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REPORT

12 Keats Grove

Structural Engineer's Basement Impact Assessment

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Job Number:	28412

Date	Version	Notes/Amendments/Issue Purpose
October 2019	1	Issued for planning
November 2019	2	Updated to reflect retention of boundary wall
November 2019	3	Introduction, section 5 & Sketch 3 Updated
December 2019	4	Drainage note added to external works

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Contents

Accompanying Documents

Existing and Proposed Architectural Drawings	Chris Dyson Architects
Ground Investigation and Basement Impact Assessment Report Note: Includes Ground Movement Analysis	Geotechnical and Environmental Associates
Arboriculturalist's Report	ADC
Planning Stage Construction Method Statement	Price & Myers

1 Introduction

This report has been prepared in support of the Planning application to alter this house which includes extending the basement on both sides of this four-storey late Georgian detached house. The house is in the Hampstead conservation area, and is listed Grade II.

The proposed alterations include demolishing and rebuilding two wings to the house with a basement under each. The wings are a relatively modern addition to the original building and have been remodelled several times over the years. The historic central part of the house has an original lower ground floor level under its entire footprint. In order to coordinate with the existing lower ground floor and raised ground floor levels, the wings will be set such that the proposed basement level is approximately 1m lower than the existing lower ground level; see site location and relevant historic maps in Appendix A. The house is in the Hampstead conservation area, but is not listed.

2 Desk Study

Site History

With reference to Figure 2 and Figure 3 the site has been a house since before 1866, and is assumed to have been previously undeveloped. As can be seen, several of the houses on the street were demolished and some rebuilt in the period between the two maps being produced. Additionally, there is record of WW2 bombing in the area and that the property suffered some minor blast damage, see Figure 4 and Figure 5. A preliminary UXO Risk Assessment was carried out before the Site Investigation and gives recommendation that a more detailed assessment should be carried out which will be completed before construction commences.

Geology

The site is underlain by London Clay formation as was found in historic record, see Figure 6, and confirmed with the site investigation carried out. The site investigation comprised trial pits and several boreholes which extended to a depth greater than the proposed excavation - ground water was not encountered.

Further information can be found in the accompanying report(s) produced by GEA which cover Geology and Hydrogeology, and includes their Interpretative Site Investigation report.

3 Hydrology, Drainage and Flood Risk

The site is not within the catchment of the Hampstead Heath Pond Chain, which is approximately 130m to the north-east of the site, see Figure 7.

The site surface area is currently approximately 1510m^2, of which approximately 700m^2 is permeable. The property drains to a combined drainage system which discharges to the combined public sewer in Keats Grove. Refer to Drainage CCTV Survey drawing, Figure 8, for existing drainage arrangements and Thames Water Asset search, see Figure 9.

The proposed site surface area will be unchanged. The footprint of the proposed building footprint is ~5m^2 larger than the existing however this area is already hardstanding and so there will be no increase in hardstanding as a result. Refer to the accompanying existing and proposed architectural drawings.

The site is located in Flood Zone 1 and is not at risk of flooding. The site is not at risk of flooding from rivers or the sea, surface water flooding or reservoir flooding. Refer to Environment Agency Flood Risk map, Figure 10, and Surface Water Flooding map, see Figure 11.

The site is not within a Critical Drainage Area.

4 CPG Basements - Screening, Scoping and Additional Assessments

Refer to Geotechnical and Environmental Associates, GEA's accompanying report for Screening, Scoping and Basement Impact Assessment. As identified in this report, part of the works will be taking place within a tree root protection zone and the impact of this is assessed in the accompanying Arboricultural Report.

5 Construction Methodology

See Appendix C for the proposed construction type and sequencing; and Appendix D for supporting calculations.
This section is to be read in conjunction with the Construction Management Plan.

Substructure
The basement structures will be formed in an underpinning construction method with base and ground floor slab to form a stiff concrete box. This is due to the relatively shallow depth of the excavation required and as it allows retention of existing structures. Notably, all of the walls that are to be underpinned are that of the Building Owner and therefore are not Party Structure.

Superstructure
The wings will be single storey above ground and are currently proposed to be constructed in loadbearing masonry with timber and steel roof structures.
Where the existing boundary wall with number 12A is retained this will be provided with temporary restraint during the work until this wall is tied back to the new roof structure.

External Works
There are no retaining walls proposed in the landscaping works. The proposed basements will conflict with existing below ground drainage, hence this will be reconfigured during the works. Existing drainage will be diverted or pumped as required during the works to ensure that the system remains in operation all the time. Details of both any temporary drainage diversions and the new proposals will be agreed through the Party Wall process.

Codes and Standards
The works will be designed in accordance with the relevant British Standards.

Loadings

Area	Load (kN/m^2)
Floors	2.5
Roofs	0.8
Retaining - surcharge	10

Design Fire Periods
The concrete structure will be designed to provide an inherent fire resistance of 60mins. Fire protection of the single storey superstructure will be by lining/encasement to the Architect's details.

Disproportionate Collapse
Class 1 to Part A3 – no special measures required.

Calculations
Conservative geotechnical parameters for soil density, cohesion and an accidental scenario of retained water level have been adopted for the design. Heave of the clays has also been considered.

Damage Classification
A ground movement analysis has been carried out by GEA as part of their report and should be referred to for the impact on 12 Keats Grove and the surrounding buildings. Refer to section 10 of their report for the ground movement analysis and section 11 for the resulting damage assessment; from which they conclude in section 12: “The analysis has concluded that the predicted damage to the neighbouring properties from the construction of the proposed basements would be largely ‘Negligible’ with one exception at ‘very slight’.”

This relates to damage on the Burland scale, a classification given in Table 6.4 of CIRIA report C760, where damage Category 0 is ‘Negligible’ and damage Category 1 is ‘very slight’.

