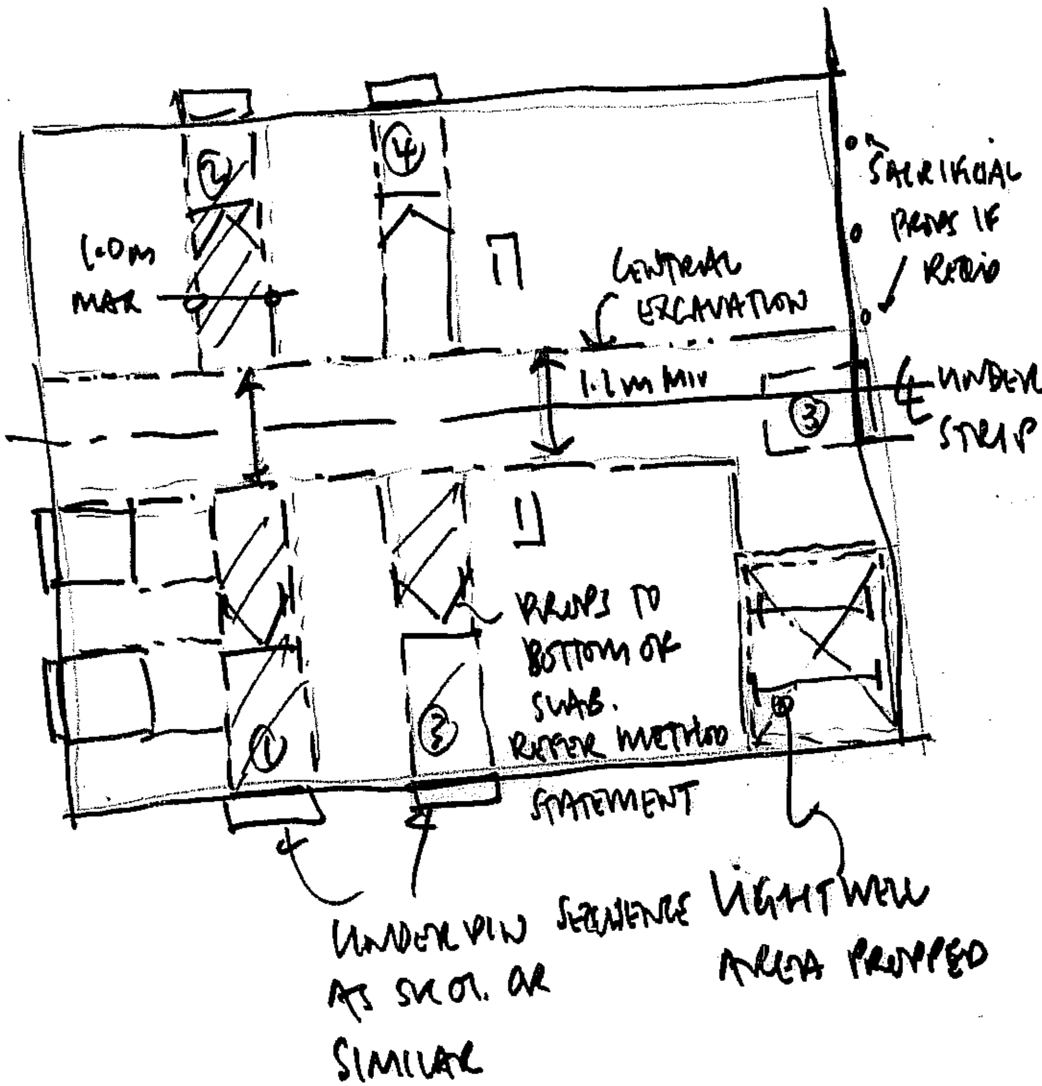
SECTION 1-1 (NTS)

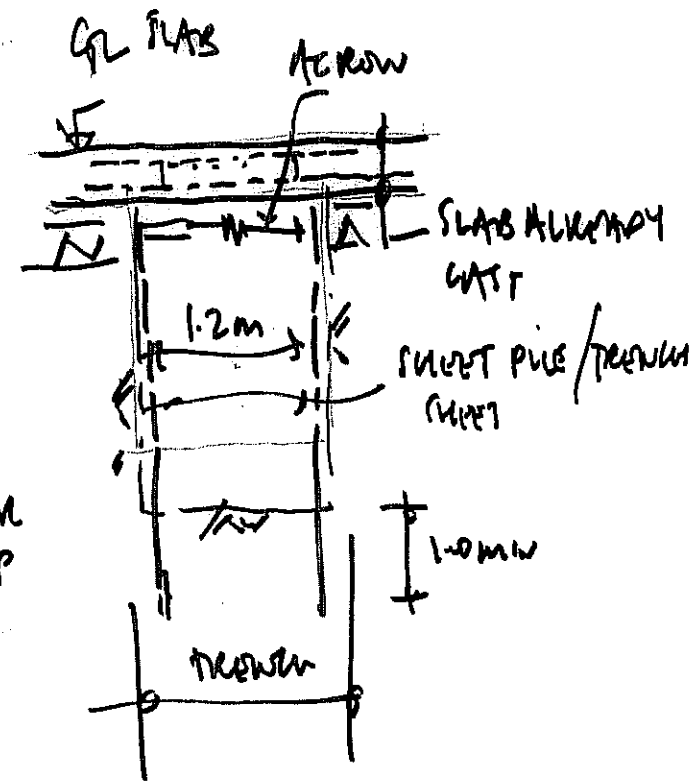
1 ELLERDALE ROAD, HAMPSTEAD 12.195/SK/07



SECTION 2-2

1 ELLERDALE ROAD HAMPSTEAD 12195/SK/05





CENTRAL TRENCH TO PHASE 2 UNDERPIN

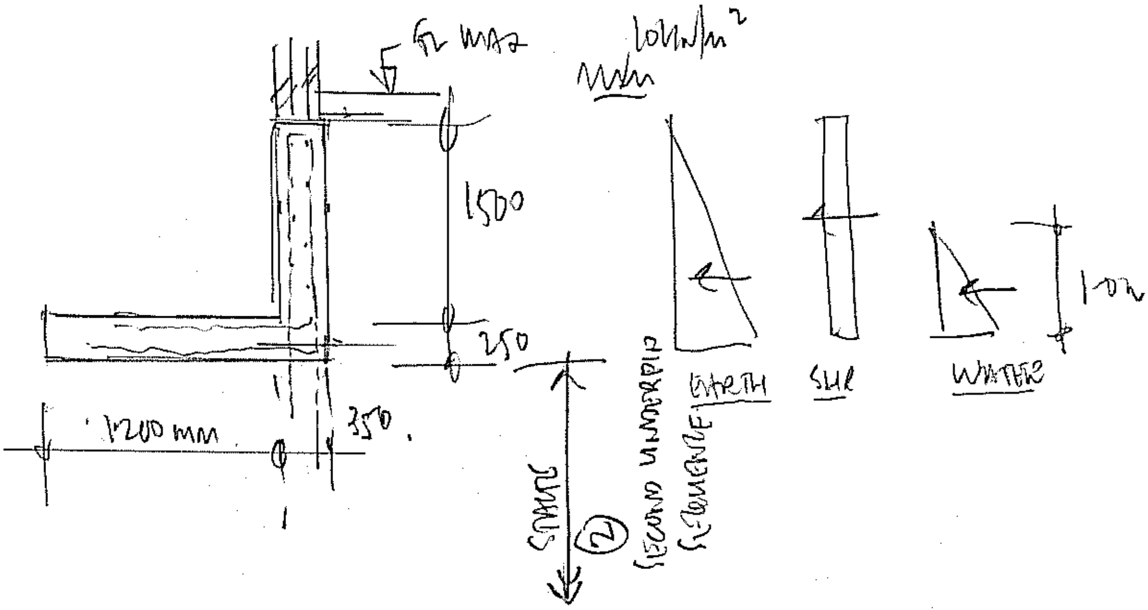
PROPOSED CENTRAL TRENCH TEMP UNDERPIN

AND DESIGNS LTD 90 MEADROW, GODALMING SURREY, GU7 3HY Tel: 01483 418 140 Fax: 01483 421 304 email: info@anddesigns.co.uk	Project 1 ELLERDALE ROAD HAMPSTEAD NW3				Job Ref. 12.195	
	Section PROPOSED BASEMENT				Sheet no./rev. 1	
	Calc. by J	Date 30/10/2012	Chk'd by	Date	App'd by	Date

STAGE ①

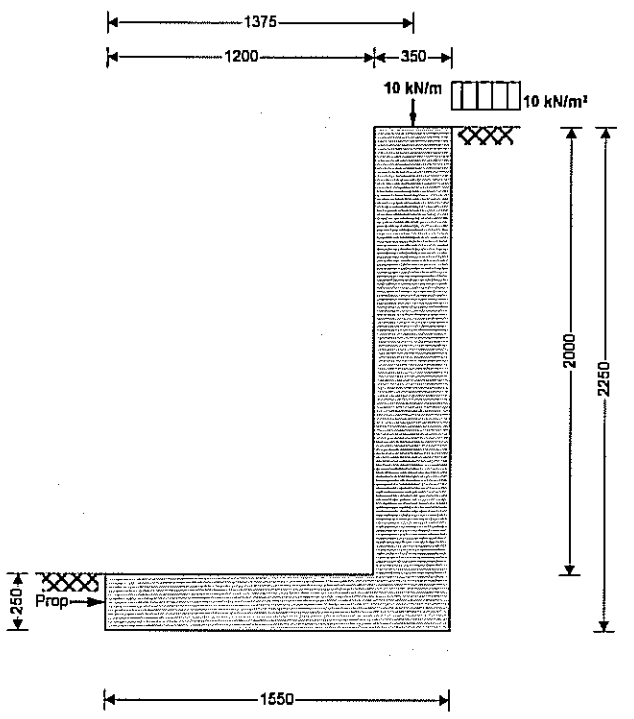
DESIGN FOR 1.5m HIGH RETAINING WALL

Kpa 25 (FILL IN SHORT TERM)



RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at base**
Height of retaining wall stem	h_{stem} = **2000** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **1200** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1550** mm
Thickness of base	t_{base} = **250** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1100** mm
Thickness of downstand	t_{ds} = **250** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2250** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **0** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **0** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **2250** mm

Retained material details

Mobilisation factor	M = **1.2**

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	Section PROPOSED BASEMENT				Sheet no./rev. 3	
	Calc. by J	Date 30/10/2012	Chk'd by	Date	App'd by	Date

Moist density of retained material $\gamma_m = \mathbf{6.5}$ kN/m^3

Saturated density of retained material $\gamma_s = \mathbf{13.0}$ kN/m^3

Design shear strength $\phi' = \mathbf{29.3}$ deg

Angle of wall friction $\delta = \mathbf{22.8}$ deg

Base material details

Moist density $\gamma_{mb} = \mathbf{6.5}$ kN/m^3

Design shear strength $\phi'_b = \mathbf{25.7}$ deg

Design base friction $\delta_b = \mathbf{19.8}$ deg

Allowable bearing pressure $P_{bearing} = \mathbf{40}$ kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.304}$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.741}$$

At-rest pressure

At-rest pressure for retained material $K_0 = 1 - \sin(\phi') = \mathbf{0.511}$

Loading details

Surcharge load on plan Surcharge = **10.0** kN/m^2

Applied vertical dead load on wall $W_{dead} = \mathbf{5.0}$ kN/m

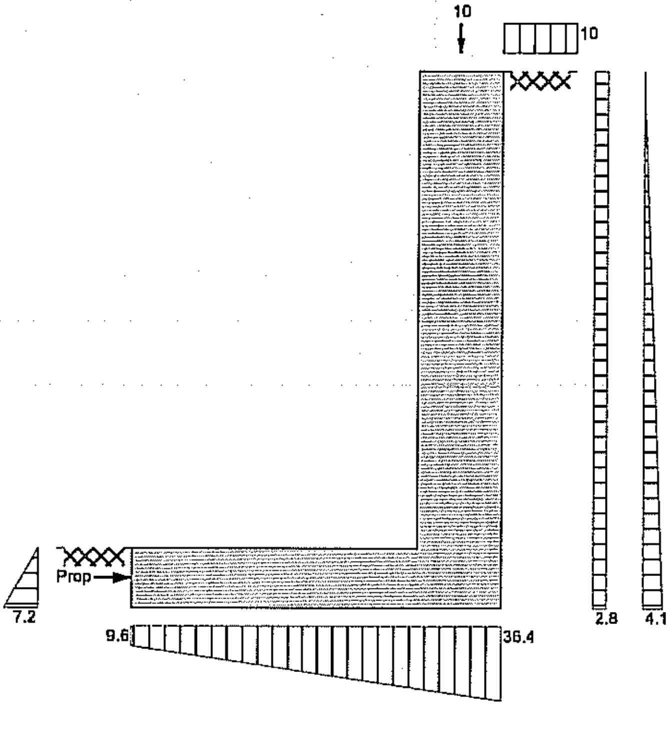
Applied vertical live load on wall $W_{live} = \mathbf{5.0}$ kN/m

Position of applied vertical load on wall $l_{load} = \mathbf{1375}$ mm

Applied horizontal dead load on wall $F_{dead} = \mathbf{0.0}$ kN/m

Applied horizontal live load on wall $F_{live} = \mathbf{0.0}$ kN/m

Height of applied horizontal load on wall $h_{load} = \mathbf{0}$ mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **16.5** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **9.1** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **10** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **35.7** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff}$ = **6.3** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2$ = **4.6** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a}$ = **10.9** kN/m

Calculate propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **0.9** kN/m

Propping force $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop} = **0.0** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **7.1** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **3.5** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a}$ = **10.6** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **22.7** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **7.1** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **6.9** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **36.7** kNm/m

Check bearing pressure

Design vertical live load $M_{live} = W_{live} \times l_{load}$ = **6.9** kNm/m

Total moment for bearing $M_{total} = M_{rest} - M_{ot} + M_{live}$ = **33** kNm/m

Total vertical reaction $R = W_{total}$ = **35.7** kN/m

Distance to reaction $x_{bar} = M_{total} / R$ = **925** mm

Eccentricity of reaction $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **150** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **9.6** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **36.4** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor γ_{f_d} = **1.4**

Live load factor γ_{f_l} = **1.6**

Earth and water pressure factor γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **23.1 kN/m**

Wall base $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **12.8 kN/m**

Applied vertical load $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **15 kN/m**

Total vertical load $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **50.9 kN/m**

Factored horizontal at-rest forces on wall

Surcharge $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times h_{eff}$ = **18.4 kN/m**

Moist backfill above water table $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **11.8 kN/m**

Total horizontal load $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f}$ = **30.1 kN/m**

Calculate propping force

Passive resistance of soil in front of wall $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **1.3** kN/m

Propping force $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop_f} = **13.4 kN/m**

Factored overturning moments

Surcharge $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **20.7 kNm/m**

Moist backfill above water table $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **8.8 kNm/m**

Total overturning moment $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f}$ = **29.5 kNm/m**

Restoring moments

Wall stem $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **31.8 kNm/m**

Wall base $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **9.9 kNm/m**

Design vertical load $M_{v\_f} = W_{v\_f} \times l_{load}$ = **20.6 kNm/m**

Total restoring moment $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **62.3 kNm/m**

Factored bearing pressure

Total moment for bearing $M_{total\_f} = M_{rest\_f} - M_{ot\_f}$ = **32.8 kNm/m**

Total vertical reaction $R_f = W_{total\_f}$ = **50.9 kN/m**

Distance to reaction $x_{bar\_f} = M_{total\_f} / R_f$ = **645** mm

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - x_{bar\_f})$ = **130** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **49.4 kN/m²**

Bearing pressure at heel $p_{heel\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **16.3 kN/m²**

Rate of change of base reaction $\text{rate} = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **21.35** kN/m²/m

Bearing pressure at stem / toe $p_{stem\_toe\_f} = \max(p_{toe\_f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2)$ = **23.8** kN/m²

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2)$ = **20.1** kN/m²

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (\text{rate} \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2)$ = **16.3** kN/m²

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Characteristic strength of concrete f_{cu} = **40** N/mm²

Characteristic strength of reinforcement f_y = **500** N/mm^2

Base details

Minimum area of reinforcement k = **0.13** %

Cover to reinforcement in toe c_{toe} = **50** mm

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{43.9}$ kN/m

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{9.9}$ kN/m

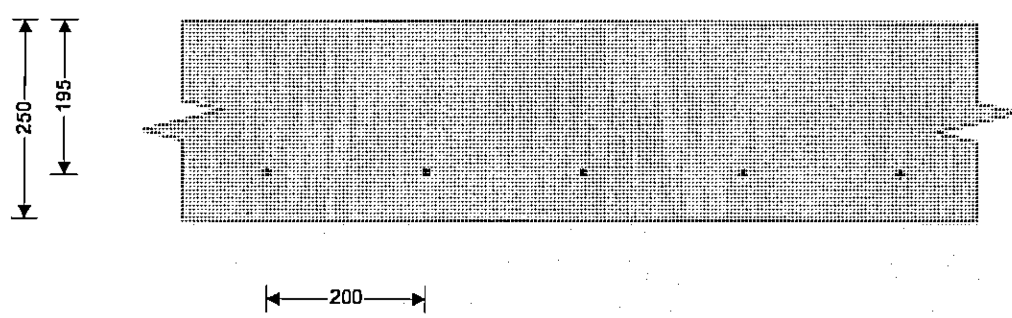
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{34}$ kN/m

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{37.5}$ kNm/m

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{7.8}$ kNm/m

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{29.6}$ kNm/m



Check toe in bending

Width of toe b = **1000** mm/m

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{195.0}$ mm

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.019}$

Compression reinforcement is not required

Lever arm $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

z_{toe} = **185** mm

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{368}$ mm^2/m

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{325}$ mm^2/m

Area of tension reinforcement required $A_{s\_toe\_req} = \mathrm{Max}(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{368}$ mm^2/m

Reinforcement provided **A393 mesh**

Area of reinforcement provided $A_{s\_toe\_prov}$ = **393** mm^2/m

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.174}$ N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 = \mathbf{5.000}$ N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_toe} = **0.519** N/mm^2

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

<u>Design of reinforced concrete retaining wall stem (BS 8002:1994)</u>

Material properties

Characteristic strength of concrete f_{cu} = **40** N/mm^2

Characteristic strength of reinforcement f_y = **500** N/mm^2

Wall details

Minimum area of reinforcement **k = 0.13 %**

Cover to reinforcement in stem c_{stem} = **50** mm

Cover to reinforcement in wall c_{wall} = **50** mm

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times$ Surcharge $\times (h_{eff} - t_{base} - d_{ds})$ = **16.3** kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2$ = **9.3** kN/m

Calculate shear for stem design

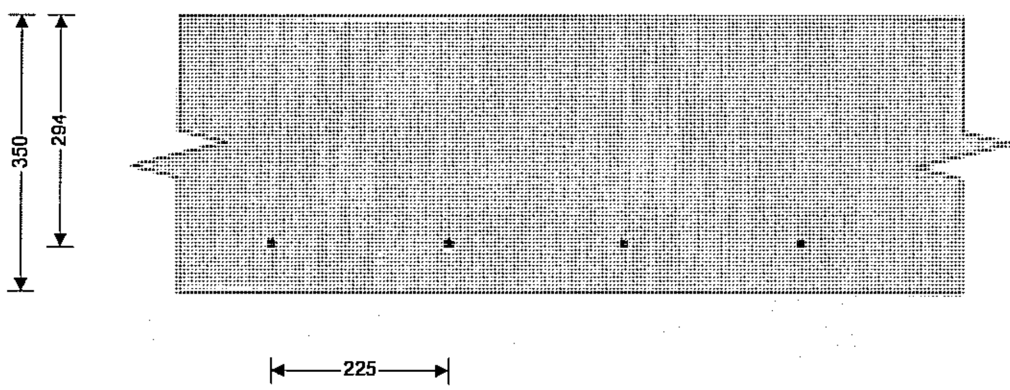
Shear at base of stem $V_{stem} = F_{s\_sur\_f} + F_{s\_m\_a\_f} - F_{prop\_f}$ = **12.2** kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times (h_{stem} + t_{base}) / 2$ = **18.4** kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3$ = **7.4** kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a}$ = **25.7** kNm/m



Check wall stem in bending

Width of wall stem b = **1000** mm/m

Depth of reinforcement $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2)$ = **294.0** mm

Constant $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu})$ = **0.007**

Compression reinforcement is not required

Lever arm $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$

z_{stem} = **279** mm

Area of tension reinforcement required $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem})$ = **212** mm^2/m

Minimum area of tension reinforcement $A_{s\_stem\_min} = k \times b \times t_{wall}$ = **455** mm^2/m

Area of tension reinforcement required $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min})$ = **455** mm^2/m

Reinforcement provided **12 mm dia.bars @ 225 mm centres**

Area of reinforcement provided $A_{s\_stem\_prov}$ = **503** mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress $v_{stem} = V_{stem} / (b \times d_{stem})$ = **0.042** N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2}$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress v_{c_stem} = **0.443** N/mm^2

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check retaining wall deflection

Basic span/effective depth ratio ratio_{bas} = **7**

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **301.7** N/mm^2

Modification factor $\text{factor}_{tens} = \min(0.55 + (477\text{ N/mm}^2 - f_s)/(120 \times (0.9\text{ N/mm}^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.77**

Maximum span/effective depth ratio $\text{ratio}_{max} = \text{ratio}_{bas} \times \text{factor}_{tens}$ = **12.39**

Actual span/effective depth ratio $\text{ratio}_{act} = h_{stem} / d_{stem}$ = **6.80**