Appendix A

Site location and Historic Maps

Figure 1: Site location Google maps 2019

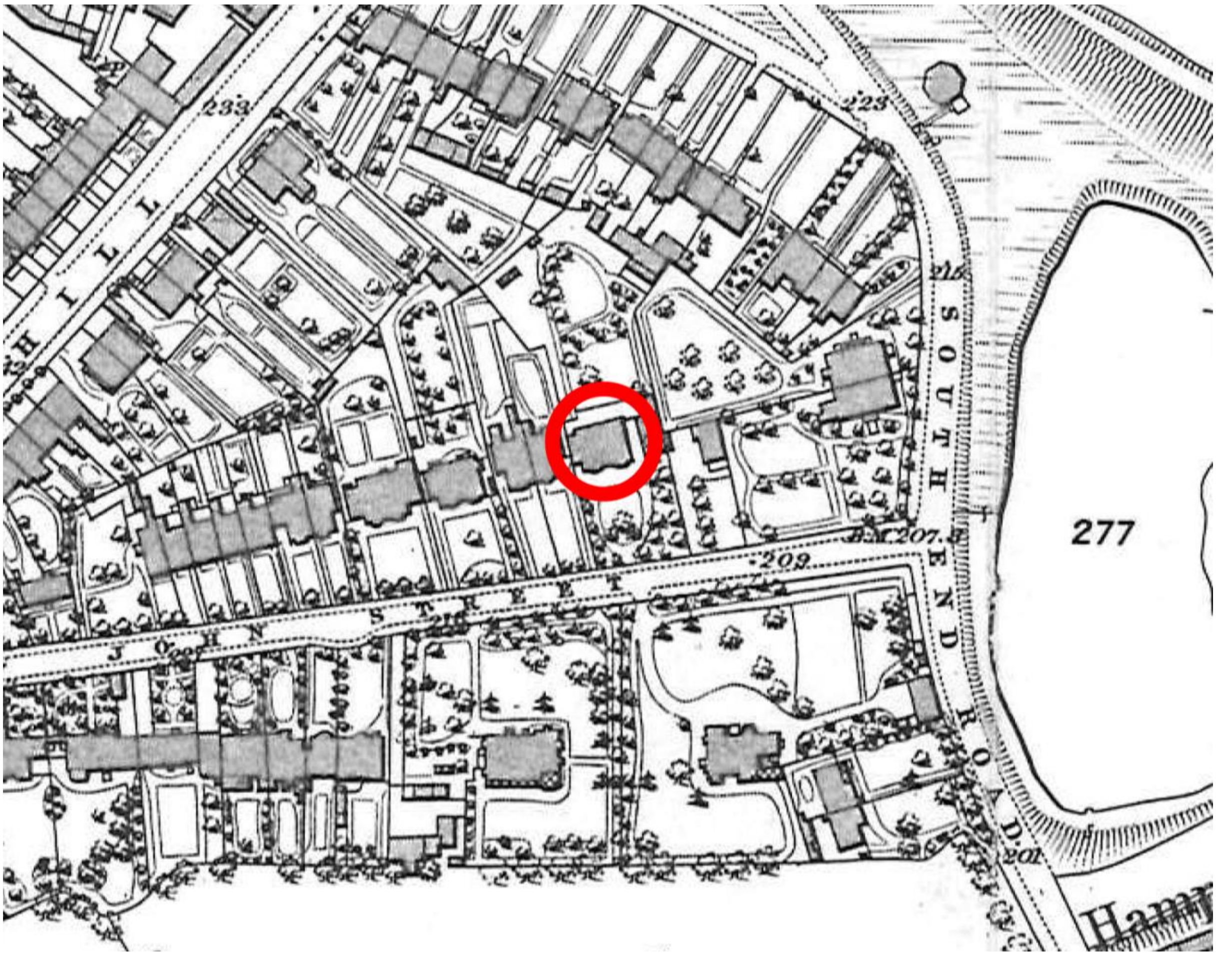

Figure 2: Extract from OS historic map of London 1866

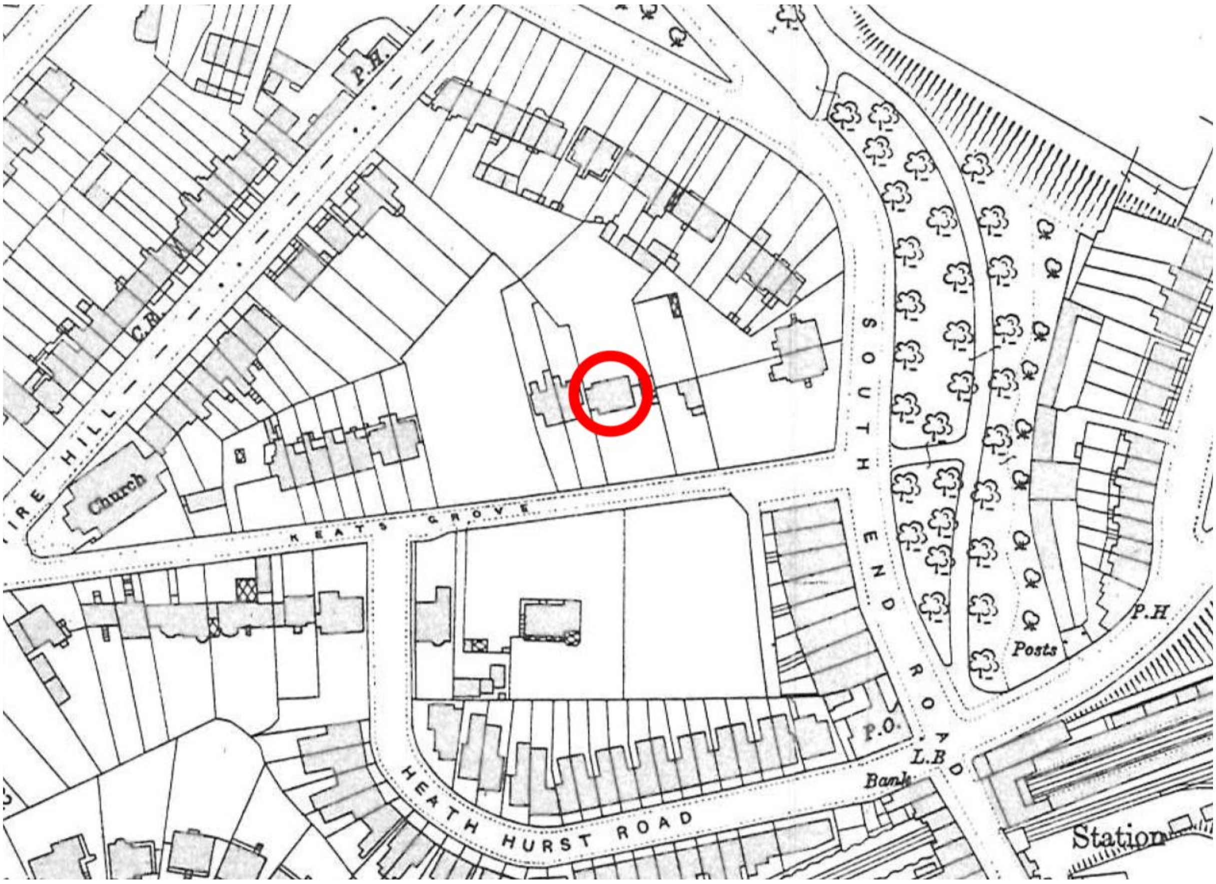

Figure 3: Extract from OS historic map of London 1915

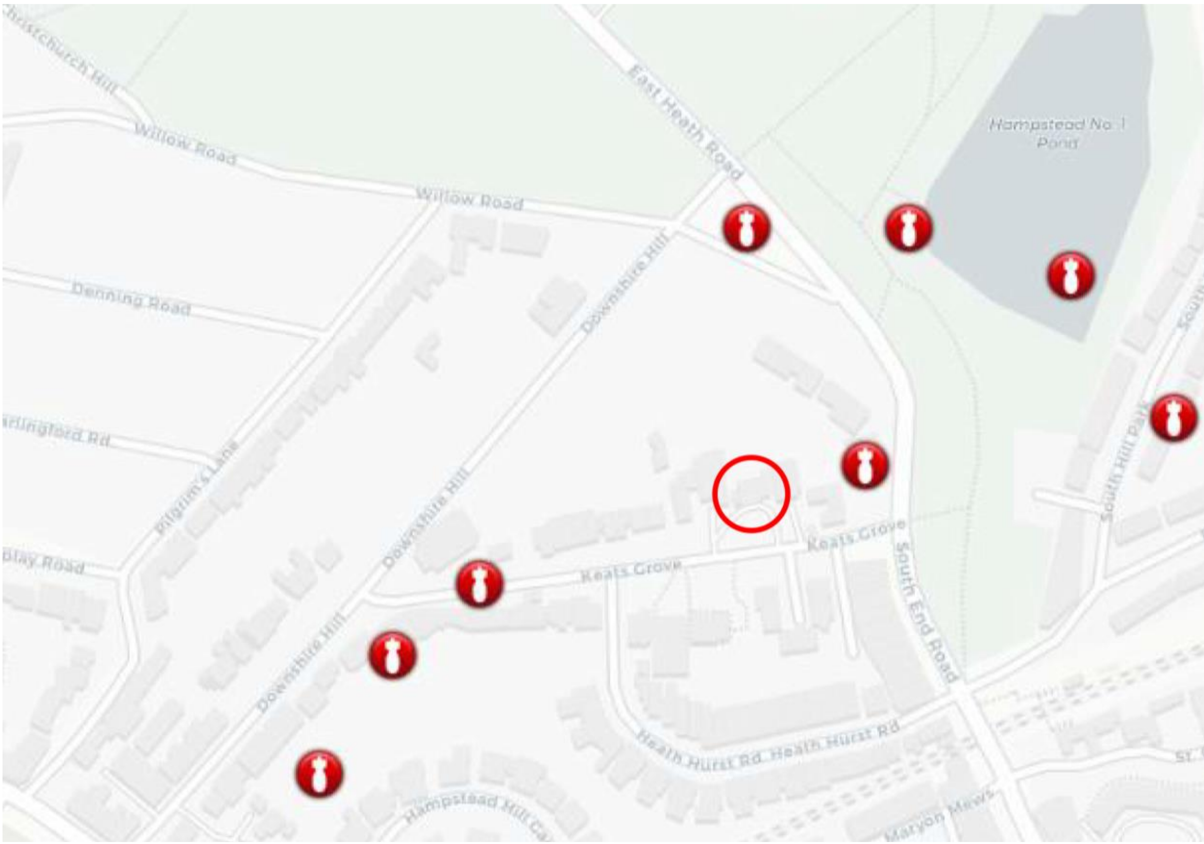

Figure 4: WW2 bomb locations: bombsight.org