PASS - Span to depth ratio is acceptable

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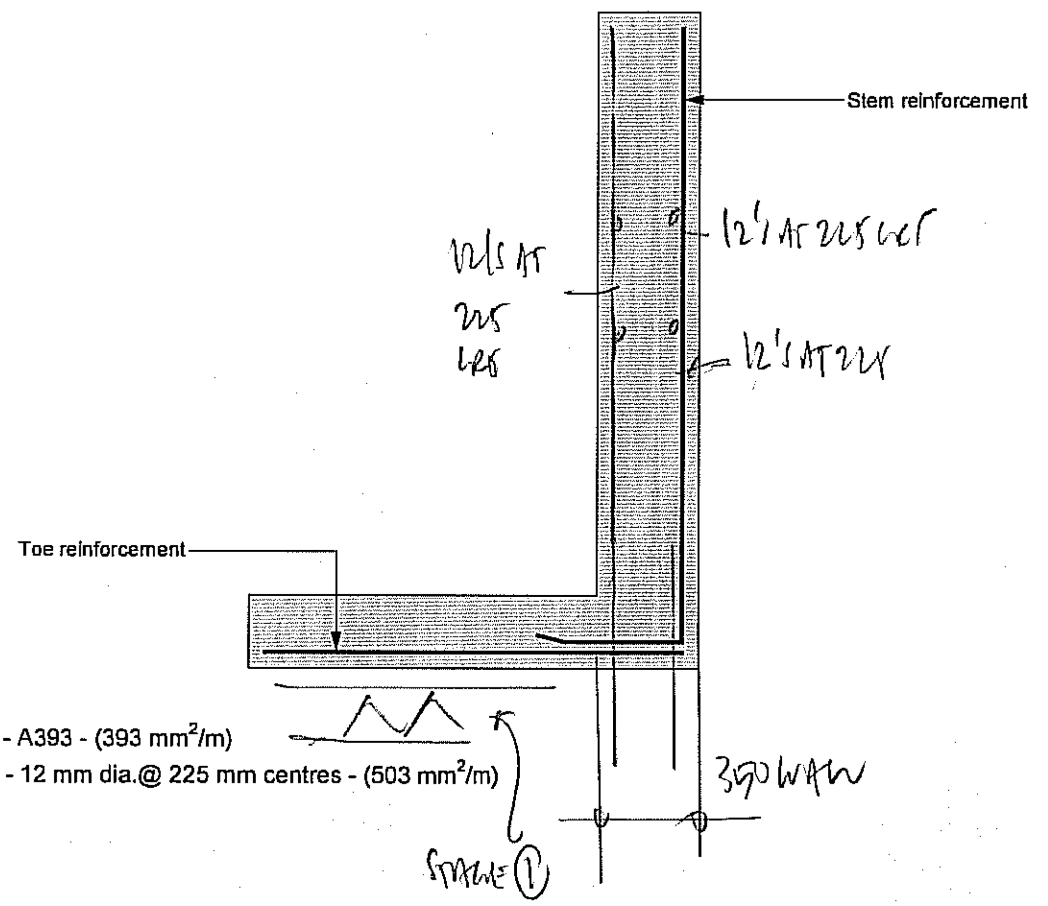
	Section				Sheet no./rev.	
	PROPOSED BASEMENT				9	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	J	30/10/2012				

Indicative retaining wall reinforcement diagram



Toe mesh - A393 - (393 mm²/m)

Stem bars - 12 mm dia.@ 225 mm centres - (503 mm²/m)

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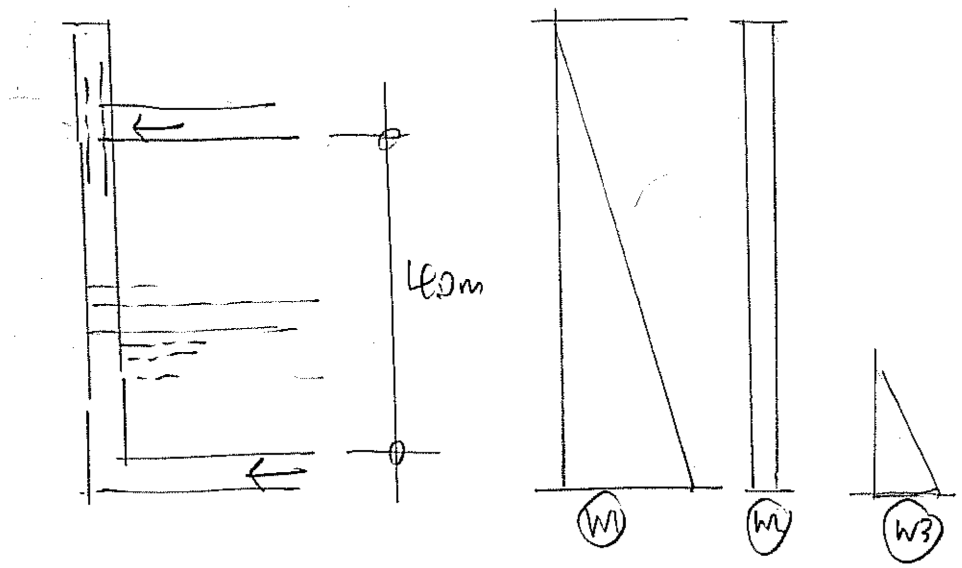
2ND STAGE DESIGN

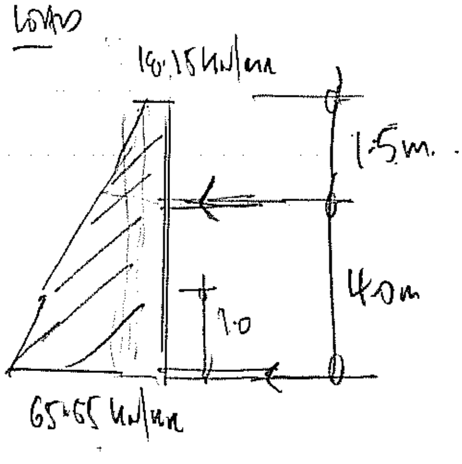


W1 = 5.5 x 18 x 0.33 = 37.7 kN/m²

W2 = 5.5 x 10 x 0.33 = 18.15 kN/m²

W3 = 1.0 x 10 = 10 kN/m²

TOTAL LOAD



DESIGN AS CONTINUOUS BEAM

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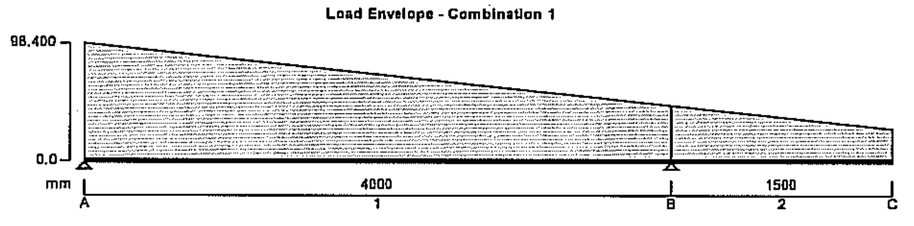
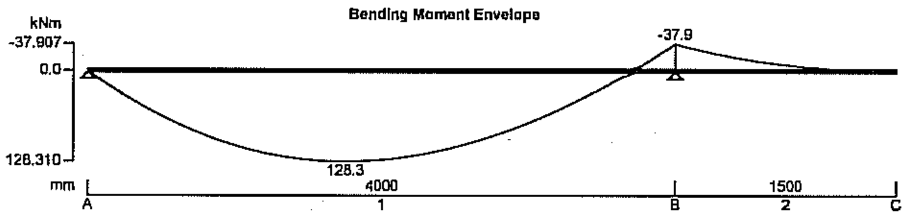
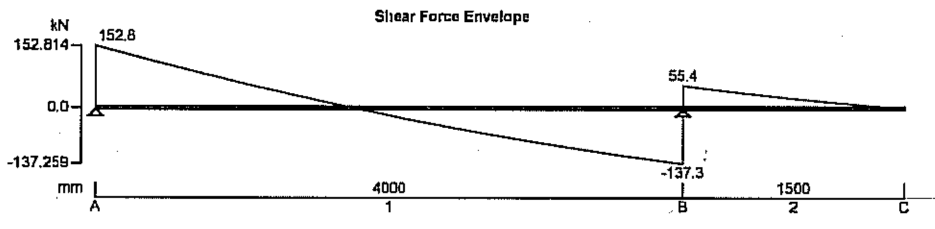
RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.10







Support conditions

Support A	Vertically restrained	
	Rotationally free	
Support B	Vertically restrained	
	Rotationally free	
Support C	Vertically free	
	Rotationally free	

Applied loading

Dead full VDL 65.6 kN/m to 18.15 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50
	Support B	Dead × 1.50
		Imposed × 1.50

	Span 2	Dead × 1.50
		Imposed × 1.50
	Support C	Dead × 1.50
		Imposed × 1.50

Analysis results

Maximum moment support A	M_{A_max} = **0** kNm	M_{A_red} = **0** kNm
Maximum moment span 1 at 1756 mm	M_{s1_max} = **128** kNm	M_{s1_red} = **128** kNm
Maximum moment support B	M_{B_max} = **-38** kNm	M_{B_red} = **-38** kNm
Maximum moment span 2 at support	M_{s2_max} = **-38** kNm	M_{s2_red} = **-38** kNm
Maximum moment support C	M_{C_max} = **0** kNm	M_{C_red} = **0** kNm
Maximum shear support A	V_{A_max} = **153** kN	V_{A_red} = **153** kN
Maximum shear support A span 1 at 282 mm	$V_{A\_s1\_max}$ = **126** kN	$V_{A\_s1\_red}$ = **126** kN
Maximum shear support B	V_{B_max} = **-137** kN	V_{B_red} = **-137** kN
Maximum shear support B span 1 at 3700 mm	$V_{B\_s1\_max}$ = **-123** kN	$V_{B\_s1\_red}$ = **-123** kN
Maximum shear support B span 2 at 300 mm	$V_{B\_s2\_max}$ = **42** kN	$V_{B\_s2\_red}$ = **42** kN
Maximum shear support C	V_{C_max} = **0** kN	V_{C_red} = **-17** kN
Maximum shear support C span 2 at 1200 mm	$V_{C\_s2\_max}$ = **9** kN	$V_{C\_s2\_red}$ = **9** kN
Maximum reaction at support A	R_A = **153** kN	
Unfactored dead load reaction at support A	R_{A_Dead} = **102** kN	
Maximum reaction at support B	R_B = **193** kN	
Unfactored dead load reaction at support B	R_{B_Dead} = **128** kN	
Maximum reaction at support C	R_C = **0** kN	
Unfactored dead load reaction at support C	R_{C_Dead} = **0** kN	

Rectangular section details

Section width	b = **1000** mm
Section depth	h = **350** mm

Concrete details

Concrete strength class	**C32/40**
Characteristic compressive cube strength	f_{cu} = **40** N/mm^2
Modulus of elasticity of concrete	E_c = 20kN/mm^2 + 200 × f_{cu} = **28000** N/mm^2
Maximum aggregate size	h_{agg} = **20** mm

Reinforcement details

Characteristic yield strength of reinforcement	f_y = **500** N/mm^2
Characteristic yield strength of shear reinforcement	f_{yv} = **500** N/mm^2

Nominal cover to reinforcement

Nominal cover to top reinforcement	c_{nom_t} = **50** mm
Nominal cover to bottom reinforcement	c_{nom_b} = **50** mm
Nominal cover to side reinforcement	c_{nom_s} = **50** mm

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DESIGN OF WALL REINFT

MAX SPAN MOMENT

$d = 350 - 50 - 10 - 5 = 285mm$.

$M/bd^2f_{cu} = 128.3 \times 10^6 / 10^3 \times 285^2 \times 40 = 0.039$. $a = 0.94$

$A_{st} = 128.3 \times 10^6 / 0.87 \times 500 \times 285 \times 0.94 = 1101\ mm^2/m$

USE H16's AT 175mm NF

USE H12' BARS AT 225 CRS DIST.

CANT MOMENT

$A_{st}\ 37.5 \times 10^6 / 0.87 \times 500 \times 285 \times 0.94 = 321\ mm^2/m$

USE H12's AT 225 OR MIN.

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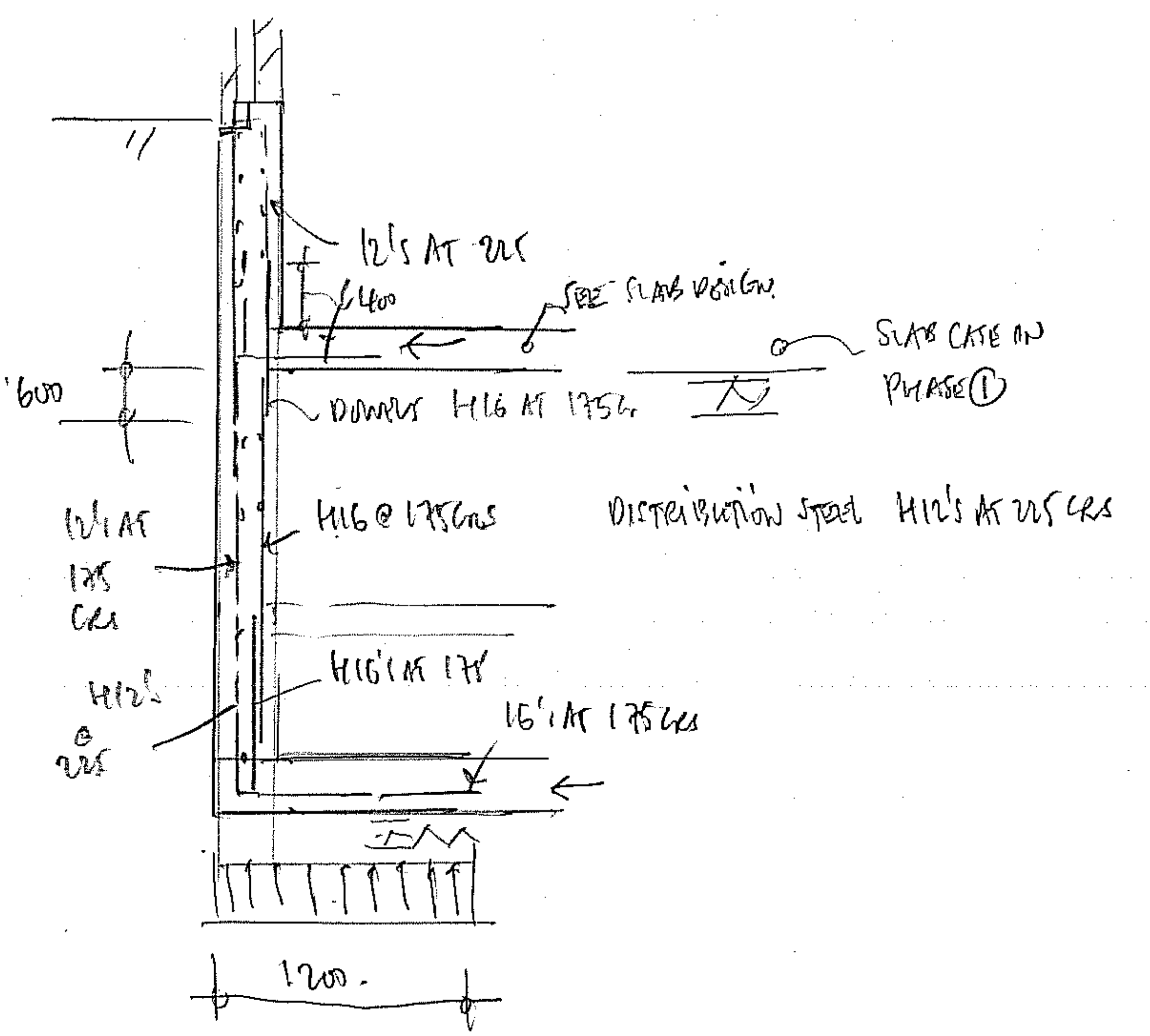
WALL REINF CONTD

CHECK SHEAR

$v/bd = 153 \times 10^3 / 285 \times 10^3 = 0.53\ N/mm^2$

$100A_s/bd = 114900 / 285 \times 10^3 = 0.4 \quad v_c = 0.55\ N/mm^2$

$0.53 \leq 0.55\ N/mm^2 \therefore OK$



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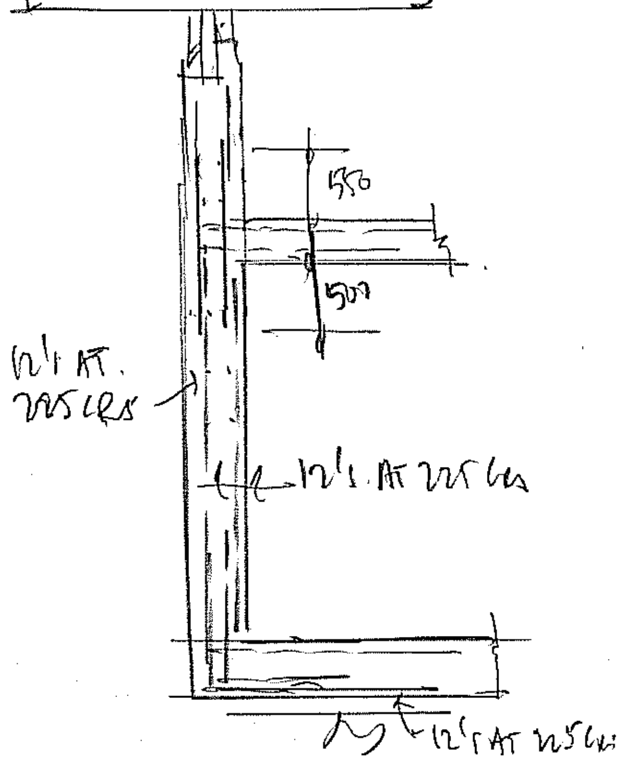
DESIGN OF RETAINING WALL. FOR 2.1m STEM.

[I.E. GROUND FLOOR LVL]



SECTION THRO' WALL

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	Section PROPOSED NEW BASEMENT				Sheet no./rev. 16	
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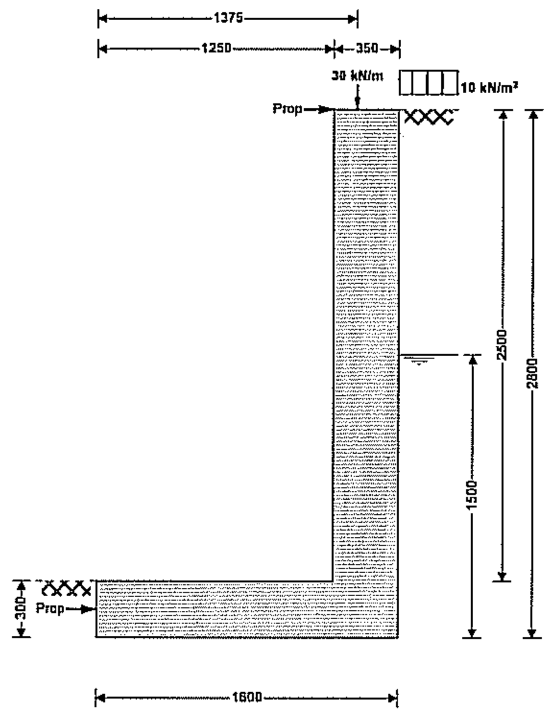
RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.06



Wall details

Retaining wall type	**Cantilever propped at both**
Height of retaining wall stem	h_{stem} = **2500** mm
Thickness of wall stem	t_{wall} = **350** mm
Length of toe	l_{toe} = **1250** mm
Length of heel	l_{heel} = **0** mm
Overall length of base	$l_{base} = l_{toe} + l_{heel} + t_{wall}$ = **1600** mm
Thickness of base	t_{base} = **300** mm
Depth of downstand	d_{ds} = **0** mm
Position of downstand	l_{ds} = **1150** mm
Thickness of downstand	t_{ds} = **300** mm
Height of retaining wall	$h_{wall} = h_{stem} + t_{base} + d_{ds}$ = **2800** mm
Depth of cover in front of wall	d_{cover} = **0** mm
Depth of unplanned excavation	d_{exc} = **0** mm
Height of ground water behind wall	h_{water} = **1500** mm
Height of saturated fill above base	$h_{sat} = \max(h_{water} - t_{base} - d_{ds}, 0 \text{ mm})$ = **1200** mm
Density of wall construction	γ_{wall} = **23.6** kN/m^3
Density of base construction	γ_{base} = **23.6** kN/m^3
Angle of rear face of wall	α = **90.0** deg
Angle of soil surface behind wall	β = **0.0** deg
Effective height at virtual back of wall	$h_{eff} = h_{wall} + l_{heel} \times \tan(\beta)$ = **2800** mm

Retained material details

Mobilisation factor	M = **1.5**

Moist density of retained material	γ_m = **18.0** kN/m^3
Saturated density of retained material	γ_s = **21.0** kN/m^3
Design shear strength	ϕ' = **24.2** deg
Angle of wall friction	δ = **0.0** deg

Base material details

Moist density	γ_{mb} = **18.0** kN/m^3
Design shear strength	ϕ'_b = **24.2** deg
Design base friction	δ_b = **18.6** deg
Allowable bearing pressure	$P_{bearing}$ = **100** kN/m^2

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))}]^2) = \mathbf{0.419}$$

Passive pressure coefficient for base material

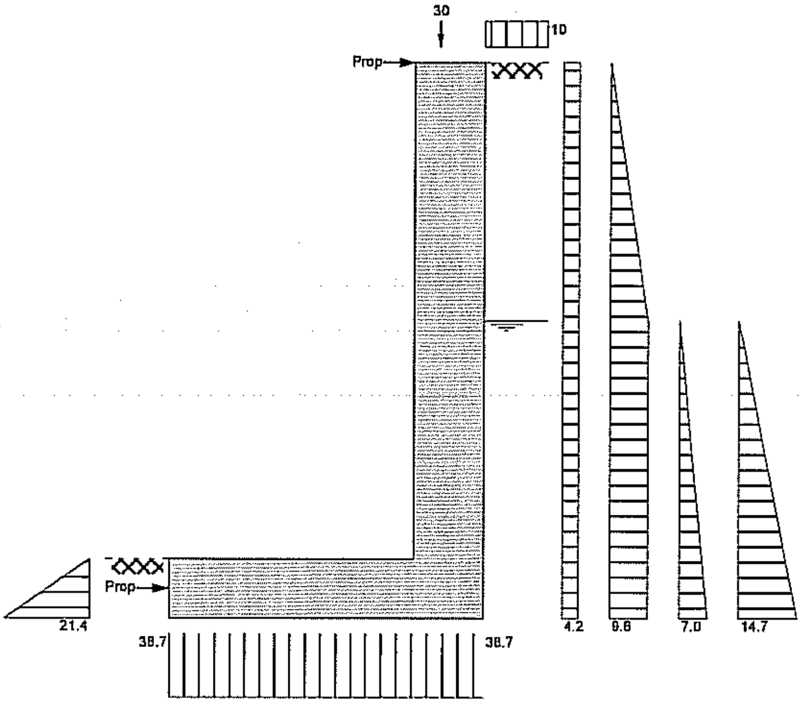
$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))}]^2) = \mathbf{4.187}$$

At-rest pressure

At-rest pressure for retained material	$K_0 = 1 - \sin(\phi')$ = **0.590**

Loading details

Surcharge load on plan	Surcharge = **10.0** kN/m^2
Applied vertical dead load on wall	W_{dead} = **20.0** kN/m
Applied vertical live load on wall	W_{live} = **10.0** kN/m
Position of applied vertical load on wall	l_{load} = **1375** mm
Applied horizontal dead load on wall	F_{dead} = **0.0** kN/m
Applied horizontal live load on wall	F_{live} = **0.0** kN/m
Height of applied horizontal load on wall	h_{load} = **0** mm



Loads shown in kN/m, pressures shown in kN/m^2

Vertical forces on wall

Wall stem $w_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall}$ = **20.7** kN/m

Wall base $w_{base} = l_{base} \times t_{base} \times \gamma_{base}$ = **11.3** kN/m

Applied vertical load $W_v = W_{dead} + W_{live}$ = **30** kN/m

Total vertical load $W_{total} = w_{wall} + w_{base} + W_v$ = **62** kN/m

Horizontal forces on wall

Surcharge $F_{sur} = K_a \times \text{Surcharge} \times h_{eff}$ = **11.7** kN/m

Moist backfill above water table $F_{m\_a} = 0.5 \times K_a \times \gamma_m \times (h_{eff} - h_{water})^2$ = **6.4** kN/m

Moist backfill below water table $F_{m\_b} = K_a \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **14.7** kN/m

Saturated backfill $F_s = 0.5 \times K_a \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **5.3** kN/m

Water $F_{water} = 0.5 \times h_{water}^2 \times \gamma_{water}$ = **11** kN/m

Total horizontal load $F_{total} = F_{sur} + F_{m\_a} + F_{m\_b} + F_s + F_{water}$ = **49.1** kN/m

Calculate total propping force

Passive resistance of soil in front of wall $F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **3.2** kN/m

Propping force $F_{prop} = \max(F_{total} - F_p - (W_{total} - W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop} = **28.4** kN/m

Overturning moments

Surcharge $M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **16.4** kNm/m

Moist backfill above water table $M_{m\_a} = F_{m\_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **12.3** kNm/m

Moist backfill below water table $M_{m\_b} = F_{m\_b} \times (h_{water} - 2 \times d_{ds}) / 2$ = **11** kNm/m

Saturated backfill $M_s = F_s \times (h_{water} - 3 \times d_{ds}) / 3$ = **2.6** kNm/m

Water $M_{water} = F_{water} \times (h_{water} - 3 \times d_{ds}) / 3$ = **5.5** kNm/m

Total overturning moment $M_{ot} = M_{sur} + M_{m\_a} + M_{m\_b} + M_s + M_{water}$ = **47.9** kNm/m

Restoring moments

Wall stem $M_{wall} = w_{wall} \times (l_{toe} + t_{wall} / 2)$ = **29.4** kNm/m

Wall base $M_{base} = w_{base} \times l_{base} / 2$ = **9.1** kNm/m

Design vertical dead load $M_{dead} = W_{dead} \times l_{load}$ = **27.5** kNm/m

Total restoring moment $M_{rest} = M_{wall} + M_{base} + M_{dead}$ = **66** kNm/m

Check bearing pressure

Total vertical reaction $R = W_{total}$ = **62.0** kN/m

Distance to reaction $x_{bar} = l_{base} / 2$ = **800** mm

Eccentricity of reaction $e = \text{abs}((l_{base} / 2) - x_{bar})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe $p_{toe} = (R / l_{base}) - (6 \times R \times e / l_{base}^2)$ = **38.7** kN/m^2

Bearing pressure at heel $p_{heel} = (R / l_{base}) + (6 \times R \times e / l_{base}^2)$ = **38.7** kN/m^2

PASS - Maximum bearing pressure is less than allowable bearing pressure

Calculate propping forces to top and base of wall

Propping force to top of wall

$F_{prop\_top} = (M_{ot} - M_{rest} + R \times l_{base} / 2 - F_{prop} \times t_{base} / 2) / (h_{stem} + t_{base} / 2)$ = **10.272** kN/m

Propping force to base of wall $F_{prop\_base} = F_{prop} - F_{prop\_top}$ = **18.101** kN/m

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.06

Ultimate limit state load factors

Dead load factor — γ_{f_d} = **1.4**

Live load factor — γ_{f_l} = **1.6**

Earth and water pressure factor — γ_{f_e} = **1.4**

Factored vertical forces on wall

Wall stem — $w_{wall\_f} = \gamma_{f\_d} \times h_{stem} \times t_{wall} \times \gamma_{wall}$ = **28.9** kN/m

Wall base — $w_{base\_f} = \gamma_{f\_d} \times l_{base} \times t_{base} \times \gamma_{base}$ = **15.9** kN/m

Applied vertical load — $W_{v\_f} = \gamma_{f\_d} \times W_{dead} + \gamma_{f\_l} \times W_{live}$ = **44** kN/m

Total vertical load — $W_{total\_f} = w_{wall\_f} + w_{base\_f} + W_{v\_f}$ = **88.8** kN/m

Factored horizontal at-rest forces on wall

Surcharge — $F_{sur\_f} = \gamma_{f\_l} \times K_0 \times Surcharge \times h_{eff}$ = **26.4** kN/m

Moist backfill above water table — $F_{m\_a\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2$ = **12.6** kN/m

Moist backfill below water table — $F_{m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - h_{water}) \times h_{water}$ = **29** kN/m

Saturated backfill — $F_{s\_f} = \gamma_{f\_e} \times 0.5 \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{water}^2$ = **10.4** kN/m

Water — $F_{water\_f} = \gamma_{f\_e} \times 0.5 \times h_{water}^2 \times \gamma_{water}$ = **15.5** kN/m

Total horizontal load — $F_{total\_f} = F_{sur\_f} + F_{m\_a\_f} + F_{m\_b\_f} + F_{s\_f} + F_{water\_f}$ = **93.8** kN/m

Calculate total propping force

Passive resistance of soil in front of wall — $F_{p\_f} = \gamma_{f\_e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb}$ = **4.5** kN/m

Propping force — $F_{prop\_f} = \max(F_{total\_f} - F_{p\_f} - (W_{total\_f} - \gamma_{f\_l} \times W_{live}) \times \tan(\delta_b), 0 \text{ kN/m})$

F_{prop_f} = **64.9** kN/m

Factored overturning moments

Surcharge — $M_{sur\_f} = F_{sur\_f} \times (h_{eff} - 2 \times d_{ds}) / 2$ = **37** kNm/m

Moist backfill above water table — $M_{m\_a\_f} = F_{m\_a\_f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3$ = **24.3** kNm/m

Moist backfill below water table — $M_{m\_b\_f} = F_{m\_b\_f} \times (h_{water} - 2 \times d_{ds}) / 2$ = **21.7** kNm/m

Saturated backfill — $M_{s\_f} = F_{s\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **5.2** kNm/m

Water — $M_{water\_f} = F_{water\_f} \times (h_{water} - 3 \times d_{ds}) / 3$ = **7.7** kNm/m

Total overturning moment — $M_{ot\_f} = M_{sur\_f} + M_{m\_a\_f} + M_{m\_b\_f} + M_{s\_f} + M_{water\_f}$ = **96** kNm/m

Restoring moments

Wall stem — $M_{wall\_f} = w_{wall\_f} \times (l_{toe} + t_{wall} / 2)$ = **41.2** kNm/m

Wall base — $M_{base\_f} = w_{base\_f} \times l_{base} / 2$ = **12.7** kNm/m

Design vertical load — $M_{v\_f} = W_{v\_f} \times l_{load}$ = **60.5** kNm/m

Total restoring moment — $M_{rest\_f} = M_{wall\_f} + M_{base\_f} + M_{v\_f}$ = **114.4** kNm/m

Factored bearing pressure

Total vertical reaction — $R_f = W_{total\_f}$ = **88.8** kN/m

Distance to reaction — $x_{bar\_f} = l_{base} / 2$ = **800** mm

Eccentricity of reaction — $e_f = abs((l_{base} / 2) - x_{bar\_f})$ = **0** mm

Reaction acts within middle third of base

Bearing pressure at toe — $p_{toe\_f} = (R_f / l_{base}) - (6 \times R_f \times e_f / l_{base}^2)$ = **55.5** kN/m²

Bearing pressure at heel — $p_{heel\_f} = (R_f / l_{base}) + (6 \times R_f \times e_f / l_{base}^2)$ = **55.5** kN/m²

Rate of change of base reaction — $rate = (p_{toe\_f} - p_{heel\_f}) / l_{base}$ = **0.00** kN/m²/m

Bearing pressure at stem / toe — $p_{stem\_toe\_f} = \max(p_{toe\_f} - (rate \times l_{toe}), 0 \text{ kN/m}^2)$ = **55.5** kN/m²

AND DESIGNS LTD 90 MEADROW, GODALMING SURREY, GU7 3HY Tel: 01483 418 140 Fax: 01483 421 304 email: info@anddesigns.co.uk	Project 1 ELLERDALE ROAD HAMPSTEAD NW3			Job Ref. 12.195	
	Section PROPOSED NEW BASEMENT			Sheet no./rev. 20	
	Calc. by J	Date 31/10/2012	Chk'd by	Date	App'd by	Date

Bearing pressure at mid stem $p_{stem\_mid\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall} / 2)), 0\ kN/m^2) = \mathbf{55.5}\ kN/m^2$

Bearing pressure at stem / heel $p_{stem\_heel\_f} = \max(p_{toe\_f} - (rate \times (l_{toe} + t_{wall})), 0\ kN/m^2) = \mathbf{55.5}\ kN/m^2$

Calculate propping forces to top and base of wall

Propping force to top of wall

$$F_{prop\_top\_f} = (M_{ot\_f} - M_{rest\_f} + R_f \times l_{base} / 2 - F_{prop\_f} \times t_{base} / 2) / (h_{stem} + t_{base} / 2) = \mathbf{16.180}\ kN/m$$

Propping force to base of wall $F_{prop\_base\_f} = F_{prop\_f} - F_{prop\_top\_f} = \mathbf{48.678}\ kN/m$

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{40}\ N/mm^2$

Characteristic strength of reinforcement $f_y = \mathbf{500}\ N/mm^2$

Base details

Minimum area of reinforcement $k = \mathbf{0.13}\ \%$

Cover to reinforcement in toe $c_{toe} = \mathbf{50}\ mm$

Calculate shear for toe design

Shear from bearing pressure $V_{toe\_bear} = (p_{toe\_f} + p_{stem\_toe\_f}) \times l_{toe} / 2 = \mathbf{69.4}\ kN/m$

Shear from weight of base $V_{toe\_wt\_base} = \gamma_{f\_d} \times \gamma_{base} \times l_{toe} \times t_{base} = \mathbf{12.4}\ kN/m$

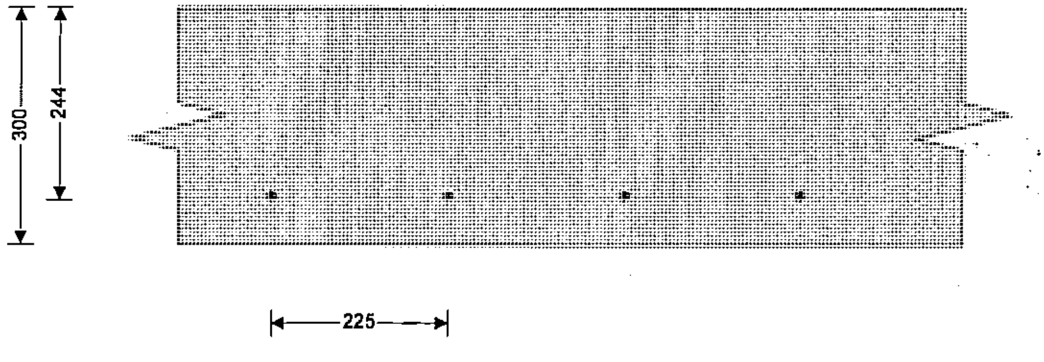
Total shear for toe design $V_{toe} = V_{toe\_bear} - V_{toe\_wt\_base} = \mathbf{57}\ kN/m$

Calculate moment for toe design

Moment from bearing pressure $M_{toe\_bear} = (2 \times p_{toe\_f} + p_{stem\_mid\_f}) \times (l_{toe} + t_{wall} / 2)^2 / 6 = \mathbf{56.3}\ kNm/m$

Moment from weight of base $M_{toe\_wt\_base} = (\gamma_{f\_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = \mathbf{10.1}\ kNm/m$

Total moment for toe design $M_{toe} = M_{toe\_bear} - M_{toe\_wt\_base} = \mathbf{46.3}\ kNm/m$



Check toe in bending

Width of toe $b = \mathbf{1000}\ mm/m$

Depth of reinforcement $d_{toe} = t_{base} - c_{toe} - (\phi_{toe} / 2) = \mathbf{244.0}\ mm$

Constant $K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = \mathbf{0.019}$

Compression reinforcement is not required

Lever arm $z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$

$z_{toe} = \mathbf{232}\ mm$

Area of tension reinforcement required $A_{s\_toe\_des} = M_{toe} / (0.87 \times f_y \times z_{toe}) = \mathbf{459}\ mm^2/m$

Minimum area of tension reinforcement $A_{s\_toe\_min} = k \times b \times t_{base} = \mathbf{390}\ mm^2/m$

Area of tension reinforcement required $A_{s\_toe\_req} = \max(A_{s\_toe\_des}, A_{s\_toe\_min}) = \mathbf{459}\ mm^2/m$

Reinforcement provided **12 mm dia.bars @ 225 mm centres**

Area of reinforcement provided $A_{s\_toe\_prov} = \mathbf{503}\ mm^2/m$

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress $v_{toe} = V_{toe} / (b \times d_{toe}) = \mathbf{0.233}$ N/mm^2

Allowable shear stress $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1 \text{ N/mm}^2)}, 5) \times 1 \text{ N/mm}^2 = \mathbf{5.000}$ N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress $v_{c\_toe} = \mathbf{0.494}$ N/mm^2

$v_{toe} < v_{c\_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall stem (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} = \mathbf{40}$ N/mm^2

Characteristic strength of reinforcement $f_y = \mathbf{500}$ N/mm^2

Wall details

Minimum area of reinforcement k = **0.13** %

Cover to reinforcement in stem $c_{stem} = \mathbf{50}$ mm

Cover to reinforcement in wall $c_{wall} = \mathbf{50}$ mm

Factored horizontal at-rest forces on stem

Surcharge $F_{s\_sur\_f} = \gamma_{f\_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = \mathbf{23.6}$ kN/m

Moist backfill above water table $F_{s\_m\_a\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = \mathbf{12.6}$ kN/m

Moist backfill below water table $F_{s\_m\_b\_f} = \gamma_{f\_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat}) \times h_{sat} = \mathbf{23.2}$ kN/m

Saturated backfill $F_{s\_s\_f} = 0.5 \times \gamma_{f\_e} \times K_0 \times (\gamma_s - \gamma_{water}) \times h_{sat}^2 = \mathbf{6.7}$ kN/m

Water $F_{s\_water\_f} = 0.5 \times \gamma_{f\_e} \times \gamma_{water} \times h_{sat}^2 = \mathbf{9.9}$ kN/m

Calculate shear for stem design

Surcharge $V_{s\_sur\_f} = 5 \times F_{s\_sur\_f} / 8 = \mathbf{14.8}$ kN/m

Moist backfill above water table $V_{s\_m\_a\_f} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - b_l^2) / (5 \times L^3) = \mathbf{5.9}$ kN/m

Moist backfill below water table $V_{s\_m\_b\_f} = F_{s\_m\_b\_f} \times (8 - (n^2 \times (4 - n))) / 8 = \mathbf{20.6}$ kN/m

Saturated backfill $V_{s\_s\_f} = F_{s\_s\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{6.3}$ kN/m

Water $V_{s\_water\_f} = F_{s\_water\_f} \times (1 - (a_l^2 \times ((5 \times L) - a_l) / (20 \times L^3))) = \mathbf{9.3}$ kN/m

Total shear for stem design $V_{stem} = V_{s\_sur\_f} + V_{s\_m\_a\_f} + V_{s\_m\_b\_f} + V_{s\_s\_f} + V_{s\_water\_f} = \mathbf{56.8}$ kN/m

Calculate moment for stem design

Surcharge $M_{s\_sur} = F_{s\_sur\_f} \times L / 8 = \mathbf{7.8}$ kNm/m

Moist backfill above water table $M_{s\_m\_a} = F_{s\_m\_a\_f} \times b_l \times ((5 \times L^2) - (3 \times b_l^2)) / (15 \times L^2) = \mathbf{4.7}$ kNm/m

Moist backfill below water table $M_{s\_m\_b} = F_{s\_m\_b\_f} \times a_l \times (2 - n)^2 / 8 = \mathbf{8.7}$ kNm/m

Saturated backfill $M_{s\_s} = F_{s\_s\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{2}$ kNm/m

Water $M_{s\_water} = F_{s\_water\_f} \times a_l \times ((3 \times a_l^2) - (15 \times a_l \times L) + (20 \times L^2)) / (60 \times L^2) = \mathbf{2.9}$ kNm/m

Total moment for stem design $M_{stem} = M_{s\_sur} + M_{s\_m\_a} + M_{s\_m\_b} + M_{s\_s} + M_{s\_water} = \mathbf{26.1}$ kNm/m

Calculate moment for wall design

Surcharge $M_{w\_sur} = 9 \times F_{s\_sur\_f} \times L / 128 = \mathbf{4.4}$ kNm/m

Moist backfill above water table $M_{w\_m\_a} = F_{s\_m\_a\_f} \times 0.577 \times b_l \times [(b_l^3 + 5 \times a_l \times L^2)/(5 \times L^3) - 0.577^2/3] = \mathbf{4}$ kNm/m

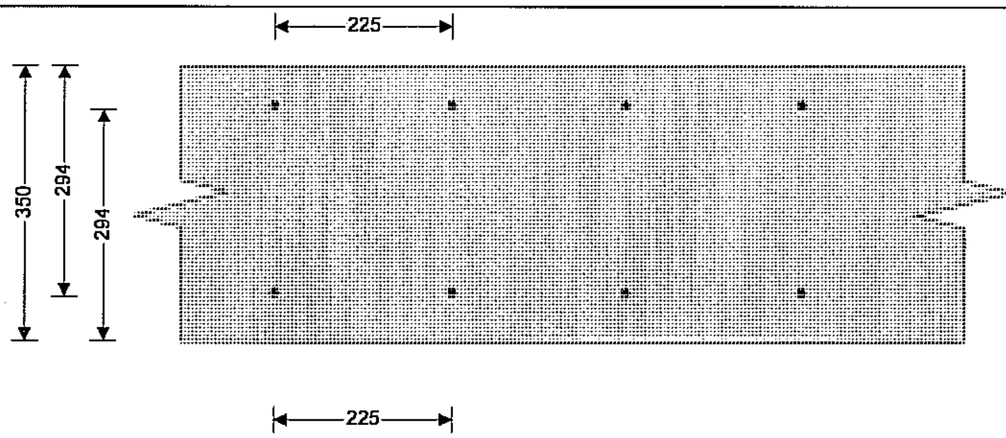
Moist backfill below water table $M_{w\_m\_b} = F_{s\_m\_b\_f} \times a_l \times [((8-n^2 \times (4-n))^2/16) - 4 + n \times (4-n)]/8 = \mathbf{3.6}$ kNm/m

Saturated backfill $M_{w\_s} = F_{s\_s\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l)/(20 \times L^3) - (x - b_l)^3/(3 \times a_l^2)] = \mathbf{0.6}$ kNm/m

Water $M_{w\_water} = F_{s\_water\_f} \times [a_l^2 \times x \times ((5 \times L) - a_l)/(20 \times L^3) - (x - b_l)^3/(3 \times a_l^2)] = \mathbf{0.9}$

kNm/m

Total moment for wall design $M_{wall} = M_{w\_sur} + M_{w\_m\_a} + M_{w\_m\_b} + M_{w\_s} + M_{w\_water} = \mathbf{13.5}$ kNm/m



Check wall stem in bending

Width of wall stem | $b = \mathbf{1000}$ mm/m

Depth of reinforcement | $d_{stem} = t_{wall} - c_{stem} - (\phi_{stem} / 2) = \mathbf{294.0}$ mm

Constant | $K_{stem} = M_{stem} / (b \times d_{stem}^2 \times f_{cu}) = \mathbf{0.008}$

Compression reinforcement is not required

Lever arm | $z_{stem} = \min(0.5 + \sqrt{(0.25 - (\min(K_{stem}, 0.225) / 0.9))}, 0.95) \times d_{stem}$

$z_{stem} = \mathbf{279}$ mm

Area of tension reinforcement required | $A_{s\_stem\_des} = M_{stem} / (0.87 \times f_y \times z_{stem}) = \mathbf{215}$ mm^2/m

Minimum area of tension reinforcement | $A_{s\_stem\_min} = k \times b \times t_{wall} = \mathbf{455}$ mm^2/m

Area of tension reinforcement required | $A_{s\_stem\_req} = \mathrm{Max}(A_{s\_stem\_des}, A_{s\_stem\_min}) = \mathbf{455}$ mm^2/m

Reinforcement provided | **12 mm dia.bars @ 225 mm centres**

Area of reinforcement provided | $A_{s\_stem\_prov} = \mathbf{503}$ mm^2/m

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress | $v_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.193}$ N/mm^2

Allowable shear stress | $v_{adm} = \min(0.8 \times \sqrt{(f_{cu} / 1\ \mathrm{N/mm^2})}, 5) \times 1\ \mathrm{N/mm^2} = \mathbf{5.000}$ N/mm^2

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress | $v_{c\_stem} = \mathbf{0.443}$ N/mm^2

$v_{stem} < v_{c\_stem}$ - No shear reinforcement required

Check mid height of wall in bending

Depth of reinforcement | $d_{wall} = t_{wall} - c_{wall} - (\phi_{wall} / 2) = \mathbf{294.0}$ mm

Constant | $K_{wall} = M_{wall} / (b \times d_{wall}^2 \times f_{cu}) = \mathbf{0.004}$