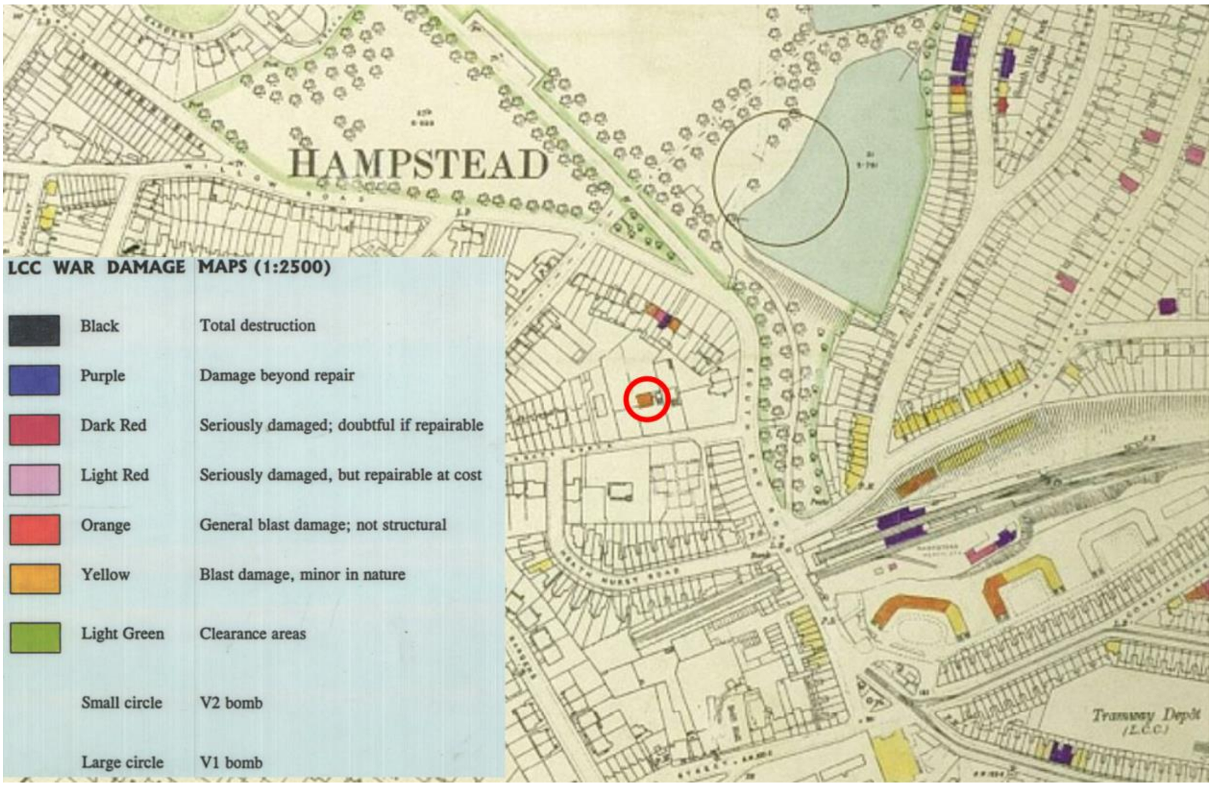

Figure 5: Bomb damage map: LCC war damage maps

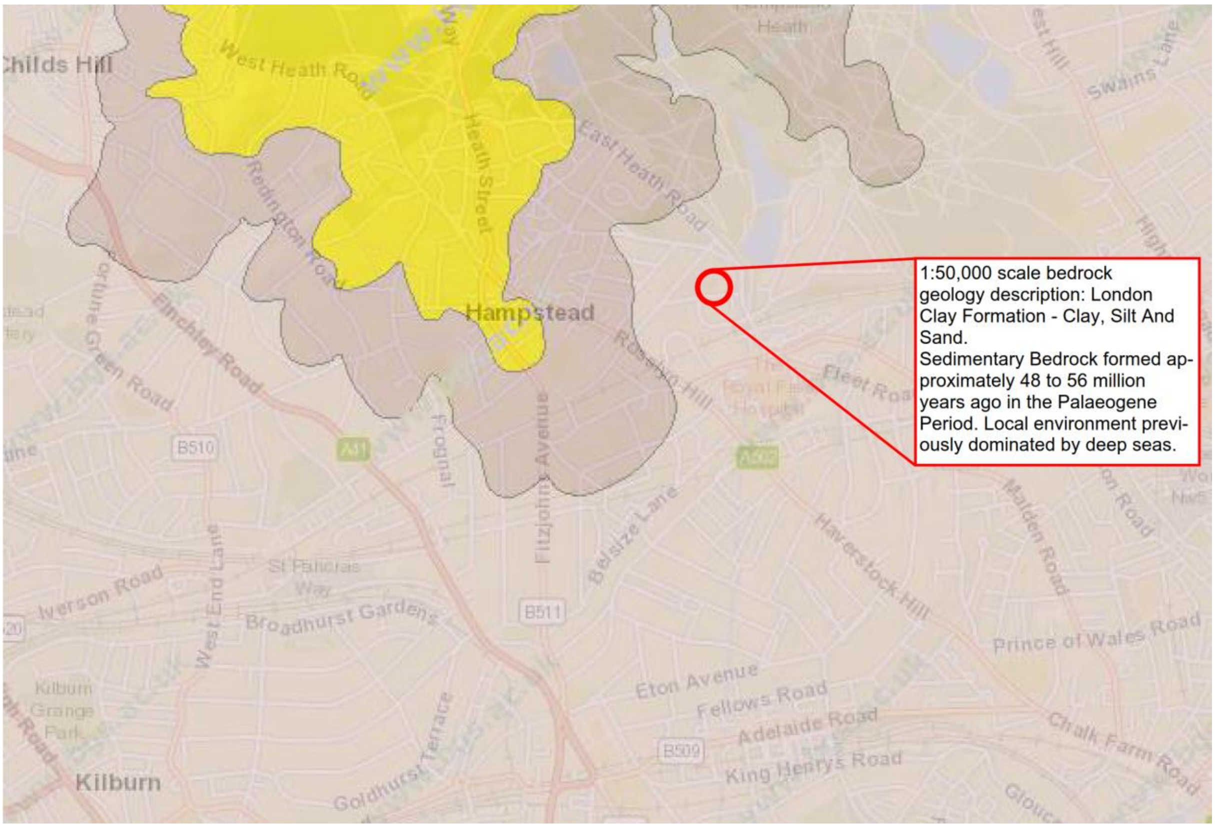

Figure 6: Site Geology: British Geological Survey – Geology of Britain Viewer accessed October 2019

Appendix B

Drainage and Hydrology Maps

Figure 7: Hampstead Chain Catchment Boundaries: Figure 4.3 of the Hampstead Heath Ponds Project (March 2013)