Compression reinforcement is not required

Lever arm | $z_{wall} = \mathrm{Min}(0.5 + \sqrt{(0.25 - (\min(K_{wall}, 0.225) / 0.9))}, 0.95) \times d_{wall}$

$z_{wall} = \mathbf{279}$ mm

Area of tension reinforcement required | $A_{s\_wall\_des} = M_{wall} / (0.87 \times f_y \times z_{wall}) = \mathbf{111}$ mm^2/m

Minimum area of tension reinforcement | $A_{s\_wall\_min} = k \times b \times t_{wall} = \mathbf{455}$ mm^2/m

Area of tension reinforcement required | $A_{s\_wall\_req} = \mathrm{Max}(A_{s\_wall\_des}, A_{s\_wall\_min}) = \mathbf{455}$ mm^2/m

Reinforcement provided | **12 mm dia.bars @ 225 mm centres**

Area of reinforcement provided | $A_{s\_wall\_prov} = \mathbf{503}$ mm^2/m

PASS - Reinforcement provided to the retaining wall at mid height is adequate

Check retaining wall deflection

Basic span/effective depth ratio | $ratio_{bas} = \mathbf{20}$

Design service stress $f_s = 2 \times f_y \times A_{s\_stem\_req} / (3 \times A_{s\_stem\_prov})$ = **301.7** N/mm^2

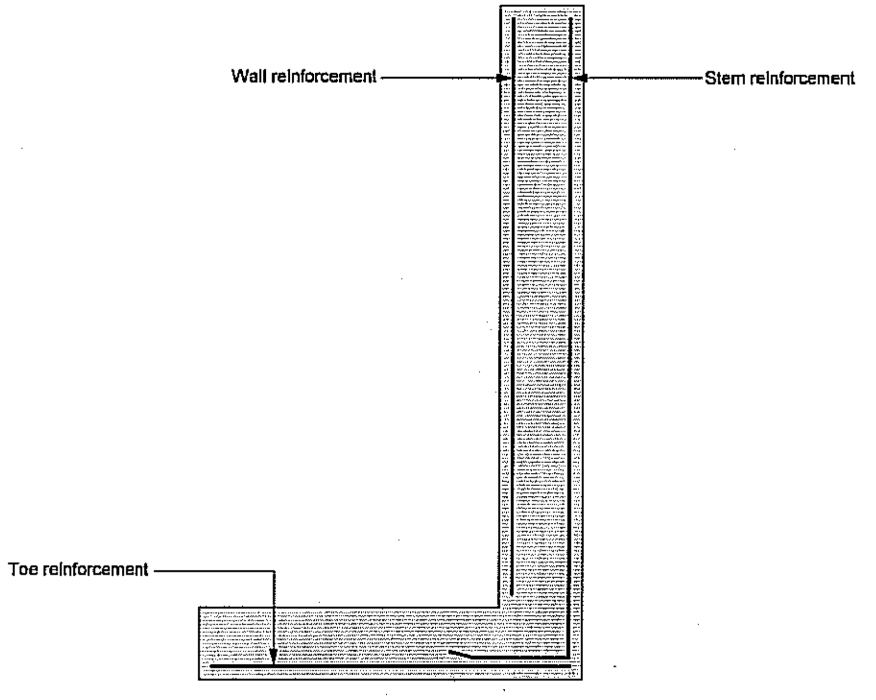
Modification factor $factor_{tens} = \min(0.55 + (477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + (M_{stem}/(b \times d_{stem}^2)))),2)$ = **1.77**

Maximum span/effective depth ratio $ratio_{max} = ratio_{bas} \times factor_{tens}$ = **35.31**

Actual span/effective depth ratio $ratio_{act} = h_{stem} / d_{stem}$ = **8.50**

PASS - Span to depth ratio is acceptable

Indicative retaining wall reinforcement diagram



Toe bars - 12 mm dia.@ 225 mm centres - (503 mm^2/m)
Wall bars - 12 mm dia.@ 225 mm centres - (503 mm^2/m)
Stem bars - 12 mm dia.@ 225 mm centres - (503 mm^2/m)

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DESIGN OF GROUND FLOOR SLAB (4.0m)

LOADING

Floor = 0.250 × 24 = 6.0 kN/m²

Screed 0.075 × 23 = 1.8

Ceiling 0.025 × 23 = 0.6

8.1 kN/m²

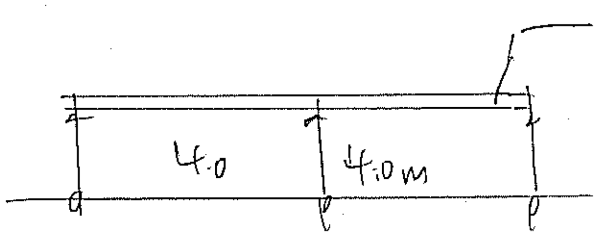
LL = 1.5 + 1.0 } 2.9 kN/m

PART 1.4

Ultimate loads = 8.1 × 1.4 = 11.3 kN/m²

Live load = 2.9 × 1.6 = 4.6 kN/m²

= 15.9 kN/m² × 4.0 = 63.6 kN



BM span = Wl/9 = 63.6 × 4/9 = 28.2 kNm (ULT)

BM sup Wl/10 63.6 × 4/10 = 25.5 kNm (ULT)

d = 250 − 40 − 5 = 205 mm

M/bd²fcu = 28.2 × 10⁶/10³ × 205² × 40 = 0.014   z = 0.94

As = 28.2 × 10⁶/0.87 × 500 × 0.94 × 205 = 336 mm²/m

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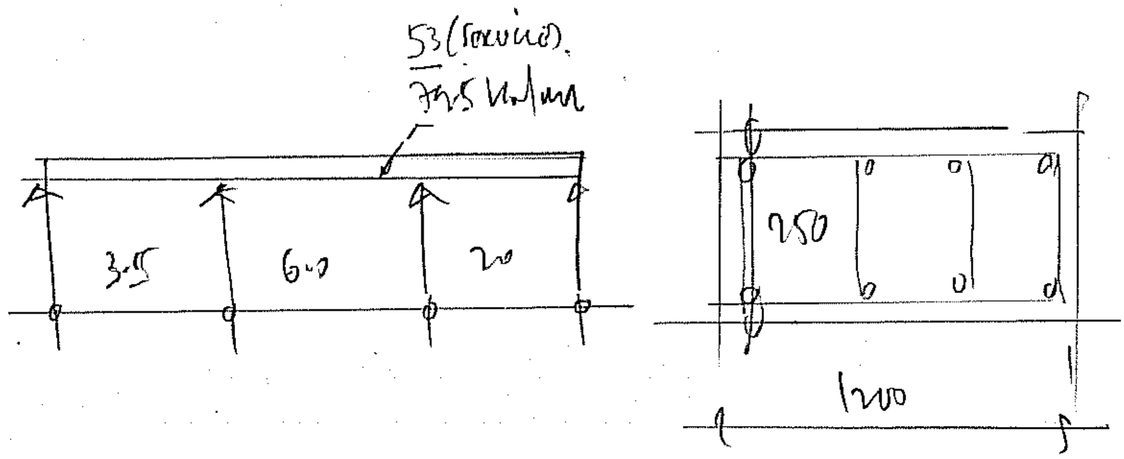
GROUND SLAB CONT'D.

USE A 393 MESH TOP & BOTTOM OF 250 THK
RC GROUND SLAB.

DESIGN OF GROUND BEAM SLABS 250 THK

LOADING

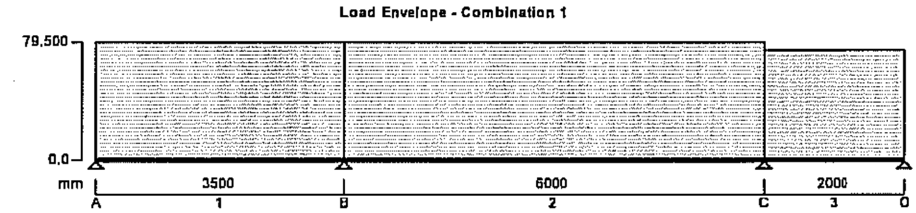
FROM SLABS = 15.9 (ULT) × 8 × 5/10 = 79.5 kN/m

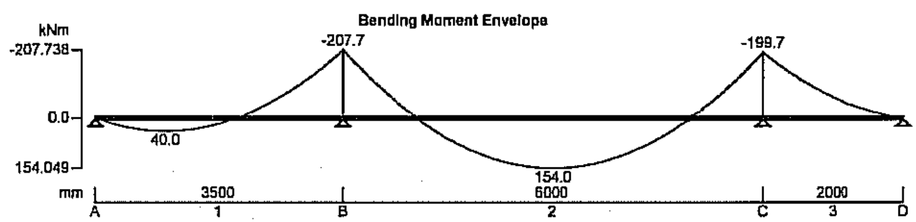


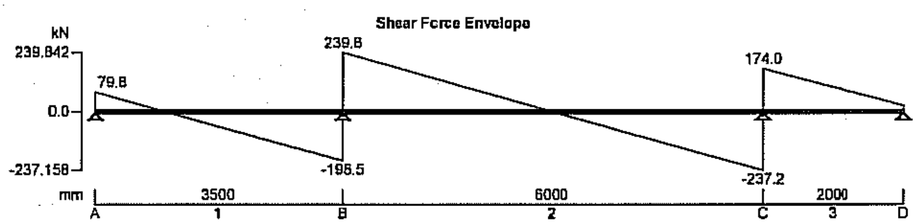
$d = 250 - 40 - 10 - 5 = \underline{195}mm = d.$

RC BEAM ANALYSIS & DESIGN BS8110

TEDDS calculation version 2.1.10







Support conditions

Support A	Vertically restrained	
	Rotationally free	
Support B	Vertically restrained	
	Rotationally free	
Support C	Vertically restrained	
	Rotationally free	
Support D	Vertically restrained	
	Rotationally free	

Applied loading

	Dead full UDL 53 kN/m	

Load combinations

Load combination 1	Support A	Dead × 1.50
		Imposed × 1.50
	Span 1	Dead × 1.50
		Imposed × 1.50

	Support B	Dead × 1.50
		Imposed × 1.50
	Span 2	Dead × 1.50
		Imposed × 1.50
	Support C	Dead × 1.50
		Imposed × 1.50
	Span 3	Dead × 1.40
		Imposed × 1.60
	Support D	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment support A	M_{A_max} = **0** kNm	M_{A_red} = **0** kNm
Maximum moment span 1 at 1003 mm	M_{s1_max} = **40** kNm	M_{s1_red} = **40** kNm
Maximum moment support B	M_{B_max} = **-208** kNm	M_{B_red} = **-208** kNm
Maximum moment span 2 at support	M_{s2_max} = **154** kNm	M_{s2_red} = **154** kNm
Maximum moment support C	M_{C_max} = **-200** kNm	M_{C_red} = **-200** kNm
Maximum moment span 3 at 2000 mm	M_{s3_max} = **0** kNm	M_{s3_red} = **0** kNm
Maximum moment support D	M_{D_max} = **0** kNm	M_{D_red} = **0** kNm
Maximum shear support A	V_{A_max} = **80** kN	V_{A_red} = **80** kN
Maximum shear support A span 1 at 182 mm	$V_{A\_s1\_max}$ = **65** kN	$V_{A\_s1\_red}$ = **65** kN
Maximum shear support B	V_{B_max} = **240** kN	V_{B_red} = **240** kN
Maximum shear support B span 1 at 3322 mm	$V_{B\_s1\_max}$ = **-183** kN	$V_{B\_s1\_red}$ = **-183** kN
Maximum shear support B span 2 at 178 mm	$V_{B\_s2\_max}$ = **224** kN	$V_{B\_s2\_red}$ = **224** kN
Maximum shear support C	V_{C_max} = **-237** kN	V_{C_red} = **-237** kN
Maximum shear support C span 2 at 5823 mm	$V_{C\_s2\_max}$ = **-221** kN	$V_{C\_s2\_red}$ = **-221** kN
Maximum shear support C span 3 at 178 mm	$V_{C\_s3\_max}$ = **159** kN	$V_{C\_s3\_red}$ = **159** kN
Maximum shear support D	V_{D_max} = **26** kN	V_{D_red} = **55** kN
Maximum shear support D span 3 at 1818 mm	$V_{D\_s3\_max}$ = **40** kN	$V_{D\_s3\_red}$ = **40** kN
Maximum reaction at support A	R_A = **80** kN	
Unfactored dead load reaction at support A	R_{A_Dead} = **53** kN	
Maximum reaction at support B	R_B = **438** kN	
Unfactored dead load reaction at support B	R_{B_Dead} = **292** kN	
Maximum reaction at support C	R_C = **411** kN	
Unfactored dead load reaction at support C	R_{C_Dead} = **278** kN	
Maximum reaction at support D	R_D = **-26** kN	
Unfactored dead load reaction at support D	R_{D_Dead} = **-14** kN	

Rectangular section details

Section width	b = **1200** mm
Section depth	h = **250** mm

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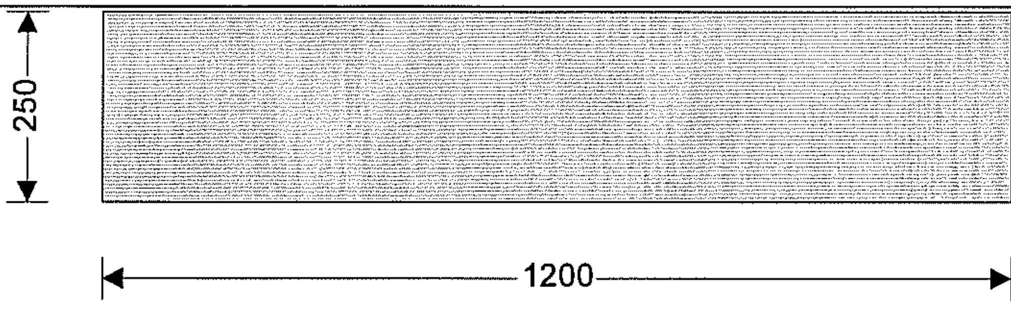


Concrete details

Concrete strength class — **C32/40**

Characteristic compressive cube strength — f_{cu} = **40 N/mm^2**

Modulus of elasticity of concrete — $E_c = 20kN/mm^2 + 200 \times f_{cu}$ = **28000 N/mm^2**

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

Reinforcement details

Characteristic yield strength of reinforcement — f_y = **500 N/mm^2**

Characteristic yield strength of shear reinforcement — f_{yv} = **500 N/mm^2**

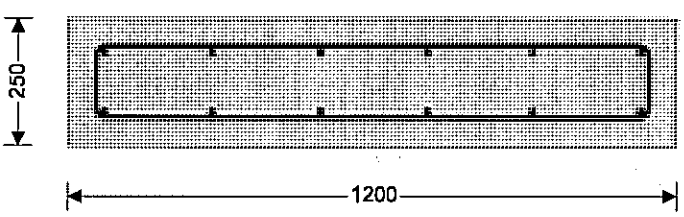
Nominal cover to reinforcement

Nominal cover to top reinforcement — c_{nom_t} = **50 mm**

Nominal cover to bottom reinforcement — c_{nom_b} = **50 mm**

Nominal cover to side reinforcement — c_{nom_s} = **50 mm**

<u>**Support A**</u>



Rectangular section in shear

Design shear force span 1 at 182 mm — $V = \max(V_{A\_s1\_max}, V_{A\_s1\_red})$ = **65 kN**

Design shear stress — $v = V / (b \times d)$ = **0.299 N/mm^2**

Design concrete shear stress — $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **0.738 N/mm^2**

Allowable design shear stress — $v_{max} = \min(0.8 N/mm^2 \times (f_{cu}/1 N/mm^2)^{0.5}, 5 N/mm^2)$ = **5.000 N/mm^2**

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 — $v < 0.5v_c$

Design shear resistance required — $v_s = \max(v - v_c, 0.4 N/mm^2)$ = **0.400 N/mm^2**

Area of shear reinforcement required — $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **1103 mm^2/m**

Shear reinforcement provided — 2 x 10ϕ legs at 125 c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **1257 mm^2/m**

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing — $s_{vl,max} = 0.75 \times d$ = **137 mm**

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

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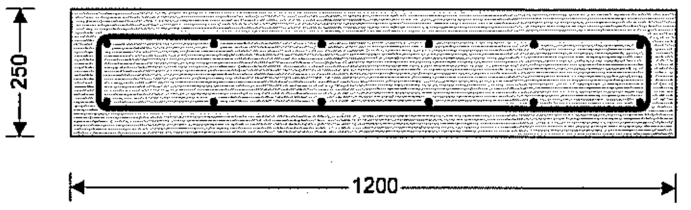
Mid span 1



6 x 16ϕ bars
2 x 10ϕ shear legs at 125 c/c
6 x 16ϕ bars

Design moment resistance of rectangular section (cl. 3.4.4) - Positive midspan moment

Design bending moment — $M = abs(M_{s1\_red})$ = **40** kNm

Depth to tension reinforcement — $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **182** mm

Redistribution ratio — $\beta_b = min(1 - m_{rs1}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{cu})$ = **0.025**

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm — $z = min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **173** mm

Depth of neutral axis — $x = (d - z) / 0.45$ = **20** mm

Area of tension reinforcement required — $A_{s,req} = M / (0.87 \times f_y \times z)$ = **532** mm^2

Tension reinforcement provided — 6 × 16ϕ bars

Area of tension reinforcement provided — $A_{s,prov}$ = **1206** mm^2

Minimum area of reinforcement — $A_{s,min} = 0.0013 \times b \times h$ = **390** mm^2

Maximum area of reinforcement — $A_{s,max} = 0.04 \times b \times h$ = **12000** mm^2

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

Rectangular section in shear

Shear reinforcement provided — 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **1257** mm^2/m

Minimum area of shear reinforcement (Table 3.7) — $A_{sv,min} = 0.4N/mm^2 \times b / (0.87 \times f_{yv})$ = **1103** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5) — $s_{vl,max} = 0.75 \times d$ = **137** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design concrete shear stress — $v_c = 0.79N/mm^2 \times min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times max(1, (400mm /d)^{1/4}) \times (min(f_{cu}, 40N/mm^2) / 25N/mm^2)^{1/3} / \gamma_m$ = **0.738** N/mm^2

Design shear resistance provided — $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b$ = **0.456** N/mm^2

Design shear stress provided — $v_{prov} = v_{s,prov} + v_c$ = **1.194** N/mm^2

Design shear resistance — $V_{prov} = v_{prov} \times (b \times d)$ = **260.8** kN

Shear links provided valid between 0 mm and 3500 mm with tension reinforcement of 1206 mm²

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension — $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) /(N_{bot} - 1) - \phi_{bot}$ = **197** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension — $s_{min} = h_{agg} + 5$ mm = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress — $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **147.0** N/mm^2

Maximum distance between bars in tension — $s_{max} = min(47000 N/mm / f_s, 300 mm)$ = **300** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9) — $span\_to\_depth_{basic}$ = **26.0**

Design service stress in tension reinforcement — $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **147.0** N/mm^2

Modification for tension reinforcement

$$f_{tens} = \min(2.0, 0.55 + (477\text{N/mm}^2 - f_s) / (120 \times (0.9\text{N/mm}^2 + (M / (b \times d^2))))) = \mathbf{1.992}$$

Modification for compression reinforcement

$$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d)))) = \mathbf{1.155}$$

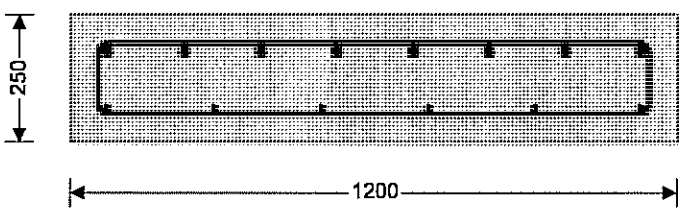
Modification for span length — $f_{long} = 1.000$

Allowable span to depth ratio — $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp}$ = **59.8**

Actual span to depth ratio — $span\_to\_depth_{actual} = L_{s1} / d$ = **19.2**

PASS - Actual span to depth ratio is within the allowable limit

Support B



8 x 25ϕ bars
2 x 10ϕ shear legs at 125 c/c
6 x 16ϕ bars

Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment — $M = abs(M_{B\_red})$ = **208** kNm

Depth to tension reinforcement — $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **178** mm

Redistribution ratio — $\beta_b = \min(1 - m_{rB}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{cu})$ = **0.137**

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **144** mm

Depth of neutral axis — $x = (d - z) / 0.45 = 74$ mm

Area of tension reinforcement required — $A_{s,req} = M / (0.87 \times f_y \times z)$ = **3313** mm^2

Tension reinforcement provided — 8 × 25ϕ bars

Area of tension reinforcement provided — $A_{s,prov}$ = **3927** mm^2

Minimum area of reinforcement — $A_{s,min} = 0.0013 \times b \times h$ = **390** mm^2

Maximum area of reinforcement — $A_{s,max} = 0.04 \times b \times h$ = **12000** mm^2

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

Rectangular section in shear

Design shear force span 1 at 3322 mm — $V = abs(\min(V_{B\_s1\_max}, V_{B\_s1\_red}))$ = **183** kN

Design shear stress — $v = V / (b \times d)$ = **0.857** N/mm^2

Design concrete shear stress — $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 / d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **1.111** N/mm^2

Allowable design shear stress — $v_{max} = \min(0.8\text{ N/mm}^2 \times (f_{cu}/1\text{ N/mm}^2)^{0.5}, 5\text{ N/mm}^2)$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 — $0.5 \times v_c < v < (v_c + 0.4\text{ N/mm}^2)$

Design shear resistance required — $v_s = \max(v - v_c, 0.4\text{ N/mm}^2)$ = **0.400** N/mm^2

Area of shear reinforcement required — $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **1103** mm^2/m

Shear reinforcement provided — 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1257** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing $s_{vl,max} = 0.75 \times d$ = **133** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear force span 2 at 178 mm $V = \max(V_{B\_s2\_max}, V_{B\_s2\_red})$ = **224** kN

Design shear stress $v = V / (b \times d)$ = **1.051** N/mm^2

Design concrete shear stress $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 /d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **1.111** N/mm^2