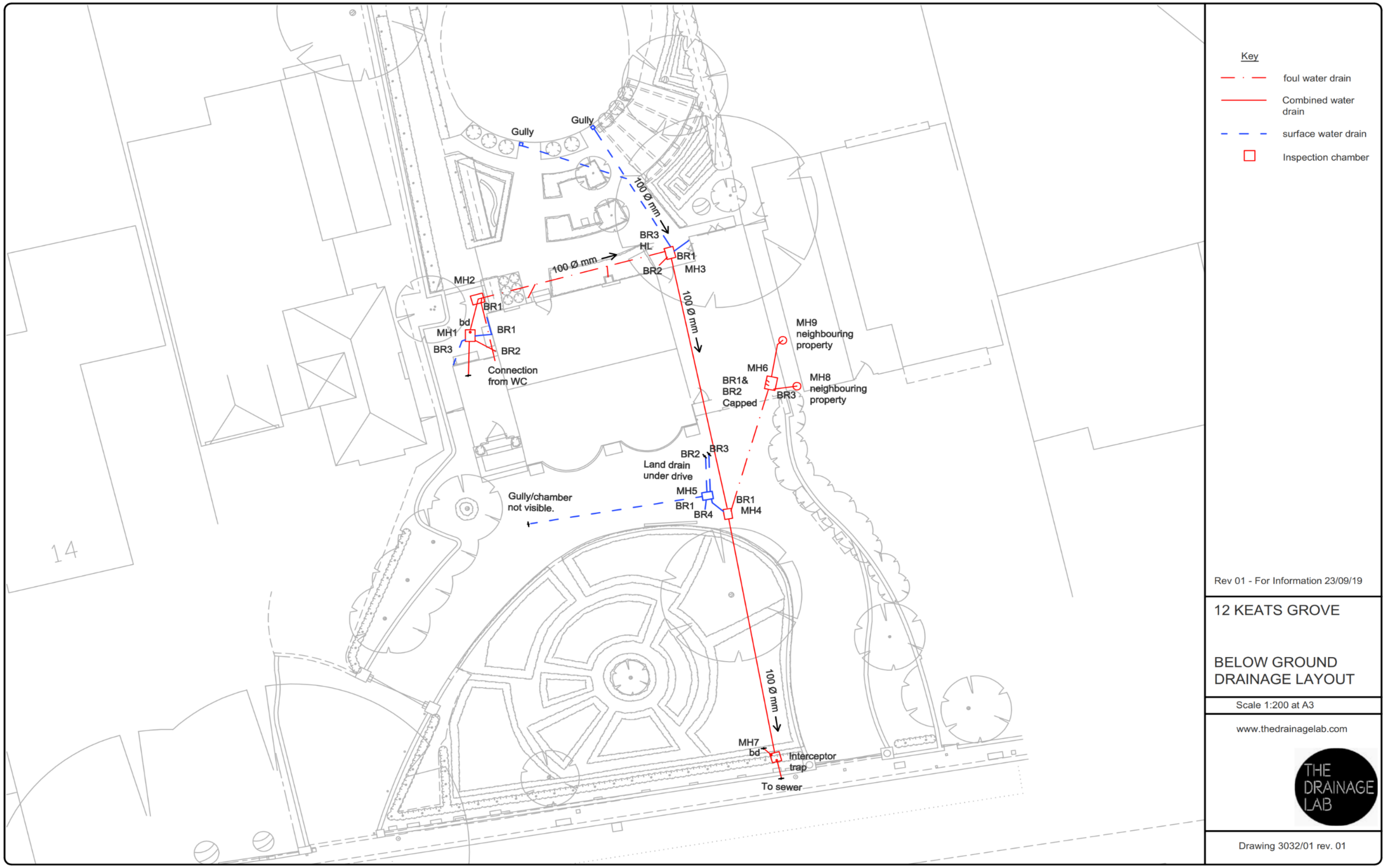

Figure 8: CCTV survey drawing

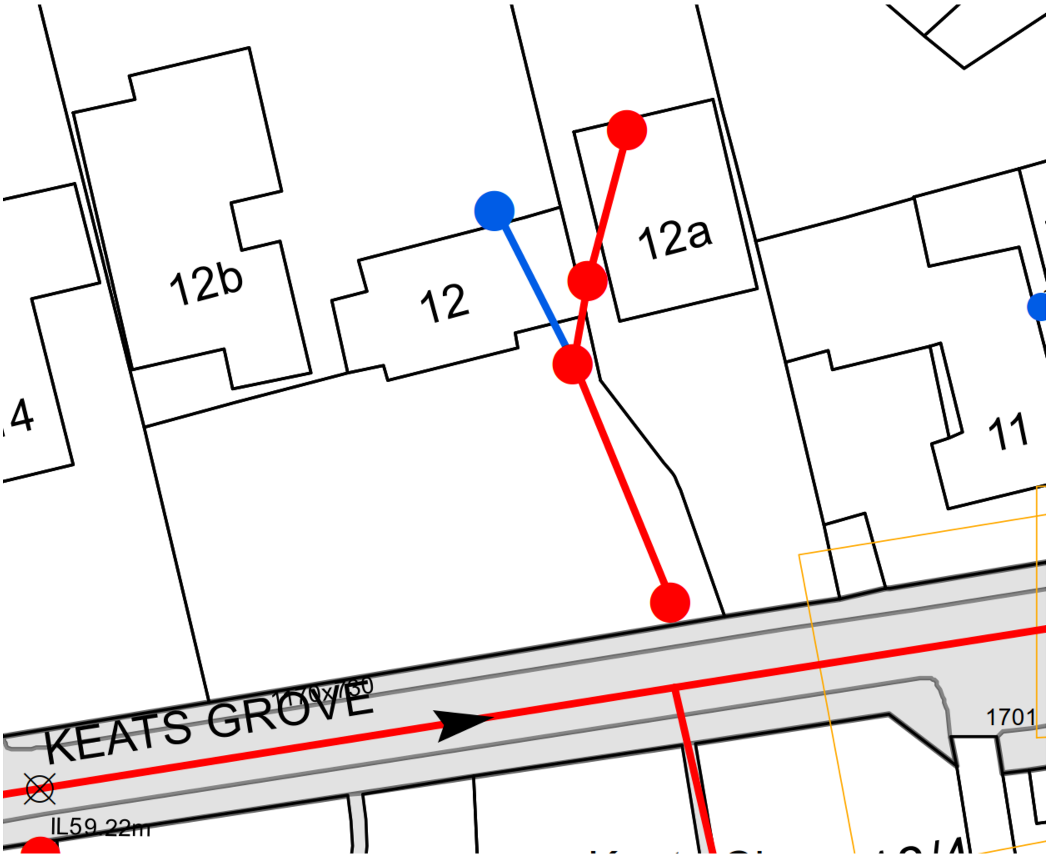

Figure 9: Extract from Thames Water Asset search

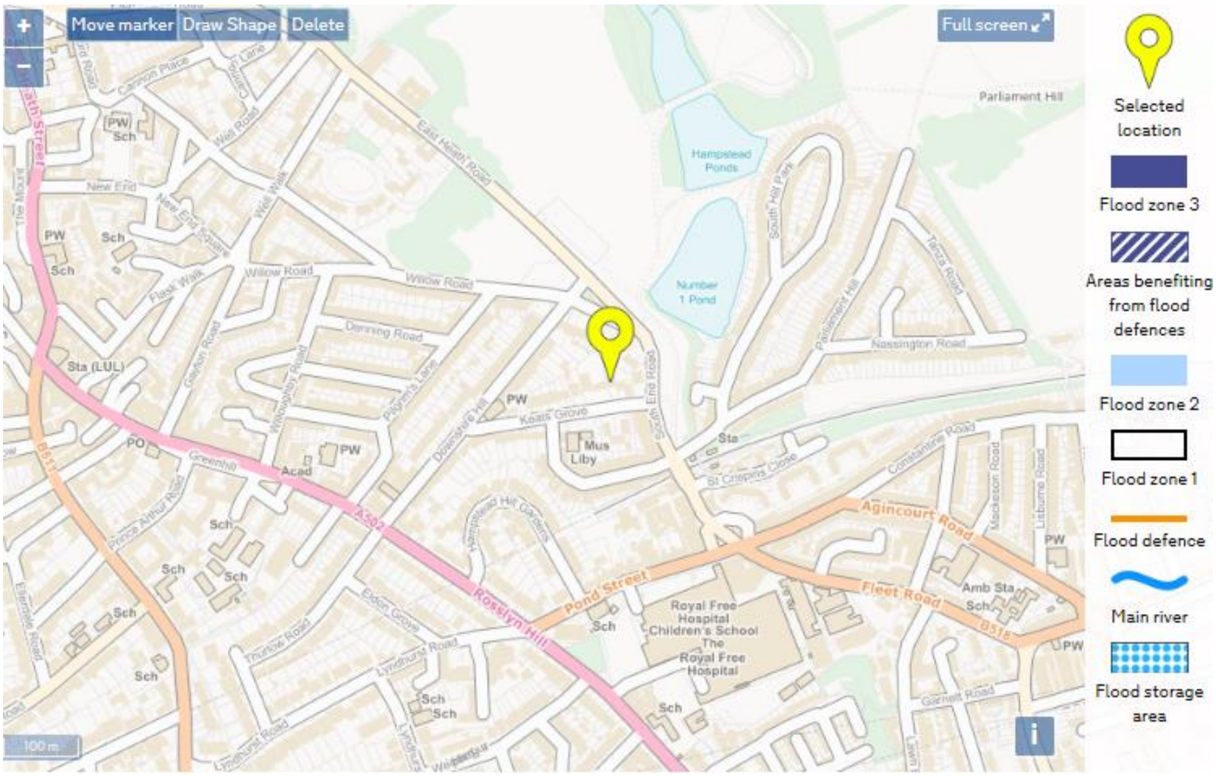