Allowable design shear stress $v_{max} = \min(0.8 \text{ N/mm}^2 \times (f_{cu}/1 \text{ N/mm}^2)^{0.5}, 5 \text{ N/mm}^2)$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 $0.5 \times v_c < v < (v_c + 0.4 \text{ N/mm}^2)$

Design shear resistance required $v_s = \max(v - v_c, 0.4 \text{ N/mm}^2)$ = **0.400** N/mm^2

Area of shear reinforcement required $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **1103** mm^2/m

Shear reinforcement provided 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1257** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing $s_{vl,max} = 0.75 \times d$ = **133** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top}/2)) /(N_{top} - 1) - \phi_{top}$ = **126** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension $s_{min} = h_{agg}$ + 5 mm = **25** mm

PASS - Satisfies the minimum spacing criteria

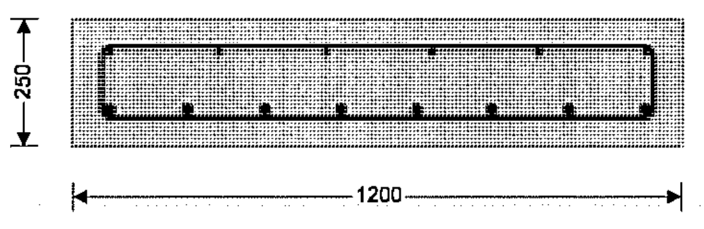
Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **281.2** N/mm^2

Maximum distance between bars in tension $s_{max} = \min(47000 \text{ N/mm} / f_s, 300 \text{ mm})$ = **167** mm

PASS - Satisfies the maximum spacing criteria

Mid span 2



6 x 16ϕ bars
2 x 10ϕ shear legs at 125 c/c
8 x 25ϕ bars

Design moment resistance of rectangular section (cl. 3.4.4) - Positive midspan moment

Design bending moment $M = \text{abs}(M_{s2\_red})$ = **154** kNm

Depth to tension reinforcement $d = h - c_{nom\_b} - \phi_v - \phi_{bot} / 2$ = **178** mm

Redistribution ratio $\beta_b = \min(1 - m_{rs2}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{cu})$ = **0.102**

K' = 0.156

K' > K - No compression reinforcement is required

Lever arm $z = \min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **154** mm

Depth of neutral axis $x = (d - z) / 0.45$ = **51** mm

Area of tension reinforcement required $A_{s,req} = M / (0.87 \times f_y \times z) = \mathbf{2294}$ mm^2

Tension reinforcement provided 8 × 25ϕ bars

Area of tension reinforcement provided $A_{s,prov} = \mathbf{3927}$ mm^2

Minimum area of reinforcement $A_{s,min} = 0.0013 \times b \times h = \mathbf{390}$ mm^2

Maximum area of reinforcement $A_{s,max} = 0.04 \times b \times h = \mathbf{12000}$ mm^2

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

Rectangular section in shear

Shear reinforcement provided 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided $A_{sv,prov} = \mathbf{1257}$ mm^2/m

Minimum area of shear reinforcement (Table 3.7) $A_{sv,min} = 0.4\text{N/mm}^2 \times b / (0.87 \times f_{yv}) = \mathbf{1103}$ mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5) $s_{vl,max} = 0.75 \times d = \mathbf{133}$ mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design concrete shear stress $v_c = 0.79\text{N/mm}^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400\text{mm} /d)^{1/4}) \times (\min(f_{cu}, 40\text{N/mm}^2) / 25\text{N/mm}^2)^{1/3} / \gamma_m = \mathbf{1.111}$ N/mm^2

Design shear resistance provided $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b = \mathbf{0.456}$ N/mm^2

Design shear stress provided $v_{prov} = v_{s,prov} + v_c = \mathbf{1.566}$ N/mm^2

Design shear resistance $V_{prov} = v_{prov} \times (b \times d) = \mathbf{333.6}$ kN

Shear links provided valid between 0 mm and 6000 mm with tension reinforcement of 3927 mm^2

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{bot}/2)) /(N_{bot} - 1) - \phi_{bot} = \mathbf{126}$ mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension $s_{min} = h_{agg} + 5$ mm $= \mathbf{25}$ mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b) = \mathbf{194.7}$ N/mm^2

Maximum distance between bars in tension $s_{max} = \min(47000\text{ N/mm} / f_s, 300\text{ mm}) = \mathbf{241}$ mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9) span_to_depth$_{basic}$ = **26.0**

Design service stress in tension reinforcement $f_s = (2 \times f_y \times A_{s,req})/ (3 \times A_{s,prov} \times \beta_b) = \mathbf{194.7}$ N/mm^2

Modification for tension reinforcement

$$f_{tens} = \min(2.0, 0.55 + (477\text{N/mm}^2 - f_s) / (120 \times (0.9\text{N/mm}^2 + (M / (b \times d^2))))) = \mathbf{1.023}$$

Modification for compression reinforcement

$$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d)))) = \mathbf{1.159}$$

Modification for span length $f_{long} = 1.000$

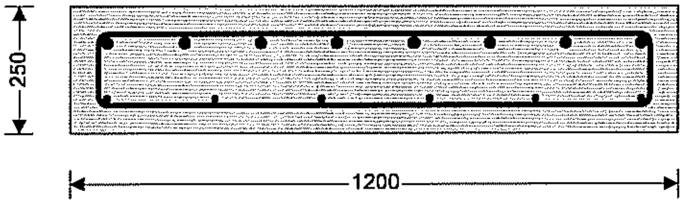
Allowable span to depth ratio span_to_depth$_{allow}$ = span_to_depth$_{basic} \times f_{tens} \times f_{comp}$ = **30.8**

Actual span to depth ratio span_to_depth$_{actual} = L_{s2} / d$ = **33.8**

FAIL - Actual span to depth ratio exceeds the allowable limit

<u>Support C</u>

Say - OK.



8 x 25ϕ bars
2 x 10ϕ shear legs at 125 c/c
6 x 16ϕ bars

Design moment resistance of rectangular section (cl. 3.4.4)

Design bending moment $M = abs(M_{C\_red})$ = **200** kNm

Depth to tension reinforcement $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **178** mm

Redistribution ratio $\beta_b = min(1 - m_{rC}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{cu})$ = **0.132**

$K' = 0.156$

K' > K - No compression reinforcement is required

Lever arm $z = min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **146** mm

Depth of neutral axis $x = (d - z) / 0.45$ = **70** mm

Area of tension reinforcement required $A_{s,req} = M / (0.87 \times f_y \times z)$ = **3149** mm^2

Tension reinforcement provided 8 × 25ϕ bars

Area of tension reinforcement provided $A_{s,prov}$ = **3927** mm^2

Minimum area of reinforcement $A_{s,min} = 0.0013 \times b \times h$ = **390** mm^2

Maximum area of reinforcement $A_{s,max} = 0.04 \times b \times h$ = **12000** mm^2

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

Rectangular section in shear

Design shear force span 2 at 5823 mm $V = abs(min(V_{C\_s2\_max}, V_{C\_s2\_red}))$ = **221** kN

Design shear stress $v = V / (b \times d)$ = **1.039** N/mm^2

Design concrete shear stress $v_c = 0.79 \times min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times max(1, (400 /d)^{1/4}) \times (min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **1.111** N/mm^2

Allowable design shear stress $v_{max} = min(0.8 N/mm^2 \times (f_{cu}/1 N/mm^2)^{0.5}, 5 N/mm^2)$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 $0.5 \times v_c < v < (v_c + 0.4 N/mm^2)$

Design shear resistance required $v_s = max(v - v_c, 0.4 N/mm^2)$ = **0.400** N/mm^2

Area of shear reinforcement required $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **1103** mm^2/m

Shear reinforcement provided 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1257** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing $s_{vl,max} = 0.75 \times d$ = **133** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design shear force span 3 at 178 mm $V = max(V_{C\_s3\_max}, V_{C\_s3\_red})$ = **159** kN

Design shear stress $v = V / (b \times d)$ = **0.747** N/mm^2

Design concrete shear stress $v_c = 0.79 \times min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times max(1, (400 /d)^{1/4}) \times (min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **1.111** N/mm^2

Allowable design shear stress $v_{max} = min(0.8 N/mm^2 \times (f_{cu}/1 N/mm^2)^{0.5}, 5 N/mm^2)$ = **5.000** N/mm^2

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 $0.5 \times v_c < v < (v_c + 0.4 N/mm^2)$

Design shear resistance required $v_s = max(v - v_c, 0.4 N/mm^2)$ = **0.400** N/mm^2

Area of shear reinforcement required $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **1103** mm^2/m

Shear reinforcement provided 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1257** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing $s_{vl,max} = 0.75 \times d$ = **133** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top}/2)) / (N_{top} - 1) - \phi_{top}$ = **126** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension $s_{min} = h_{agg} + 5$ mm = **25** mm

PASS - Satisfies the minimum spacing criteria

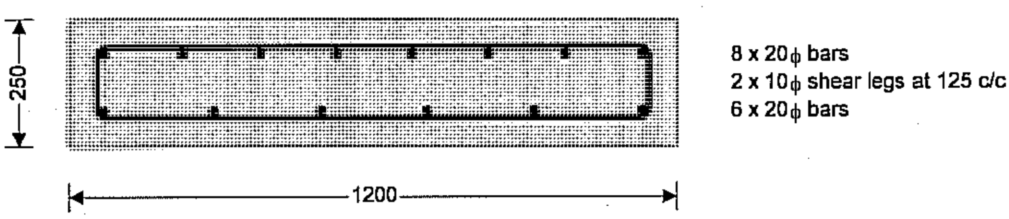
Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **267.3** N/mm^2

Maximum distance between bars in tension s_{max} = min(47000 N/mm / f_s, 300 mm) = **176** mm

PASS - Satisfies the maximum spacing criteria

<u>**Mid span 3**</u>



Design moment resistance of rectangular section (cl. 3.4.4) - Negative span moment

Design bending moment $M = abs(M_{s3\_neg})$ = **122** kNm

Depth to tension reinforcement $d = h - c_{nom\_t} - \phi_v - \phi_{top} / 2$ = **180** mm

Redistribution ratio $\beta_b = min(1 - m_{rs3}, 1)$ = **1.000**

$K = M / (b \times d^2 \times f_{cu})$ = **0.078**

K' = 0.156

K' > K - No compression reinforcement is required

Lever arm $z = min(d \times (0.5 + (0.25 - K / 0.9)^{0.5}), 0.95 \times d)$ = **163** mm

Depth of neutral axis $x = (d - z) / 0.45$ = **39** mm

Area of tension reinforcement required $A_{s,req} = M / (0.87 \times f_y \times z)$ = **1724** mm^2

Tension reinforcement provided 8 × 20ϕ bars

Area of tension reinforcement provided $A_{s,prov}$ = **2513** mm^2

Minimum area of reinforcement $A_{s,min} = 0.0013 \times b \times h$ = **390** mm^2

Maximum area of reinforcement $A_{s,max} = 0.04 \times b \times h$ = **12000** mm^2

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

Rectangular section in shear

Shear reinforcement provided 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided $A_{sv,prov}$ = **1257** mm^2/m

Minimum area of shear reinforcement (Table 3.7) $A_{sv,min}$ = 0.4N/mm^2 × b / (0.87 × f_{yv}) = **1103** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing (cl. 3.4.5.5) $s_{vl,max} = 0.75 \times d$ = **135** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

Design concrete shear stress $v_c = 0.79\text{N/mm}^2 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400\text{mm} / d)^{1/4}) \times (\min(f_{cu}, 40\text{N/mm}^2) / 25\text{N/mm}^2)^{1/3} / \gamma_m$ = **0.949** N/mm²

Design shear resistance provided $v_{s,prov} = A_{sv,prov} \times 0.87 \times f_{yv} / b$ = **0.456** N/mm²

Design shear stress provided $v_{prov} = v_{s,prov} + v_c$ = **1.405** N/mm²

Design shear resistance $V_{prov} = v_{prov} \times (b \times d)$ = **303.4** kN

Shear links provided valid between 0 mm and 2000 mm with tension reinforcement of 2513 mm²

Spacing of reinforcement (cl 3.12.11)

Actual distance between bars in tension $s = (b - 2 \times (c_{nom\_s} + \phi_v + \phi_{top}/2)) / (N_{top} - 1) - \phi_{top}$ = **131** mm

Minimum distance between bars in tension (cl 3.12.11.1)

Minimum distance between bars in tension $s_{min} = h_{agg} + 5\text{ mm}$ = **25** mm

PASS - Satisfies the minimum spacing criteria

Maximum distance between bars in tension (cl 3.12.11.2)

Design service stress $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **228.6** N/mm²

Maximum distance between bars in tension $s_{max} = \min(47000\text{ N/mm} / f_s, 300\text{ mm})$ = **206** mm

PASS - Satisfies the maximum spacing criteria

Span to depth ratio (cl. 3.4.6)

Basic span to depth ratio (Table 3.9) $span\_to\_depth_{basic}$ = **26.0**

Design service stress in tension reinforcement $f_s = (2 \times f_y \times A_{s,req}) / (3 \times A_{s,prov} \times \beta_b)$ = **228.6** N/mm²

Modification for tension reinforcement

$f_{tens} = \min(2.0, 0.55 + (477\text{N/mm}^2 - f_s) / (120 \times (0.9\text{N/mm}^2 + (M / (b \times d^2)))))$ = **1.063**

Modification for compression reinforcement

$f_{comp} = \min(1.5, 1 + (100 \times A_{s2,prov} / (b \times d)) / (3 + (100 \times A_{s2,prov} / (b \times d))))$ = **1.225**

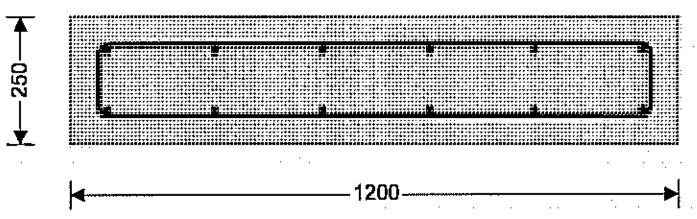
Modification for span length f_{long} = 1.000

Allowable span to depth ratio $span\_to\_depth_{allow} = span\_to\_depth_{basic} \times f_{tens} \times f_{comp}$ = **33.9**

Actual span to depth ratio $span\_to\_depth_{actual} = L_{s3} / d$ = **11.1**

PASS - Actual span to depth ratio is within the allowable limit

__Support D__



Rectangular section in shear

Design shear force span 3 at 1818 mm $V = \max(V_{D\_s3\_max}, V_{D\_s3\_red})$ = **40** kN

Design shear stress $v = V / (b \times d)$ = **0.185** N/mm²

Design concrete shear stress $v_c = 0.79 \times \min(3,[100 \times A_{s,prov} / (b \times d)]^{1/3}) \times \max(1, (400 / d)^{1/4}) \times (\min(f_{cu}, 40) / 25)^{1/3} / \gamma_m$

v_c = **0.738** N/mm²

Allowable design shear stress $v_{max} = \min(0.8\text{ N/mm}^2 \times (f_{cu}/1\text{ N/mm}^2)^{0.5}, 5\text{ N/mm}^2)$ = **5.000** N/mm²

PASS - Design shear stress is less than maximum allowable

Value of v from Table 3.7 $v < 0.5v_c$

Design shear resistance required $v_s = \max(v - v_c, 0.4\text{ N/mm}^2)$ = **0.400** N/mm²

Area of shear reinforcement required $A_{sv,req} = v_s \times b / (0.87 \times f_{yv})$ = **1103** mm²/m

Shear reinforcement provided — 2 × 10ϕ legs at 125 c/c

Area of shear reinforcement provided — $A_{sv,prov}$ = **1257** mm^2/m

PASS - Area of shear reinforcement provided exceeds minimum required

Maximum longitudinal spacing — $s_{vl,max} = 0.75 \times d =$ **137** mm

PASS - Longitudinal spacing of shear reinforcement provided is less than maximum

AND DESIGNS LTD

SPECIFICATION

FOR UNDERPINNING

TO : 11 Ellerdale Road, Hampstead

Job No: 12.195

The Institution of Structural Engineers

90 Meadrow, Godalming, Surrey, GU7 3HY
Tel: 01483 418 140, Fax: 01483 421 304, Email: info@anddesigns.co.uk
Company Registration No. 3401843, VAT No. 757 0489 03
Director: Jeff Walker CEng MIStructE

AND DESIGNS LTD

RESPONSIBILITIES

1) The Contractor shall be completely responsible for the safety of the existing structure during the underpinning operations and he shall design, supply and erect all the temporary supports that may be required or prove necessary during the course of the work.

2) The details of such supports shall be agreed with the Engineer and other interested parties prior to their erection.

SURVEY AND CONDITION OF BUILDING

1) Before commencing work the Contractor shall carry out an inspection and produce a Schedule of Conditions for the building to be underpinned. This shall be agreed with the Architects before commencing work. Where necessary repairs shall be effected to enable the underpinning to be carried out.

PROGRAMME AND SEQUENCE

1) The contractor shall follow the method statement as listed below. Any variation from the method statement must be agreed with the Engineer and other interested parties prior to work commencing.

   a) Excavate pits Mk 1 to below ground floor level.
   b) Bearing to be approved by engineer / L.A. Inspector
   c) Ram 4 no. 12 mm diameter MS dowel bars into excavation each side of pit leaving 400 mm projecting into excavation.
   d) Clean underside of existing foundation with stiff brush.
   e) Shutter face of pour to line through with wall above.
   h) Pour lift of concrete to within 75 mm of underside of existing foundation.
   i) Allow 24 hours for concrete to set.
   j) Ram hand damp dry pack (1:3 cement: sharp sand) into 75 mm void to pin up tight with existing foundation.
   k) Allow 24 hours for dry pack to set.
   l) Excavate pits Mk 2 and follow above sequence for all remaining pit excavation.

PROTECTION

1) The Contractor shall protect the area in which the work is being carried out by the provision of suitable hoarding, fences etc.

2) Unless otherwise instructed by the Architect all work shall be carried out from within the site.

EXCAVATION

1) The underpinning shall be carried out in sections not exceeding 1200 mm. The excavation and construction of the sections shall be carried out in a "hit and miss" pattern such that a maximum degree of support is offered to the wall at all times.

2) Unless otherwise stated on the drawings the underpinning shall be carried out for the whole width of the existing foundation.

3) Where excavations exceed 1000 mm in depth or wherever it is found necessary or called for on the drawings, all excavations shall be fully planked and strutted. Reference should be made to Specification "Earthworks", in this regard.

4) The material providing the support to the remote earth face below the foundations shall, if necessary, be left in position. It must not therefore be subject to deterioration. Any gaps between this support and the earth face shall be filled with cementatious grout. All timber planking and strutting shall be removed.

5) The underside of the exposed foundations shall be thoroughly cleaned of all soil and other loose material before the section of underpinning is constructed.

6) Excavations which are left open overnight shall be blinded with 50 mm of 1.8 concrete with sulphate resisting cement.

7) If water is stuck during excavation, excavation shall cease until a method of dewatering has been devised which will not be detrimental to the adjoining foundations and has been agreed with the Engineer.

CONSTRUCTION OF UNDERPINNING

1) It is recommended that the underpinning is carried out in concrete sections and this has been detailed on the drawings.

2) In the event that the Contractor requests that the work be carried out in brickwork then his alternative proposals will be considered by the Engineer.

For the concrete work:

a) The concrete mix shall be grade 25 with sulphate resisting cement unless noted otherwise on the drawings.

b) Where dowels are shown on the drawings they shall be so provided or toggle joints at 1/3rd positions as noted on the drawings.

c) The concrete shall be brought to within 75 mm of the underside of the foundations.

d) A period of 24 hours shall elapse between completion of the new concrete foundation and the commencement of the dry packing.

e) A period of 24 hours shall elapse between the dry packing operation and the commencement of excavations to the adjoining section of underpinning.

PINNING UP

1) A semi-dry 1.3 mix with 10 mm aggregate shall be thoroughly rammed into position between the concrete stool and the underside of the existing foundation. A suitable tool shall be used to ensure that no voids are left in the dry pack zone.

2) A non-shrinking grout agent may be employed in the mix with the Engineers approval.

BACKFILL

1) After completion of underpinning and curing, backfill with lean mix 15N / mm^2 min or alternative material to be agreed with the Engineer. Under no circumstances shall the Contractor replace with existing excavated material without permission of the Engineer

2) The Contractor shall take into account all necessary carting away of existing material during the tender/pricing period.

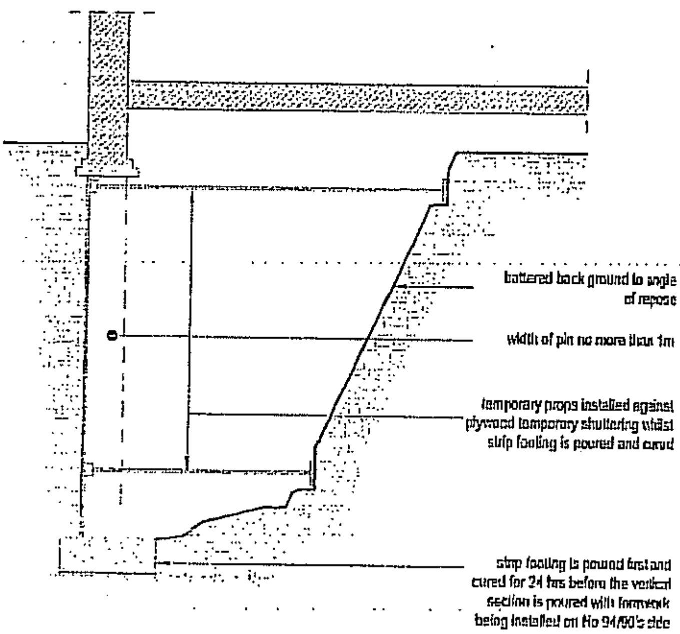
RECORDS

1) The contractor shall keep an accurate record of the progress of underpinning operations which shall be available for reference at any time.

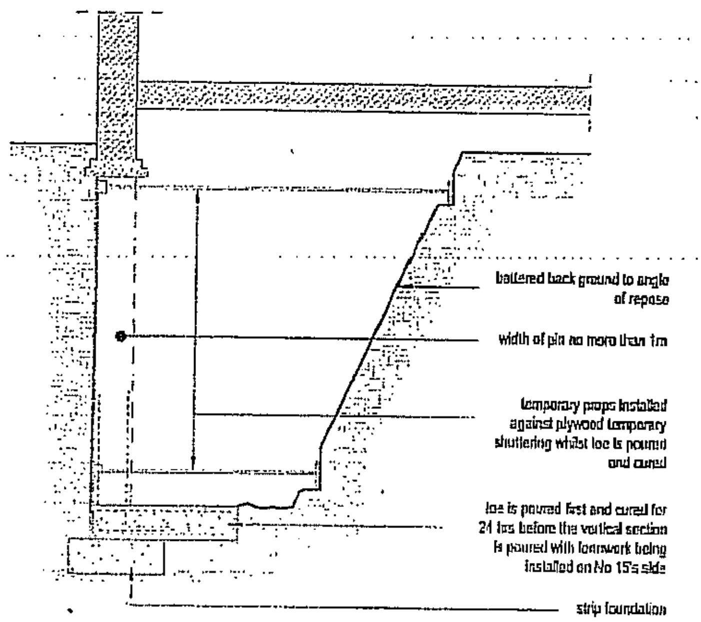
1 ELLERDALE ROAD, HAMPSTEAD

STRIP FOUNDATION / UNDERPINNING DETAILS

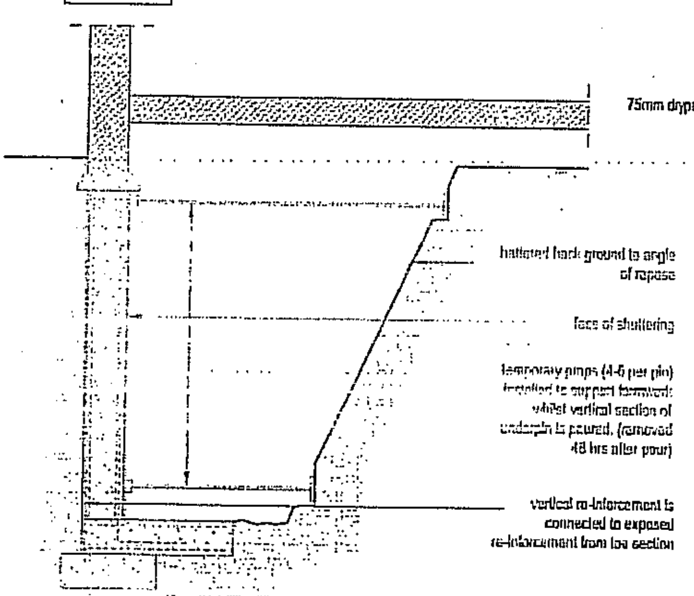
STAGE 1 POUR STRIP FOUNDATION



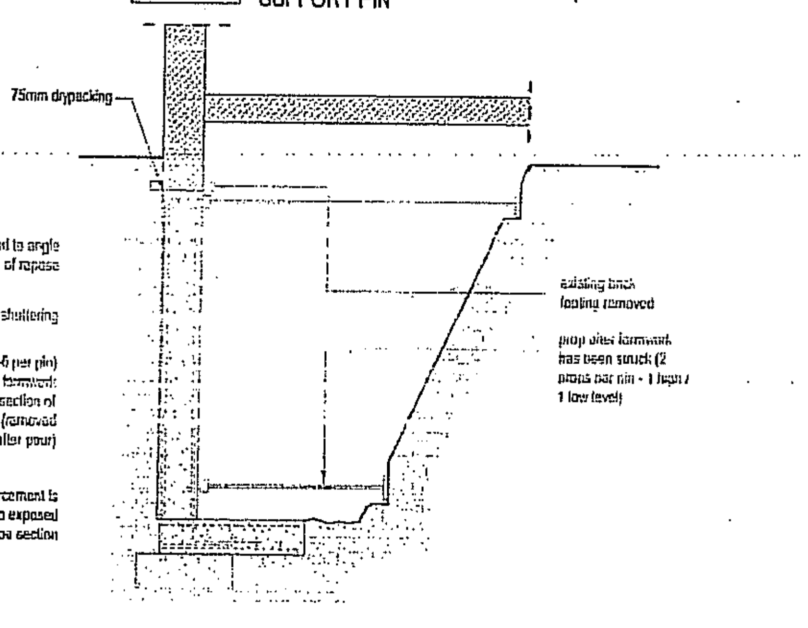
STAGE 2 POUR TOE / BASE FOUNDATION



STAGE 3 POUR VERTICAL PIN TIED INTO TOE



STAGE 4 DRY PACK TO TOP OF PIN AND SUPPORT PIN



AND
DESIGNS LTD

SPECIFICATION

FOR

STRUCTURAL WORKS

TO : 1 Ellerdale Road, Hampstead

JOB.NO : 12.195

The Institution of Structural Engineers

90 Meadrow, Godalming, Surrey, GU7 3HY
Tel: 01483 418 140, Fax: 01483 421 304, Email: info@anddesigns.co.uk
Director: Jeff Walker CEng MIStructE
Company Registration No. 3401843, VAT No. 757 0489 03
Registered Office: 180 London Road, Kingston upon Thames, Surrey KT2 6QW

TRADA member

GENERAL NOTES

1. This drawing is to be read in conjunction with all relevant contract documentation.

2. Dimensions marked * thus to be checked on site.

3. The contractor shall be responsible for the existing structure during the course of the works.

4. This drawing is not to be scaled, if in doubt ask.

5. All proprietary products to be installed in accordance with the Manufacturers Recommendations.

6. Underside of foundation to obtain a minimum bearing pressure of 100kN / m^2 to the satisfaction of the Building Control / Structural Engineer. Foundations to be a minimum of 1m below ground level unless noted otherwise.

7. Where no site investigation has taken place and where trees are adjacent to the site in clay soils the Contractor shall inform the Engineer / B.C.O. and allow for the foundations to be designed in accordance with NHBC Practice Note 4.2 recommendations.

8. Where service invert levels are below the formation level the footing must be increased to a minimum of 300mm below the invert of the pipe and 300mm surround to the pipe. The pipe must be sleeved with a minimum of 25mm clearance to any face of the pipework by either low-density polystyrene or UPVC sleeve.

9. Ground bearing slabs to be as follows unless noted otherwise and in strict accordance with the soils investigation :
   150 mm thick with A142 mesh top (40mm cover)
   300-mm minimum laps on 1200 gauge visqueen, on sand / concrete blinding, on minimum 150 mm hardcore compacted free from impurities

10. All footings adjacent to existing footings shall have a dispersal angle of 45 degrees to the underside of the footing or at the same level as the existing on no account shall the footings surcharge the existing footings or drains where this occurs refer to the Engineer for further instructions.

11. Where temporary works are required the Contractor shall allow in his pricing. For a competent temporary works Engineer to design all necessary supports for the existing structure during the course of the works.

12. Fire casing to beams to be in accordance with Architect's details and drawings.

13. All propriety lintels are to be designed by the Manufacturers and in accordance with their recommendations. Where PC floors are supported on the proposed lintels, the correct type of lintels should be installed in accordance with the Manufacturers design.

14. All existing lintels are to be checked prior to installation of any steel work or timbers.

SITE INVESTIGATION

A basic investigation of the site shall be carried out and recorded by a suitable person to the satisfaction of NHBC

Where the results of an initial assessment indicate that hazards are not suspected on the site, this should be substantiated by carrying out a **basic investigation**

This approach is to provide assurance for all sites, regardless of how free of hazards they may appear.

Only suitable persons with the skills and knowledge should carry out the basic investigation.

The following provides a specification for the basic investigation for all sites.

Trial pits should be located so as to be representative of the site. (For more detailed information refer to BS 5930.)

The number and depth of trial pits needed depends upon:

- the proposed development
- how inconsistent the soil and geology is across the site
- the nature of the site.

The depth of the trial pits should not usually be less than 3m.

Items to be taken into account include:

(a) geotechnical investigation

A basic geotechnical investigation should be carried out. This will include trial pits and, where they do not provide sufficient information, boreholes will be necessary.

Physical tests, such as plasticity index tests, should be carried out as appropriate to support the result of the initial assessment.

Trial pits should be located outside the likely foundation area. The distance from the edge of the foundation should not be less than the trial pit depth.

(b) contamination investigation

A basic contamination investigation should be carried out as part of the geotechnical investigation.
This should consist of sampling and testing of soil taken from trial pits during the geotechnical investigation, as found to be necessary from the outcome of the initial assessment.
During the excavation of the pits the use of sight and smell may help to identify certain contaminants.
Where there is any doubt about the condition of the ground a detailed investigation should be carried out

SITE INVESTIGATION Continued

Further Investigation

If the basic investigation reveals the presence of geotechnical and/or contamination hazards further assessment is required and a **detailed investigation** should be carried out

STRUCTURAL STEELWORK

1. All materials and workmanship to be in accordance with BS5950. The structural use of steelwork in building.

2. Structural steelwork sections to be Grade S275 mild steel in accordance with BS 4360.

3. Bolts to be grade 8.8 unless noted otherwise.

4. Welds to be 6mm continuous fillet, unless noted otherwise.

5. Contractor must verify all dimensions on site before commencing any work or making shop drawings which are to be issued to the Engineer. No dimensions to be scaled from drawings. Discrepancies must be reported to the engineer prior to proceeding. 7 working days are required by the Engineer to check and comment on any working drawings prior to fabrication.

6. Contractor to design all connections for maximum moments and reactions indicated on drawings or calculations issued to the Contractor.

7. Steelwork which is not required to be encased in concrete to be blast cleaned / wire brushed* free from mill scale, rust and other contamination and painted with two coats of approved primer as soon as practicable but not more than four hours after cleaning.

8. Uncased stanchions and beams located within an external wall to have a minimum gap of 40mm from face of external brickwork or alternately 25mm min impermeable insulation from face of steel to the external wall, unless galvanised or similar treatment.

9. All concrete encased steelwork to be unpainted.

10. All pockets formed in brickwork or blockwork for steel beams to be made good in Grade 35 concrete.

11. Bolted connections to have a minimum connection of 4 N°. M20 bolts per member, unless noted otherwise.

12. Minimum bearing of steels to be 100mm, unless noted otherwise.

13. External steelwork, and where indicated, are to be galvanised steel to a minimum of 140 microns thickness unless noted otherwise in accordance with BS 728.

14. Workmanship erection and tolerances to be in accordance with the National Structural Steelwork Specification for building construction.

STRUCTURAL STEELWORK continued

15. HSFG Bolt connections are to be metal to metal and painted on site after the connection has been completed and load indicating washers are in their final position.

16. All beams bearing onto walls are to be built in with brickwork or alternatively encased around with concrete grade C35. The steelwork at roof level where beams cannot be built into brickwork are to have the top flange restrained by metal straps onto the timber plates / brickwork.

CONCRETE NOTES

1. All materials and workmanship to be in accordance with BS 8110 Parts 1 & 2 - The structural use of concrete.

2. Concrete quality to be 35N / mm^2 at 28 days unless noted otherwise, Max nominal aggregate to be 20mm.

   Minimum cement content 330kg / m^3.
   Maximum free water cement ratio 0.6

3. Reinforcement to be placed in accordance with BS 8110.

4. For details of relevant schedules refer to schedule N°......

5. Cement content not less than 330kg / m^3, Free water cement ratio not more than 0.5.

6. Concrete cubes to be taken at 7 & 28 days to obtain required crushing strengths (one cube to be taken as a spare cube.)

7. Concrete qualities for Mass Concrete foundations to low rise structures in non-aggressive soils to be Gen 3. Minimum cement content not less than 220kg / m^3 (If normal prescribed mix is used will be acceptable) or FN4 sulphate resisting cement).

8. No reinforcement to be cut displaced or omitted without prior written agreement of the engineer.

9. Cover to reinforcement to be ....

10. The ground is to be blinded into 50 mm of lean mix prior to reinforcement being placed in position, blinded concrete mix to be 1:10 to all reinforcement bases etc except water resisting structures refer to Specification. Gen 1.

11. If no soil investigation has been instigated then sulphate-resisting cement should be used in the ground construction.

TIMBER NOTES

1. All timber materials and workmanship to be in accordance with BS 5268: Part 2 - Structural Use of Timber.

2. Timber roof trusses and bracing to be designed and detailed by specialist sub-contractor. Trusses to be designed and fabricated in accordance with BS 5268: Parts 2 and 3.

3. All timbers to be a minimum strength class C16 (unless noted otherwise) and have max. moisture content of 18%. All external timbers are to be of a durable grade i.e. oak or similar approved to the Architect/Engineers Approval

4. Multiple joists / trusses to be bolted together at 600 centres with 12mm dia. bolts and 50x50x3 washer plates.

5. No notches, holes or rebates etc. to be cut in any member without the written agreement of the Engineers.

6. All trusses to be connected to timber wall plate by means of approved truss clips.

7. Site storage, handling and erection procedures of trusses are to be in accordance with BS 5269: Part 3.

8. All structural timber and trusses to be adequately protected against adverse weather conditions during stacking and after erection.

9. All structural timber is to be treated by vacuum pressure impregnation of organic or water borne preservative, to a dry salt retention in accordance with the manufacturer's recommendations. Type of treatment may be:- 'Tanalith', 'Celcure', 'Protim', or other only with the prior approval of the Architect.

10. Finger joints are not acceptable.

11. All fixings in roof space (nails, screws, bolts, hangers etc.) are to be galvanised unless noted otherwise.

12. Any lateral support system necessary to prevent buckling of compression members in trusses is to be designed, specified, detailed and supplied by the truss designer / fabricator.

13.

Joist span	Rows of Strutting
Up to 2.5	None
2.5 to 4.5	1 (located at mid-span)
Over 4.5	2 (located at third points)

TIMBER NOTES Continued

Solid strutting should be used instead of herring-bone strutting where the distance between joists is greater than three times the depth of the joists. In all other instances the use of herring-bone strutting is recommended to reduce the risk of creaking floors due to shrinkage.

Timber for herringbone strutting should be at least 38 x 38 mm.

Solid strutting should be at least 38mm thick and at least three quarters of the joist depth.

14. Strutting should be blocked solidly to perimeter walls.

15. Strutting or blocking should not block the ventilation space in cold deck flat roofs.

16. Joist should have a minimum end bearing of 50mm.

17. Ends of joists built into cavity walls should not project into the cavity, and should be painted with two coats of bituminous primer.

BRICKWORK AND BLOCKWORK

1. All materials and workmanship to be in accordance with BS 5628 Code of Practice for the Structural Use of Brickwork.

2. Brickwork to have average crushing strength of 20.5 N / mm^2 bricks (Class 3 min) unless noted otherwise.

3. Blockwork above ground to be 3.5N / mm^2 minimum, Blockwork below ground to be 7.0N / mm^2 minimum unless otherwise noted.

4. Mortar designations above ground to be 1:1:6 Cement / Lime / Sand.

5. Mortar designation below ground to be 1:3 Cement / Sand unless noted otherwise.

6. 'Hyload' DPC or similar approved to all walls.

7. Wall ties to be stainless steel vertical twist type ties to comply with BS 1243. Max spacing to be 900mm horizontally, 450mm vertically and with a 50mm embedment in the mortar joint of each leaf, unless noted otherwise. Wall ties to be placed in walls where cavities exceed 90mm to have wall ties placed at 450c/c vertically, 450c/c horizontally. Additional ties are to be provided at the sides of all openings so that there is at least one tie at 300c/c maximum.

8. Blockwork indicated thus.

9. Brickwork restraints to be in accordance with BS 5628 Part 1 at 1200mm c/c restraints to brickwork and 1200mm c/c for vertical straps.

10. For position and details of joints in masonry walls see drg: _..

11. At brick / block junctions, brickwork is to be blockbonded into blockwork unless noted otherwise.

12. Wall ties shall not slope inwards.

13. All brickwork is to be laid with frogs, if any, uppermost.

14. Where blocks are laid flat they are to be solid and no shell bedding shall be allowed.

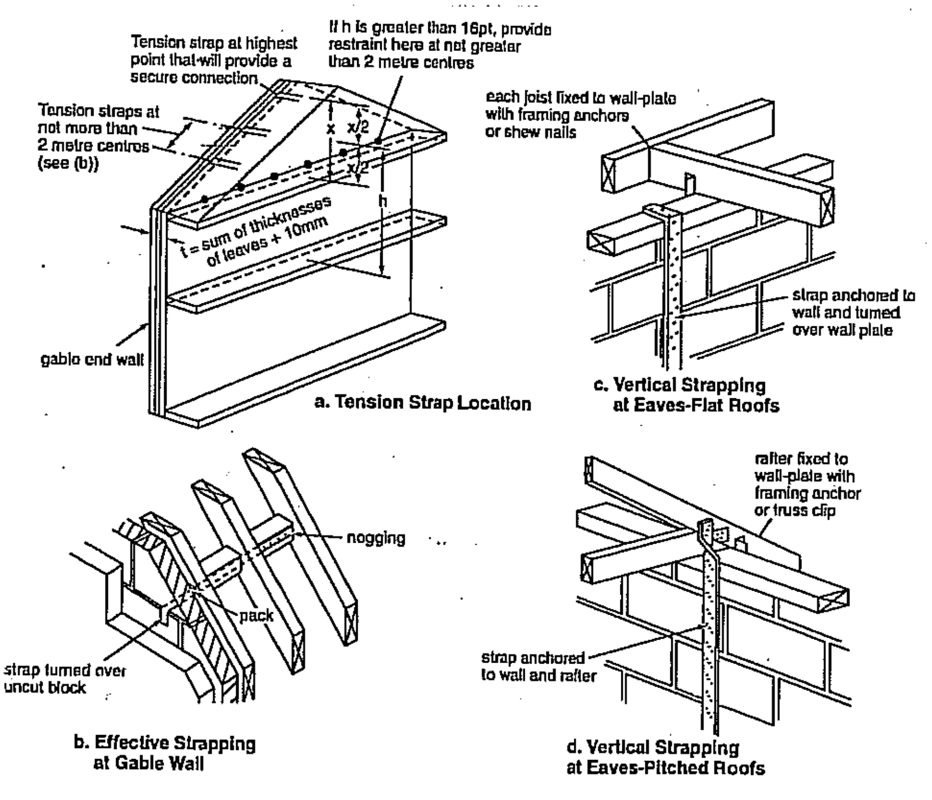
15. Lintel Bearings to be in accordance with the Manufacturers recommendations or Engineers Comments.

16. Movement joints unless otherwise stated on the drawings are to be at a maximum of 6m centres in accordance with the Architects details and the manufacturers recommendations. Where this compromises the design of the blockwork / brickwork panels the contractor shall inform the Engineer prior to construction.

17. The contractor shall put forward his proposals for cold weather laying, no laying of bricks below 5 degrees Celsius.

SIZES OF STRUCTURAL ELEMENTS

Lateral support at roof level



Interruption of lateral support

2C37 Where an opening in a floor or roof for a stairway or the like adjoins a supported wall and interrupts the continuity of lateral support, the following conditions should be satisfied for the purposes of Section 2C:

a. the maximum permitted length of the opening is to be 3m, measured parallel to the supported wall, and

b. where a connection is provided by means other than by anchor, this should be provided throughout the length of each portion of the wall situated on each side of the opening, and

c. where connection is provided by mild steel anchors, these should be spaced closer than 2m on each side of the opening to provide the same number of anchors as if there were no opening, and

d. there should be no other interruption of lateral support.

Small single-storey non-residential buildings and annexe

2C38 Size and proportion

(i) General

The guidance given applies in the following circumstances:-

a. The floor area of the building or annexe does not exceed 36m²

b. The walls are solidly constructed in brickwork or blockwork using materials which comply with paragraphs 2C19 to 2C22.

c. Where the floor area of the building or annexe exceeds 10m² the walls have a mass of not less than 130 kg/m².

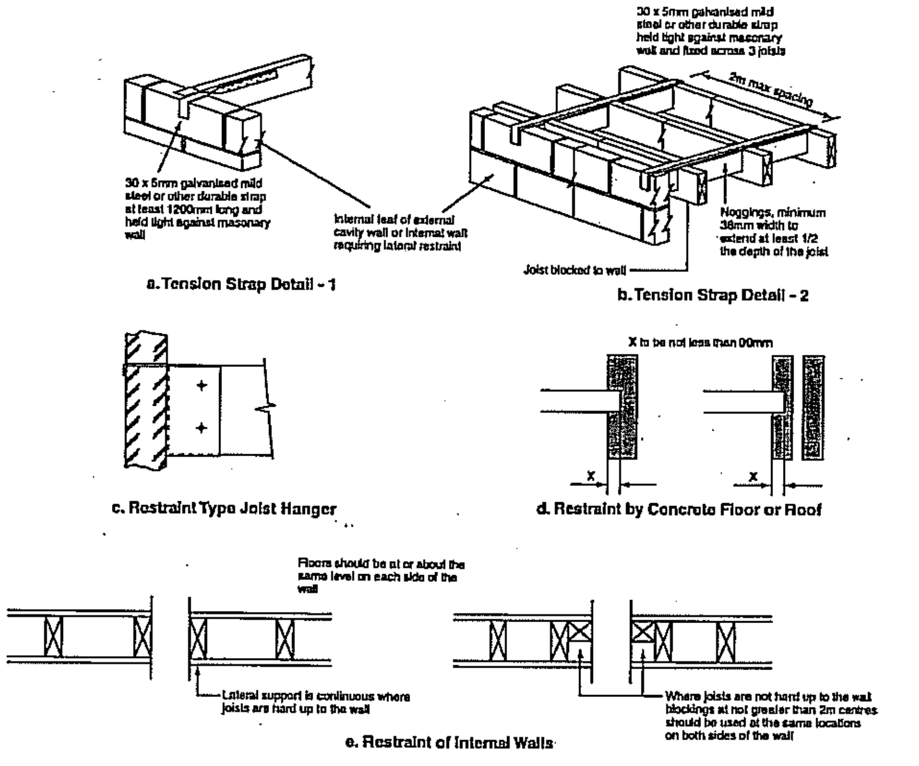
Note: There is no surface mass limitation recommended for floor areas of 10m² or less.

d. Access to the roof is only for the purposes of maintenance and repair.

e. The only lateral loads are wind loads.

SIZES OF STRUCTURAL ELEMENTS

Lateral support by floors



specifications including material references 1 or 3 (austenitic stainless steel). The declared tensile strength of tension straps should not be less than 8 kN.