Figure 10: Extract from Environment Agency – Flood Map for Planning accessed October 2019

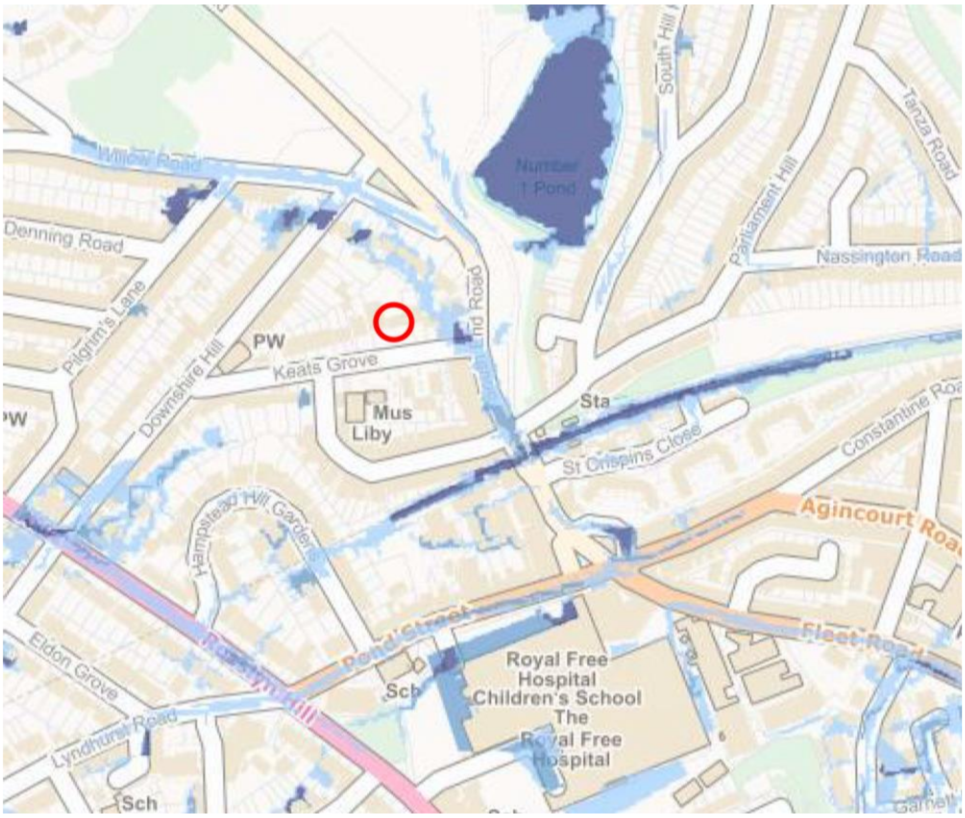

Figure 11: Environment Agency - Surface water flood risk map accessed October 2019