Tension straps need not be provided:

a. in the longitudinal direction of joists in houses of not more than 2 storeys, if the joists are at not more than 1.2m centres and have at least 90mm bearing on the supported walls or 75mm bearing on a timber wall-plate at each end, and

b. in the longitudinal direction of joists in houses of not more than 2 storeys, if the joists are carried on the supported wall by joist hangers in accordance with BS EN 845-1 of the restraint type described in BS 5628: Part 1 and shown in Diagram 16(c), and are incorporated at not more than 2m centres, and

c. when a concrete floor has at least 90mm bearing on the supported wall (see Diagram 16(d)), and

d. where floors are at or about the same level on each side of a supported wall, and contact between the floors and wall is either continuous or at intervals not exceeding 2m. Where contact is intermittent, the points of contact should be in line or nearly in line on plan. (see Diagram 16(e))

2C36 Gable walls should be strapped to roofs as shown in Diagram 17(a) and (b) by tension straps as described in 2C35.

Vertical strapping at least 1m in length should be provided at eaves level at intervals not exceeding 2m as shown in Diagram 17 (c) and (d). Vertical strapping may be omitted if the roof:

a. has a pitch of 15° or more, and

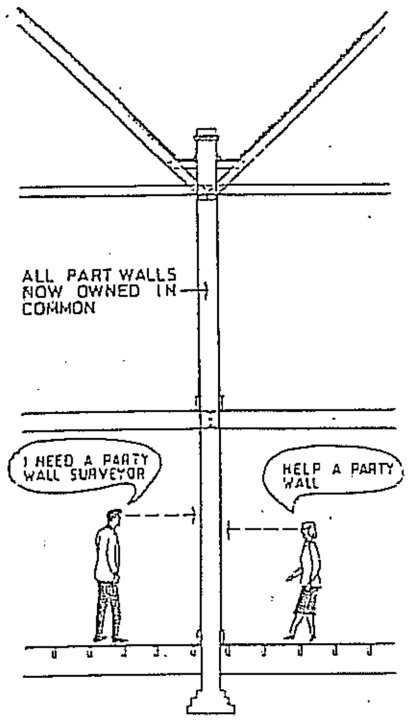
b. is tiled or slated, and

c. is of a type known by local experience to be resistant to wind gusts, and

d. has main timber members spanning onto the supported wall at not more than 1.2m centres.

PARTY WALL ETC ACT 1996

(EFFECTIVE 6/4/97)



THE ACT REQUIRES STRICT OBSERVANCE

PARTY WALL NOTICES REQUIRED FOR

- RAISING OR REBUILDING PARTY WALL
- INCREASED LOADING AND BEAMS
- DAMP PROOF INJECTION
- UNDERPINNING
- CHASING WALLS
- REMOVAL OF CHIMNEY BREASTS AND RAISE CHIMNEY STACK
- ANY OTHER WORKS WHICH AFFECT PARTY WALLS

THE ACT PROVIDES FOR

PARTY WALL AWARDS

COVERING:

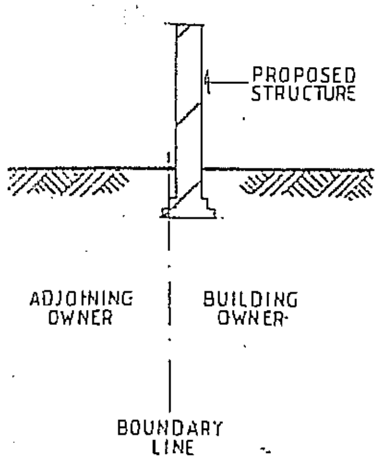
- PROPOSED WORKS & RESTRICTION ON WORKS.
- PROTECTION & SECURITY
- MAKING GOOD DAMAGE
- RECORDING CONDITION BEFORE WORKS START
- INSURANCE: DISPUTES PROCEDURE
- ADDITIONAL WORKS

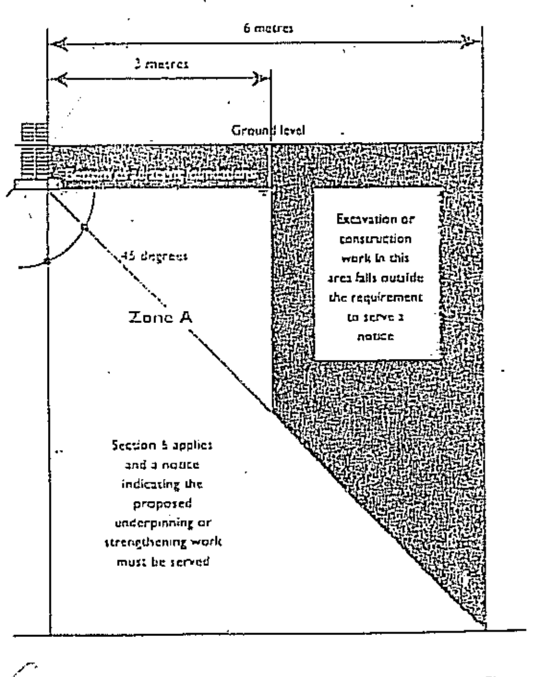
ADJOINING EXCAVATIONS

NOTICES REQUIRED FOR FOUNDATIONS WITHIN 3 METRES & 6 METRES OF ADJOINING PROPERTY.

LINE OF JUNCTION NOTICES

REQUIRED FOR CONSTRUCTION & REPAIR OF STRUCTURES BUILT TO BOUNDARY LINES





AND DESIGNS LTD

TABLE OF SPACING OF SPACERS AND CHAIRS TO BS 7973:2001

Size of bar reinforcement (mm)	Spacing of spacers and chairs not to exceed 50 d where d is the size of the bar reinforcement in mm which is supported by the spacer or chair. 50d =
8	400
10	500
12	600
16	800
20	1000
25	1250
32	1600

Welded steel fabric to BS 4483	Spacing of spacers and chairs supporting the fabric:-
Square Fabric	
A393	Not to exceed 500mm in each direction
A252	Not to exceed 500mm in each direction
A193	Not to exceed 500mm in each direction
A142	Not to exceed 500mm in each direction
Structural Fabric	
B1131	Not to exceed 500mm in each direction
B785	Not to exceed 500mm in each direction
B503	Not to exceed 500mm in each direction
B385	Not to exceed 500mm in each direction
B283	Not to exceed 500mm in each direction
Long Fabric	
C785	Not to exceed 500mm in each direction
C636	Not to exceed 500mm in each direction
C503	Not to exceed 500mm in each direction
C385	Not to exceed 500mm in each direction

APPENDIX E

Soil Borehole Records

REPORT ON A SITE INVESTIGATION

at

1 ELLERDALE ROAD, HAMPSTEAD, LONDON, NW3

for

MR G GALBERG

CONSULTING ENGINEERS: GTA

Report No 12/9705/KJC **October, 2012**

ALBURY S.I. LTD

Miltons Yard Petworth Road
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www.alburysi.co.uk

CONTENTS

FOREWORD

The following notes should be read in conjunction with the report. Any variations on the general procedures outlined below are indicated in the text.

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General

The recommendations made and opinions expressed in the report are based on the strata conditions revealed by the fieldworks as indicated on the boring and trialpit records, together with an assessment of the data from insitu and laboratory tests. No responsibility can be accepted for conditions, which have not been revealed by the fieldworks, for example, between borehole and/or trialpit positions. While the report may offer opinions on the possible configuration of strata, both between the excavations and below the maximum depth achieved by the investigation, these comments are for guidance only and no liability can be accepted for their accuracy. For investigations, which include environmental issues, the data obtained relate to the conditions which are relevant at the time of the investigation.

Boring Techniques

Unless otherwise stated, the light cable percussion technique of soft ground boring has been used. This method generally enables the maximum information to be obtained in respect of strata conditions, but a degree of mixing if some layered soils, for example, thin bands of coarse and fine granular soils, is inevitable. Specific attention is drawn to this occurrence where evidence of such a condition is available.

The penetration resistances quoted on the boring records have been determined generally in accordance with the procedure given in BS1377 : 1990. The suffix '+' donates that the results has been extrapolated from less than 0.3m penetration into undisturbed soil.

Routine Sampling

During construction of boreholes, sampling and insitu testing will be completed in general accordance with Eurocode EN 1997-2 : 2007 and BS5930 : 1999. Variations to this code of practice will only occur where the strata conditions preclude implementation or the contract specifies alternatives.

Samples which are required for environmental testing will be stored in suitable glass containers in accordance with current guidelines.

Groundwater

The groundwater observations entered on boring and trialpit records are those noted at the time of the investigation. The normal rate of progress does not usually permit the recording of any equilibrium water level for any one water strike. Moreover, groundwater levels are prone to seasonal variation and to changes in local drainage conditions. The table on each boring record shows the groundwater level at the quoted borehole and casing depths usually at the start and finish of a day's work. The word 'none' indicates that groundwater was sealed off by the borehole casing, or that no water was observed in the borehole.

Trialpits

The method of construction employed to form the trialpits is entered in their records. In general, it is not possible to extend machine excavated trialpits to depths significantly below the water table, especially in predominantly granular soils. Except for manually excavated pits, and unless otherwise stated, the trialpits have not been provided with temporary side support during their construction, hence personnel have not entered them and examined the insitu exposed strata.

Window Sampling

Window sampling comprises driving a probe into the ground. On extraction of the probe the strata encountered are logged and representative disturbed samples recovered. In general, window sampling cannot be completed in granular soils, or below the water table.

Laboratory Testing

Unless stated in the tests, all laboratory tests have been performed in accordance with the requirements detailed in BS1377 (1990) : Parts 1-9, or other standards or specifications that may by appropriate.

REPORT ON A SITE INVESTIGATION

at

1 ELLERDALE ROAD, HAMPSTEAD, LONDON, NW3

for

MR G GALBERG

CONSULTING ENGINEERS: GTA

Report No 12/9705/KJC **October, 2012**

Prepared by **K J Clark BSc Hons**
Senior Geotechnical Engineer

1.0 SYNOPSIS

This investigation has demonstrated that made ground overlies soils thought to be associated with the Bagshot Beds of late Eocene age. The groundwater observations noted at the time of the fieldworks indicate that this phenomenon should not constitute a significant engineering problem at this site.

It is understood that it is proposed to construct a new garden house within the rear garden of the existing property. The proposal will result in excavation of up 2m depth in order to accommodate the new structure. Made ground has been revealed at the location of the borehole to 2.9m depth. Hence, the foundations to the structure should be located at depths of the order of 3m. It is recommended that foundations placed in the Bagshot Beds are designed to accept a maximum increase in load of 100kPa. The structure should be designed and constructed as a water tight element.

2.0 INTRODUCTION

It is understood that it is proposed to construct a new garden house within the rear garden of the existing property at 1 Ellerdale Road, Hampstead. Consequently, a site investigation has been undertaken in order to ascertain the nature and engineering properties of the soils underlying this site, and to obtain data which will assist in the formulation of a safe and economical foundation solution.

The programme of this investigation comprised the construction of a light cable percussive or shell and auger borehole. Due to limited access to the area of the proposed development a demountable boring unit was utilised. During this work, samples were recovered for further examination and laboratory testing. This report describes the work undertaken, presents the information obtained and discusses the ground conditions with respect to foundation design and construction. A copy of the order for these works is presented as Appendix 1. This report is for the benefit of the Client alone and cannot be assigned to a third party without the consent of Albury SI Ltd.

3.0 FIELDWORKS

The borehole was constructed on 3rd October, 2012, at a position as shown on the site plan, drawing no 12/9705/1, which is presented in Appendix 2 to this report. The salient details of this drawing have been extracted from a site layout plan supplied by the Client's representative.

The depths and descriptions of the strata encountered in the borehole are given on the record in Appendix 3 to this report. This records note the depths at which samples were taken, the results of standard penetration test and any groundwater observations noted at the time of the fieldworks.

4.0 GEOLOGY AND STRATA CONDITIONS

An examination of the 1:50,000 British Geological Survey map of the area, together with the relevant Handbook of Regional Geology, suggests that the site is underlain by Bagshot Beds of late Eocene age. This deposit consists of fine grained soils.

A study of the borehole record indicates that made ground, varying in composition from gravel/sand and brick to brown/grey very sandy clay with gravel and brick fragments was noted at the investigatory location. This fill material was proved to 2.9m depth.

Brown clayey sand with very occasional gravel or very sandy clay were encountered beneath the made ground and were proved to a depth of 6m. Brown very sandy clay with partings of sand was exposed below the clayey sand/very sandy clay and was shown to extend to the concluding depth of the borehole at 9m. It is suggested that these soils are associated with the Bagshot Beds.

No groundwater strikes were noted during the siteworks completed at this site. Temporary casing was installed to 6m depth. On completion of the borehole the casing was withdrawn. The borehole was allowed to remain open for a period of time on its completion. The borehole was noted to be dry at this time.

5.0 LABORATORY TESTING

A programme of laboratory testing has been undertaken and the results are presented as Appendix 4 to this report. Each type of test is summarised below, and the results obtained have been used to assist in the formulation of the discussion of ground conditions.

5.1 Particle Size Distribution

Samples of the soils encountered have been subjected to sieve and sedimentation analysis in order to ascertain the soils particle distribution and establish the soils clay fraction. The results of this work are presented in the form of grading curves.

5.2 Index Properties

The liquid and plastic limits of a sample of the soils have been determined. This work indicates that the soil sample tested is of intermediate plasticity. The plasticity index result has been corrected for the percentage of granular soil that is retained on the 425µm sieve. The percentage retained was 55%. Hence, the corrected plasticity index indicates that the soil analysed can be regarded as being of a non-shrinkable nature.

5.3 Triaxial Compression

The undrained shear strength characteristics of a sample of the soils encountered have been determined by testing specimens in the triaxial compression apparatus. A cohesion of 80kPa has been established which is indicative of a stiff condition insitu for a purely cohesive soil.

5.4 Oedometer- Consolidation - Heave

The one dimensional settlement heave characteristics of a sample of the soils underlying this site has been determined by testing a specimen in the Terzaghi Oedometer or Consolidation apparatus. The test was made by preparing the specimen in the oedometer cell and applying an initial load which corresponds to the approximate existing overburden pressure. Two cycles of consolidation loading were then applied followed by two unloading cycles taking the final load back to the initial overburden pressure. The results of this unloading cycles have been used to calculate the coefficient of volume increase which is quoted in the test results. The results obtained suggest that low magnitudes of heave may be expected. The results also indicates that movements would occur in a short period of time.

5.5 Chemical Analyses - Soluble Sulphates & pH Values

Samples of the soils encountered at this site have been subjected to chemical analyses in order to determine their soluble sulphate content and pH values. Under the conditions of this work low to moderate concentrations of soluble sulphate contents have been recorded in association with near neutral pH values.

6.0 DISCUSSION OF GROUND CONDITIONS

It is understood that it is proposed to redevelop the site by the construction of a new single storey garden house. At the time of the preparation of this report, no precise information was available with regard to the likely structural loadings generated by the proposed construction. It is further understood that the structure will be set in to the garden by up to 2m depth. Moreover, it is possible that a swimming pool may also be incorporated in the development.

It cannot be recommended that major structural foundations be located within the made ground revealed by this investigation. Soils of this origin are frequently present in a weak and variable condition, such that unacceptable settlement could occur even under the action of light loading intensities. The above precaution need not necessarily be applied to light ancillary structures, which will be formed structurally discrete from the main development and in which a greater degree of settlement can be tolerated.

Made ground has been noted to be present at the borehole location. Hence, it is likely that the foundations to the proposed structure will be constructed at depths of the order of 3m in order to locate footings within naturally occurring soils thought to be associated with the Bagshot Beds. It is recommended that new foundations within these soils can be designed to apply a maximum increase in load of 100kPa. At this loading intensity a factor of safety of 3 against general shear failure will be operative. Moreover, control will provided over settlements. The nature of the soils encountered suggests that these movements should be sensibly complete in the short-term as opposed to an extended period of time.

The groundwater observations noted at the time of the fieldworks suggest that this phenomenon should not constitute a significant engineering problem at this site. Nevertheless, should slight seepages be encountered or surface water run off drain into excavations, then these minor amounts should be removed expeditiously by the construction of sumps from which water can be pumped. It will be prudent to design and construct any structures below ground level as water tight units.

With regard to the construction works, it is evident that it is unlikely that it will be possible to construct any sort of strutted cofferdam in order to provide clear access for construction works in view of the limited working space/access and presence of adjacent existing buildings. Therefore, it is assumed that the basement will have to be excavated and the retaining walls constructed using manual techniques in panels. It should be possible to construct the necessary excavation in panels of convenient width and depth. This work will be completed by constructing new foundations and include underpinning of the existing structures where necessary. Evidently, where appropriate support to excavation sides should be provided

In the design of the retaining walls account should be taken of the earth pressures and any surcharge loadings that will be applied to the walls. In the design of such a structure, it is normally necessary to employ the use of effective stress parameters such that the long term stability of the structure can be assured. Bearing this in mind it is recommended that the following design parameters are employed in the calculations.

Table 1 – Retaining Wall Design.

Soil Parameter	**Effective Cohesion**	**Effective Angle of Friction**	**Soil Density**
Made ground	0	15	1850
Bagshot Beds	2	25	1900

In view of the presence of made ground to approximately 3m, it is evident recommended that a fully suspended floor slab is adopted in the design of the proposed redevelopment.

The excavation of soil will result in a reduction in the overburden load to the underlying strata of approximately 20kPa. The suspended floor slab will ensure that any heave of the soils underlying the site will not represent an engineering problem. However, this figure is likely approach 40kPa should a swimming pool be incorporated at the new floor level - assuming a 2m deep construction. Evidently, it is likely that the pool slab can be constructed on the naturally occurring soils. This increased excavation may result in the development of elastic movement together with the potential for long-term heave under the revised stress conditions when the development is completed.

The magnitude of long-term uplift forces that may be applied to the swimming pool is difficult to predict. Computer programmes are available which attempt to model the problem. However, the complex nature of the proposed structure and difficulties in assuming the soil parameters would limit the validity of the calculations.

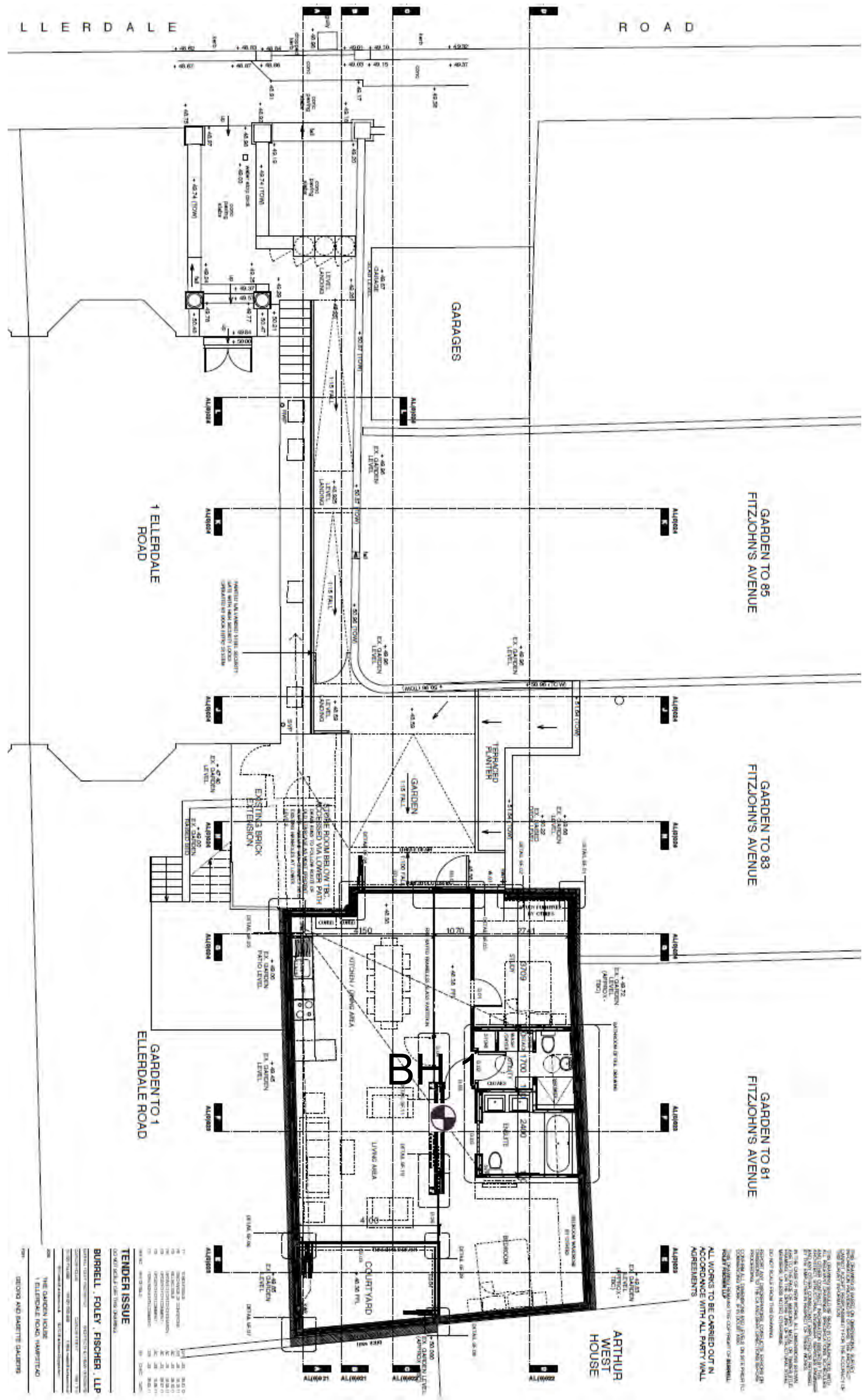
It should be appreciated that the magnitude of heave is dependent upon the loads and stiffness of the structure and performance of the underlying strata. The soils at this site are thought to comprise Bagshot Beds, the upper levels of which generally comprise brown/grey clayey sand. The results of laboratory analysis completed on the upper levels of the Barton Beds suggest that the soils are of non- shrinkable potential which implies that the soil does not contain a significant amount of active clay minerals. Hence, it is considered that the completed structure is likely to experience nominal heave/uplift force derived from the clayey sand which underlies this site.

7.0 EFFECT OF SULPHATES

The information obtained from this investigation has been compared with the criteria proposed in BRE Special Digest 1; 2005, Edition, Concrete in Aggressive Ground. Using the information in Table C1 (natural ground) of this publication the Aggressive Chemical Environment for Concrete Classification is AC-1s, which coincides with a Design Sulphate Class DS-1. This Design Sulphate Class can be used to establish the design mix for buried concrete in accordance with Part D of the Digest.

A P P E N D I X 2

Site Plan



Site:	ELLERDALE ROAD, HAMPSTEAD
Drawing No:	12/9705/1
Title:	SITE PLAN
Scale:	NTS
Revision:	A
Issue Date:	1.0 – 18/4/2011

Albury S.I. Ltd
Miltons Yard
Petworth Road
Witley,
Surrey, GU8 5LH
www.alburysi.co.uk

A P P E N D I X 3

Boring Records

Albury S.I. Ltd Petworth Road, Witley, Godalming, Surrey, GU8 5LH		Borehole No	1
Contract	Ellerdale Road, Hampstead	Report No	12/9705/KJC
Client	Mr G Galberg	Ground Level	mOD
Site Address	1 Ellerdale Road, Hampstead, London, NW3	Boring Commenced	03/10/12
		Boring Completed	03/10/12

Type and diameter of boring: Light cable percussion (shell and auger): 150mm diameter

Water Strikes, m		Water levels recorded during boring, m							
1.	none	Date	03/10	03/10					
2.		Hole Depth	9.00	9.00					
3.		Casing Depth	6.00	none					
4.		Water Level	none	none					

Remarks

Excavation of starter pit to clear services.

Samples or tests		SPT			Strata Description
Type	Depth, m	N	Depth	Legend	
D	0.20				Made ground (gravel/sand, gravel and brick)
			0.40		
B	0.50				Made ground (brown/grey very sandy clay with gravel and occasional brick particles)
D	1.00-1.50	8			
D	1.75				
D	2.00-2.50	8	2.00		
					Made ground (brown/grey clayey sand with gravel, pockets of clay and very occasional brick particles)
D	2.75				
			2.90		
D	3.00-3.50	11			
					Medium dense brown/grey clayey silty sand with very occasional gravel
D	4.00		4.00		
					Brown very sandy clay with partings of sand
U	4.50-5.00		4.50		
					Brown/grey clayey sand/very sandy clay with gravel
D	5.50		5.50		
					Brown clayey sand
D	6.00-6.50	16	6.00		
					Stiff brown very sandy clay with partings of sand
U	6.50-7.00				
D	7.00				
D	7.50				
D	8.50-9.00	19			
			9.00		

Sampling Code: **U**- Undisturbed, **B** - Large Disturbed, **D** - Small Disturbed, **W**- Water Sample, **(U)***- Non-recovery of undisturbed sample

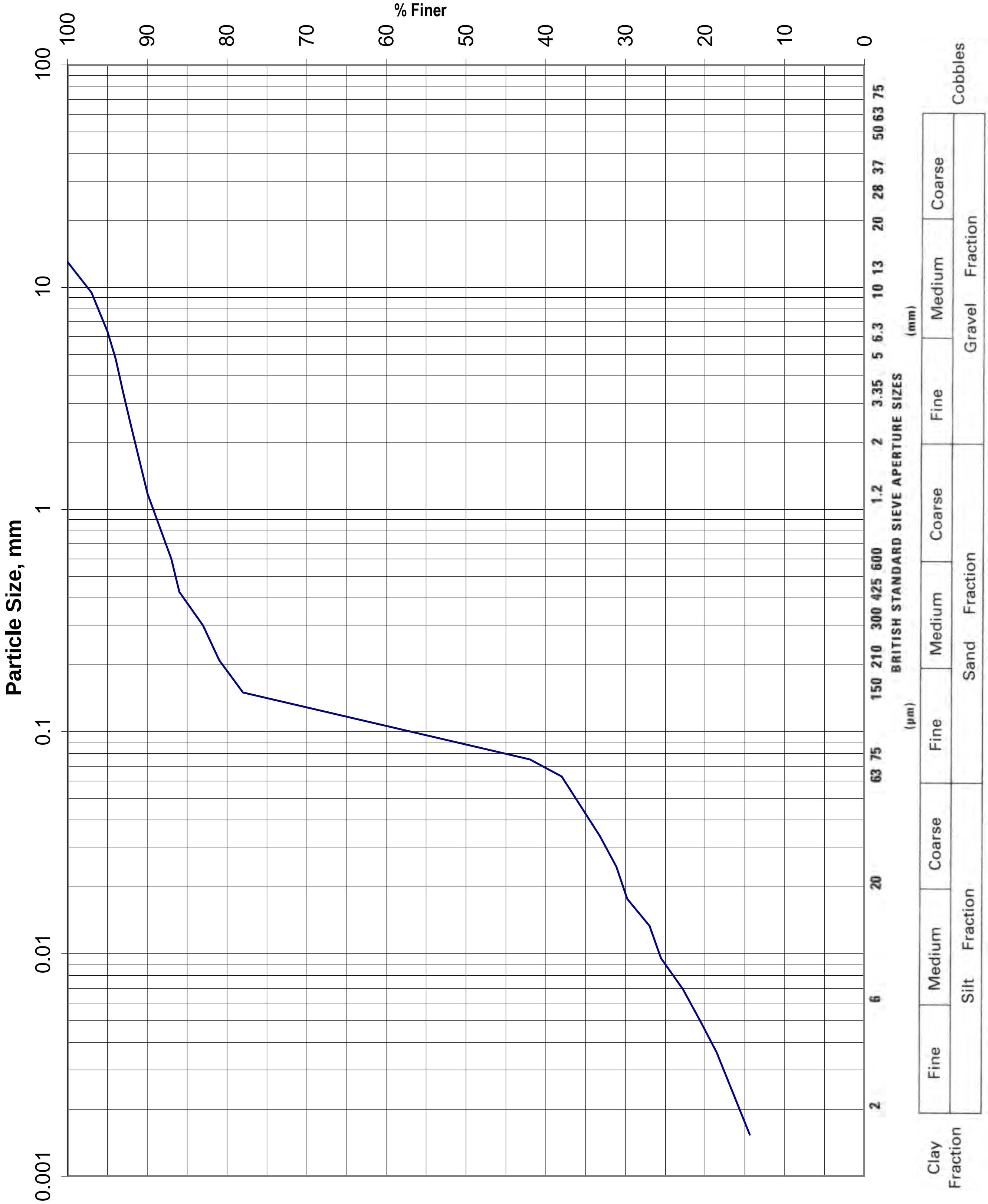
A P P E N D I X 4

Laboratory Test Results

PARTICLE SIZE DISTRIBUTION - GRADING CURVE

Contract: Ellerdale Road, Hampstead

Report No. 12/9705/KJC



Borehole No. 1

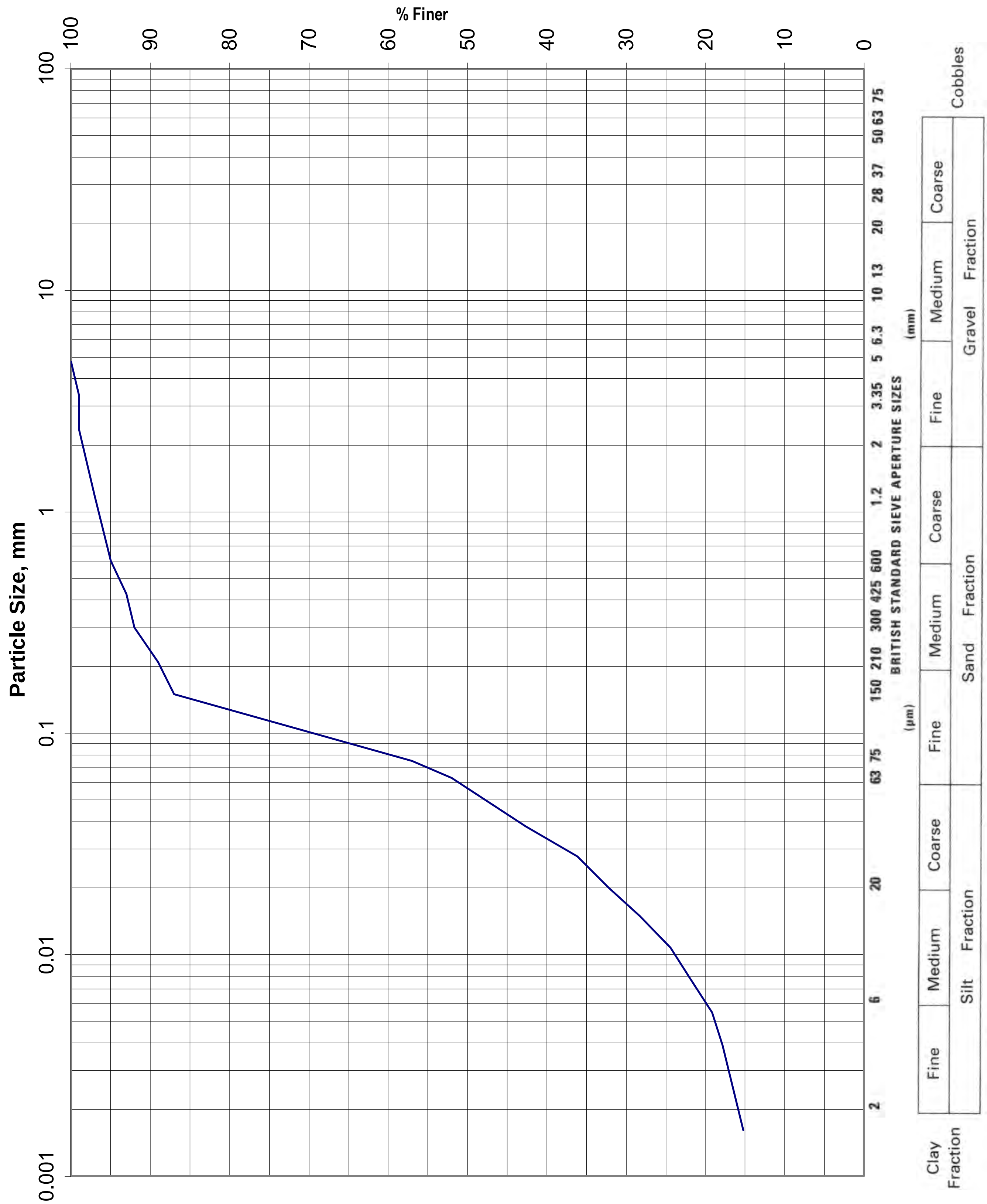
Depth of Sample, m: 1.75

Visual Description: Made ground (grey/brown clayey sand with gravel and brick fragments)

Albury S. I. Ltd Miltons Yard Petworth Road Witley Surrey GU8 5LH

PARTICLE SIZE DISTRIBUTION - GRADING CURVE

Contract: Ellerdale Road, Hampstead	Report No. 12/9705/KJC



Borehole No. 1	Depth of Sample, m: 3.00-3.50

Visual Description: Brown/grey clayey silty sand with very occasional gravel

PARTICLE SIZE DISTRIBUTION - GRADING CURVE

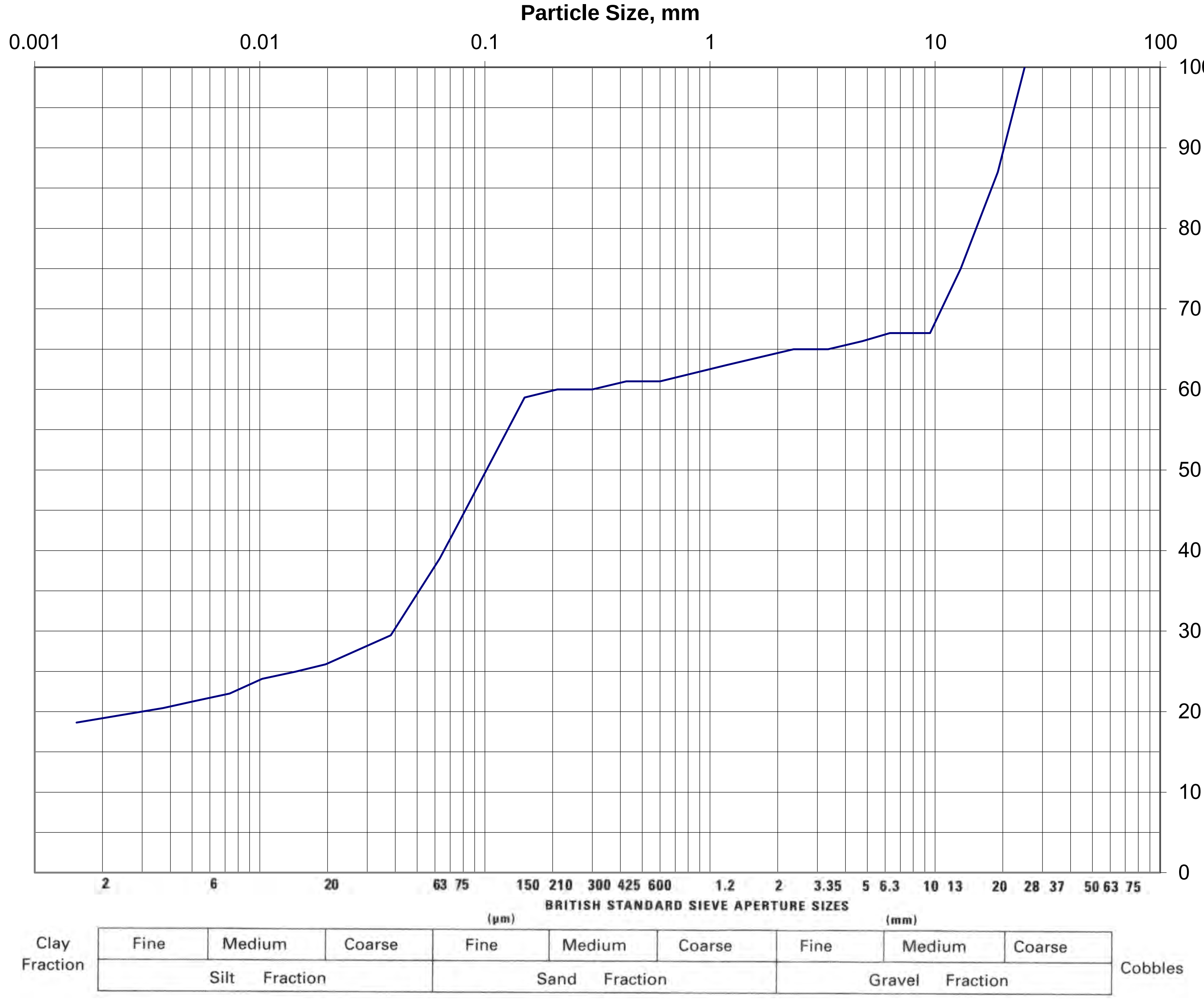
Contract: Ellerdale Road, Hampstead

Report No. 12/9705/KJC

Borehole No. 1

Depth of Sample, m: 4.50-5.00

Visual Description: Brown clayey sand with gravel



Albury S. I. Ltd Miltons Yard Petworth Road Witley Surrey GU8 5LH

RESULTS OF CONSOLIDATION TESTS

Contract: Ellerdale Road, Hampstead
Report No: 12/9705/KJC

BH no	Depth of Sample m	Description of Sample	INDEX PROPERTIES				TRIAXIAL COMPRESSION							CONSOLIDATION			REMARKS
			Liquid Limit %	Plastic Limit %	Plasticity Index %	Soil Classif-cation	Code	Lateral Pressure kPa	Com-pressive Strength kPa	Cohesion kPa	Angle of Friction (degrees)	Bulk Density kg/m^3	Water Content (% dry wt)	Pressure Range kPa	Coefficient of Volume Decrease mm^2/kN	Coefficient of Consolidation $m^2/year$	
								150									Specimens failed during preparation
1	4.50-5.00	Brown/grey clayey sand/very sandy clay with gravel (55% retained on 425µm sieve Corrected PI = 9%)	37	17	20	CI	38U	300					10.1				
								450									
								150	155			1925	28.5	150-300	175	1.24	
	6.50-7.00	Brown very sandy clay with partings of sand					38U	300	175	80	0	1925	28.3	300-600	135	0.83	
								450	150			1960	27.8	600-300	-25	-2.90	
														300-150	-80	-0.70	

Sheet No 1 of 1

TRIAXIAL COMPRESSION TEST CODE: **38-38mm dia specimen** **100-100mm dia specimen** **U-Undrained** **CD-Consolidated Drained** **CU-Consolidated Undrained**
P-Pore water pressure measurement **M-Multistage** **F-Functional** **R-Remoulded** **LV-Laboratory Vane Test**

RESULTS OF CHEMICAL ANALYSES

Determination of Sulphate Content and pH value

Contract: Ellerdale Road, Hampstead **Report No:** 12/9705/KJC

BH No	Depth of sample, m	Description	Concentrations of Sulphates expressed as SO_4			pH value
			In soil		In ground-Water *g/l*	
			Total SO_4 (%)	2:1 water:soil extract *g/l*		
1	1.75	Made ground		<0.25		7.4
	3.00-3.50	Clayey sand		<0.25		7.3
	6.50-7.00	Very sandy clay		<0.25		5.8

APPENDIX F

Planning Decision Notice

Camden

Development Control
Planning Services
London Borough of Camden
Town Hall
Argyle Street
London WC1H 8ND

Tel 020 7974 4444
Fax 020 7974 1680
Textlink 020 7974 6866

env.devcon@camden.gov.uk
www.camden.gov.uk/planning

Mr Andrew de Carteret
Burrell Foley Fischer LLP
Carlow House
Carlow Street
London
NW1 7LH

Application Ref: **2011/4005/P**
Please ask for: **Elaine Quigley**
Telephone: 020 7974 **5101**

2 November 2011

Dear Sir/Madam

DECISION

Town and Country Planning Acts 1990 (as amended)
Town and Country Planning (General Development Procedure) Order 1995
Town and Country Planning (Applications) Regulations 1988

Variation or Removal of Condition(s) Granted Subject to a Section 106 Legal Agreement

Address:
The Garden House
1 Ellerdale Road
London
NW3 6BA

Proposal:
Amendments to amended planning permission granted 24/05/2011 (ref: 2010/5841/P) for the erection of a new dwelling house on land to the rear 81 Fitzjohn's Avenue to include increase in site area for enlarged garden, increase in built footprint of house and rebuild of boundary walls.
Drawing Nos: Site location plan; BFF/777 AL(0)100.P4, 200.P6, 210.P5, 300.P4, 301.P3, 400.P3, 401.P3, 402.P3, 410.P2, 002.P1, and 950.P1.

The Council has considered your application and decided to grant permission subject to the following condition(s):

Condition(s) and Reason(s):

1 The development hereby permitted shall be carried out in accordance with the

following approved plans Site location plan; BFF/777 AL(0)100.P4, 200.P6, 210.P5, 300.P4, 301.P3, 400.P3, 401.P3, 402.P3, 002.P1, 901.P4, 950.P1, 410.P2;

Reason:
For the avoidance of doubt and in the interest of proper planning.

2 Prior to the commencement of development, details of the design of building foundations and new wall footings and the layout, with dimensions and levels, of service trenches and other excavations on site in so far as these items may affect trees on or adjoining the site, shall be submitted to and approved in writing by the Council as the local planning authority. The relevant part of the works shall not be carried out otherwise than in accordance with the details thus approved.

Reason: To ensure that the Council may be satisfied that the development will not have an adverse effect on existing trees and in order to maintain the character and amenities of the area in accordance with the requirements of policy CS15 of the London Borough of Camden Local Development Framework Core Strategy.

Informative(s):

1 Reasons for granting permission.

The proposed development is in general accordance with the London Borough of Camden Local Development Framework Core Strategy, with particular regard to policies CS5 (Managing the impact of growth and development), CS14 (Promoting high quality places and conserving our heritage), CS15 (Protecting and improving our parks and open spaces & encouraging biodiversity) and CS17 (Dealing with our waste and encouraging recycling); and the London Borough of Camden Local Development Framework Development Policies, with particular regard to policies DP19 (Managing the impact of parking), DP24 (Securing high quality design), DP25 (Conserving Camden's heritage), DP26 (Managing the impact of development on occupiers and neighbours), DP27 (Basements and lightwells) and DP29 (Improving access). For a more detailed understanding of the reasons for the granting of this planning permission, please refer to the officers report.

2 Your proposals may be subject to control under the Party Wall etc Act 1996 which covers party wall matters, boundary walls and excavations near neighbouring buildings. You are advised to consult a suitably qualified and experienced Building Engineer.

3 Your proposals may be subject to control under the Building Regulations and/or the London Buildings Acts which cover aspects including fire and emergency escape, access and facilities for people with disabilities and sound insulation between dwellings. You are advised to consult the Council's Building Control Service, Camden Town Hall, Argyle Street WC1H 8EQ, (tel: 020-7974 2363).

4 Noise from demolition and construction works is subject to control under the Control of Pollution Act 1974. You must carry out any building works that can be

heard at the boundary of the site only between 08.00 and 18.00 hours Monday to Friday and 08.00 to 13.00 on Saturday and not at all on Sundays and Public Holidays. You are advised to consult the Council's Compliance and Enforcement team [Regulatory Services], Camden Town Hall, Argyle Street, WC1H 8EQ (Tel. No. 020 7974 4444 or on the website http://www.camden.gov.uk/ccm/content/contacts/council-contacts/environment/contact-the-environmental-health-team.en or seek prior approval under Section 61 of the Act if you anticipate any difficulty in carrying out construction other than within the hours stated above.

5 Your attention is drawn to the fact that there is a separate legal agreement with the Council which relates to the development for which this permission is granted. Information/drawings relating to the discharge of matters covered by the Heads of Terms of the legal agreement should be marked for the attention of the Planning Obligations Officer, Sites Team, Camden Town Hall, Argyle Street, WC1H 8EQ

6 You are reminded of the need to comply with the conditions attached to the original planning permission dated 28/05/2010 (ref. 2010/0861/P). You are also advised to take note of the informatives attached to that original decision notice.

Your attention is drawn to the notes attached to this notice which tell you about your Rights of Appeal and other information.

Yours faithfully

Rachel Stopard

Rachel Stopard
Director of Culture & Environment

It's easy to make, pay for, track and comment on planning applications on line. Just go to www.camden.gov.uk/planning.