Appendix C

Proposed Structural Drawings and Construction Sequence

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Consulting Engineers

Sequence

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3

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PROPOSED BASEMENT CONSTRUCTION AND METHOD SEQUENCE

Sequence

1 Demolish structures, retaining and underpinning the garage wall adjacent to 12A.

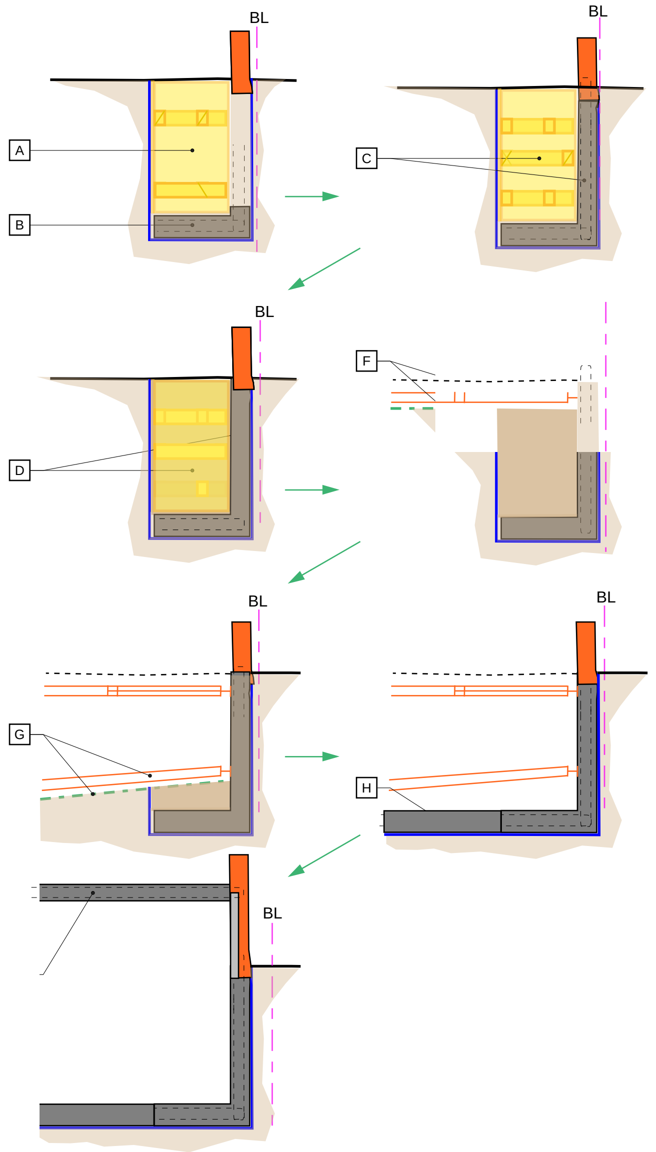
Note: Temporary support to be provided to retained wall

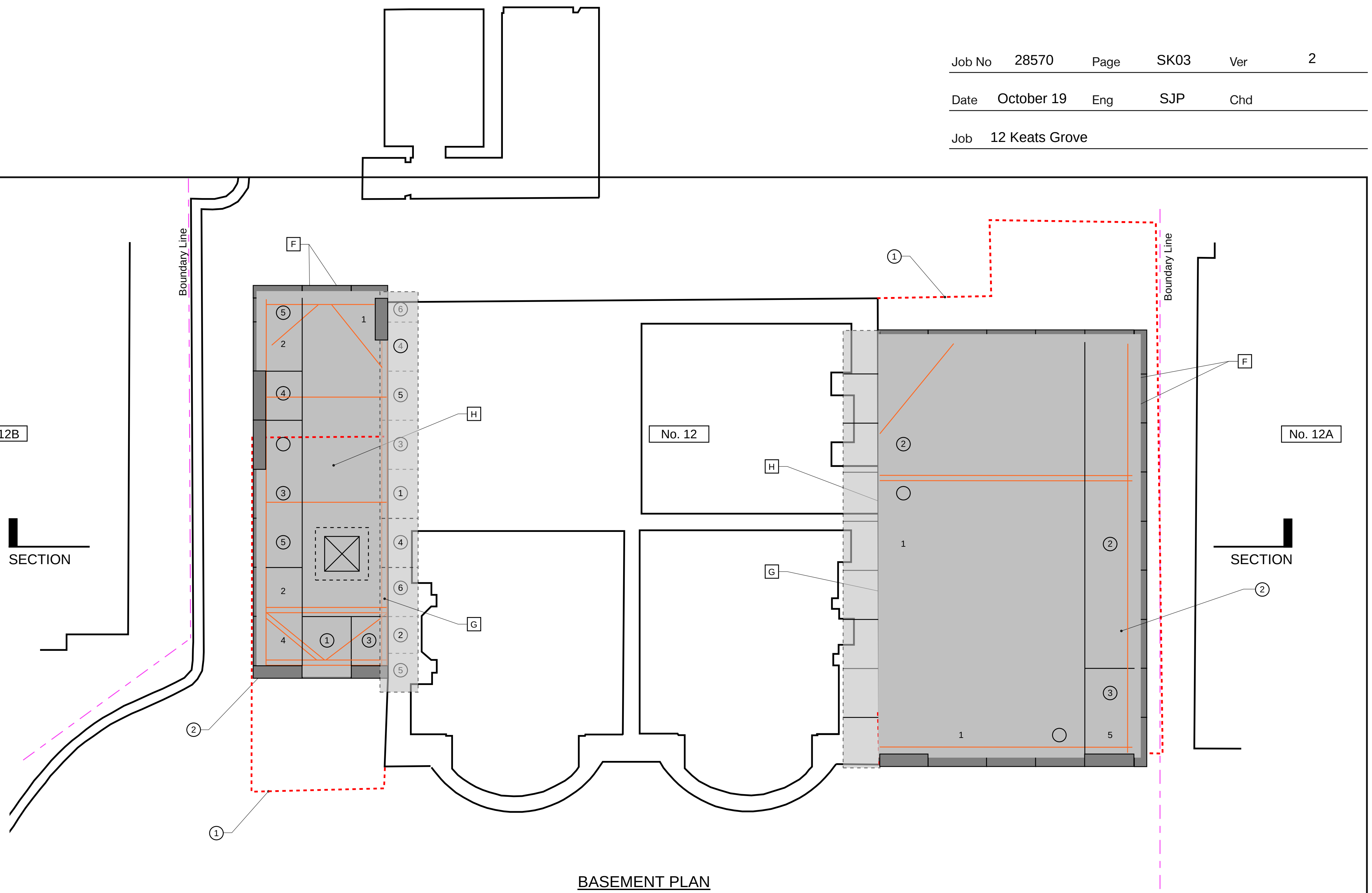
2 Construct basement in an underpinning sequence (1,4,2,5,3), see sequence below

3 Construct superstructure

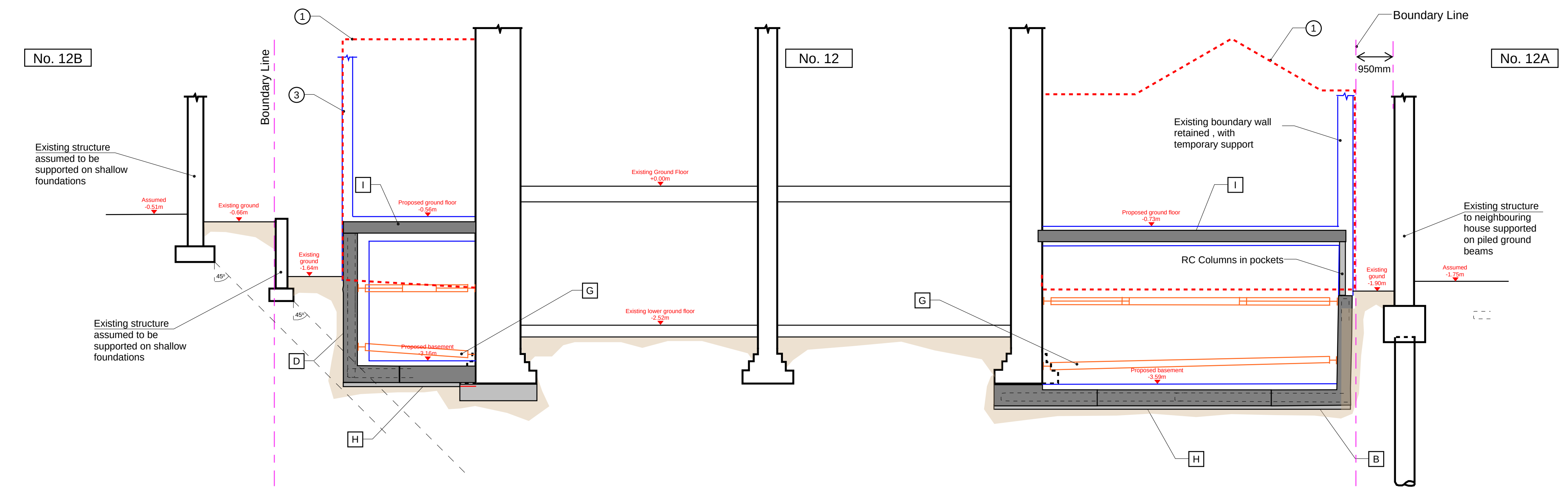
Underpinning Construction Sequence

A. Excavate for underpin, installing shoring as progressing
B. Cast RC base (install drainage if required)
C. Cast RC walls & underpins and additional strutting across
D. Backfill
E. Repeat until all underpins are complete
F. Excavate and install walers inc. diagonal bracing struts to mid-height
G. Batter down to existing foundation level and install lower waler and continue to excavate until lower level propping can be completed
H. Excavate remaining soil, install drainage, cast base slab.
I. Cast ground floor slab & stub columns and remove temporary works





BASEMENT PLAN



SECTION

Appendix D

Structural Calculations

RETAINING WALL ANALYSIS

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

Tedds calculation version 2.9.10

Analysis summary

Description	Unit	Capacity	Applied	F o S	Result
Bearing pressure	kN/m²	100	22.8	4.385	PASS

Design summary

Description	Unit	Provided	Required	Utilisation	Result
Stem p0 rear face - Flexural reinforcement	mm²/m	754.0	680.7	0.90	PASS
Stem p0 - Span/Depth ratio		11.5	16.6	1.44	FAIL
Base bottom face - Flexural reinforcement	mm²/m	1340.4	718.7	0.54	PASS
Base - Shear resistance	kN/m	135.1	46.5	0.34	PASS
Transverse stem reinforcement	mm²/m	392.7	250.0	0.64	PASS
Transverse base reinforcement	mm²/m	392.7	268.1	0.68	PASS

In reality the stem will be propped mid height during construction and the ground slab will provide a permanent prop at the top. This simplistic analysis also neglects the stiffening due to the return walls. Overall the deflection will be very low from experience. Considered OK

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **2800** mm
Stem thickness	t_{stem} = **250** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m³
Toe length	l_{toe} = **3000** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m³
Height of retained soil	h_{ret} = **1800** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **1000** mm
Water density	γ_w = **9.8** kN/m³

Retained soil properties

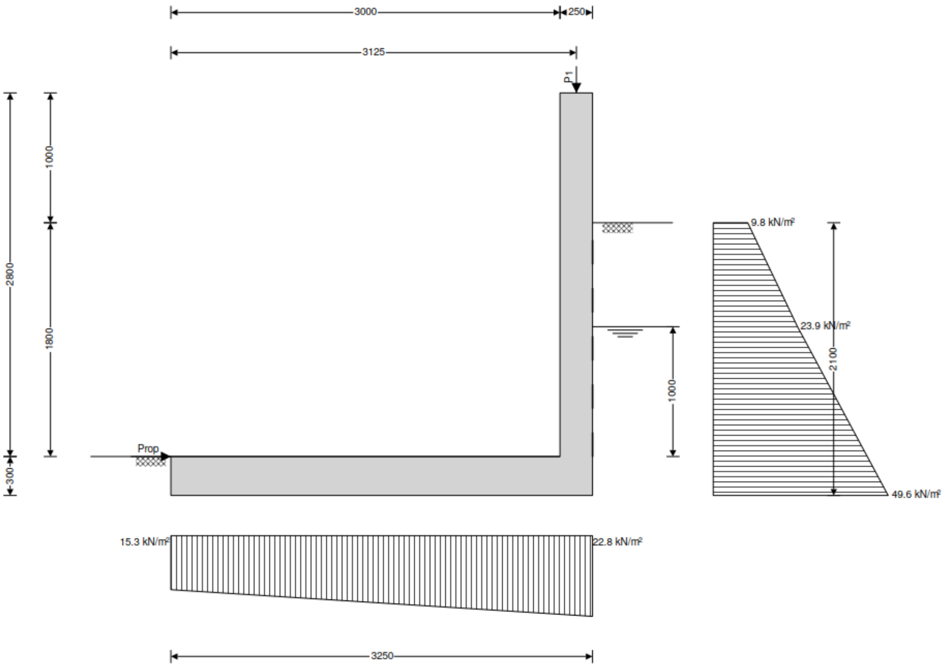
Soil type	Organic clay
Moist density	γ_{mr} = **18** kN/m³
Saturated density	γ_{sr} = **20** kN/m³
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **23** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **11.5** deg

Base soil properties

Soil type	Organic clay
Soil density	γ_b = **18** kN/m³
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **23** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **11.5** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **12** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m²

Loading details

Variable surcharge load	$Surcharge_Q$ = **10** kN/m²
Vertical line load at 3125 mm	P_{G1} = **15** kN/m
	P_{Q1} = **5** kN/m



General arrangement

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} = \mathbf{3250}$ mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover} = \mathbf{1000}$ mm
Moist soil height	$h_{moist} = h_{ret} - h_{water} = \mathbf{800}$ mm
Length of surcharge load	$l_{sur} = l_{heel} = \mathbf{0}$ mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2 = \mathbf{3250}$ mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret} = \mathbf{2100}$ mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2 = \mathbf{1050}$ mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem} = \mathbf{0.7}$ m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2 = \mathbf{3125}$ mm
Area of wall base	$A_{base} = l_{base} \times t_{base} = \mathbf{0.975}$ m^2
- Distance to vertical component	$x_{base} = l_{base} / 2 = \mathbf{1625}$ mm

For undrained soils - Annex C.1(2)

At rest pressure coefficient	$K_0 = \mathbf{1.000}$
Passive pressure coefficient	$K_P = \mathbf{1.000}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{17.5}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{24.4}$ kN/m
Line loads	$F_{P\_v} = P_{G1} + P_{Q1} = \mathbf{20}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{P\_v} + F_{water\_v} = \mathbf{61.9}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_0 \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = \mathbf{20.6}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_0 \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{8.4}$ kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{8.3}$ kN/m
Moist retained soil	$F_{moist\_h} = K_0 \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{24}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-0.8}$ kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{60.5}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{54.7}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{39.6}$ kNm/m
Surcharge load	$M_{sur} = -F_{sur\_h} \times x_{sur\_h} = \mathbf{-21.6}$ kNm/m
Line loads	$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{62.5}$ kNm/m
Saturated retained soil	$M_{sat} = -F_{sat\_h} \times x_{sat\_h} = \mathbf{-3.7}$ kNm/m
Water	$M_{water} = -F_{water\_h} \times x_{water\_h} = \mathbf{-3.6}$ kNm/m
Moist retained soil	$M_{moist} = -F_{moist\_h} \times x_{moist\_h} = \mathbf{-20.8}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{107.2}$ kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h} = \mathbf{60.5}$ kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{1732}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{107}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{3250}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{15.3}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{22.8}$ kN/m^2
Factor of safety	$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{4.385}$

Base resistance provided by friction with soil
Friction 0.33
G = 15kPa
A = 5m2/m
Friction resistance = 75kN/m
Additionally existing foundation and passive resistance will help significantly

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

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

Tedds calculation version 2.9.10

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

Concrete strength class	C30/37
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{30}$ N/mm^2
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{37}$ N/mm^2
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8$ N/mm$^2 = \mathbf{38}$ N/mm^2
Mean value of axial tensile strength	$f_{ctm} = 0.3$ N/mm$^2 \times (f_{ck} / 1$ N/mm$^2)^{2/3} = \mathbf{2.9}$ N/mm^2
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.0}$ N/mm^2
Secant modulus of elasticity of concrete	$E_{cm} = 22$ kN/mm$^2 \times (f_{cm} / 10$ N/mm$^2)^{0.3} = \mathbf{32837}$ N/mm^2
Partial factor for concrete - Table 2.1N	$\gamma_C = \mathbf{1.50}$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = \mathbf{0.85}$
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{17.0}$ N/mm^2
Maximum aggregate size	$h_{agg} = \mathbf{20}$ mm
Ultimate strain - Table 3.1	$\varepsilon_{cu2} = \mathbf{0.0035}$
Shortening strain - Table 3.1	$\varepsilon_{cu3} = \mathbf{0.0035}$

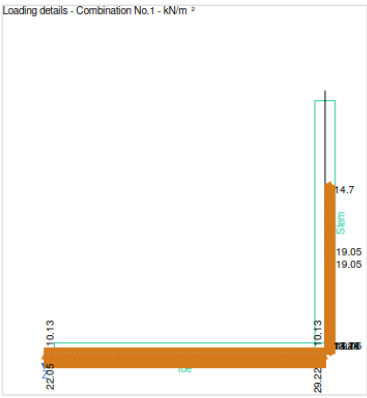
Effective compression zone height factor	λ = **0.80**
Effective strength factor	η = **1.00**
Bending coefficient k_1	K_1 = **0.40**
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ = **1.00**
Bending coefficient k_3	K_3 =**0.40**
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2})$ =**1.00**

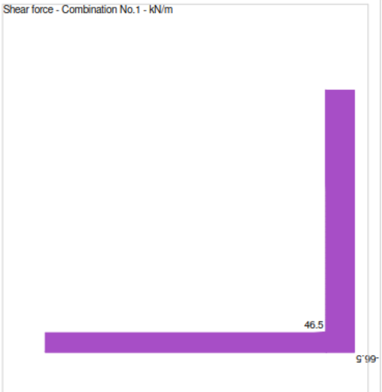
Reinforcement details

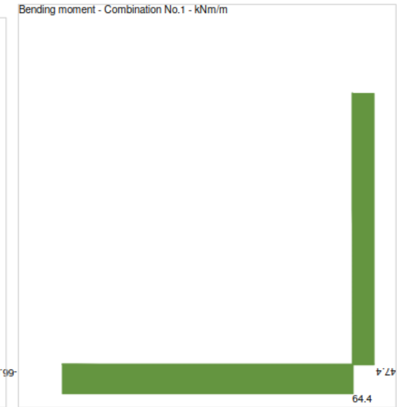
Characteristic yield strength of reinforcement	f_{yk} = **500** N/mm^2
Modulus of elasticity of reinforcement	E_s = **200000** N/mm^2
Partial factor for reinforcing steel - Table 2.1N	γ_S = **1.15**
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S$ = **435** N/mm^2

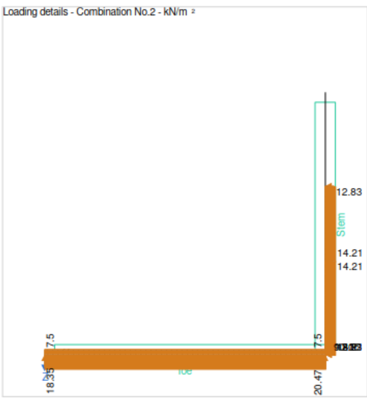
Cover to reinforcement

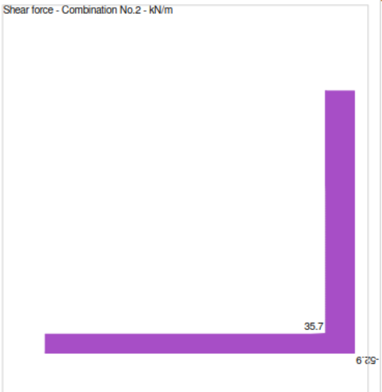
Front face of stem	c_{sf} = **30** mm
Rear face of stem	c_{sr} = **75** mm
Top face of base	c_{bt} = **50** mm
Bottom face of base	c_{bb} = **75** mm

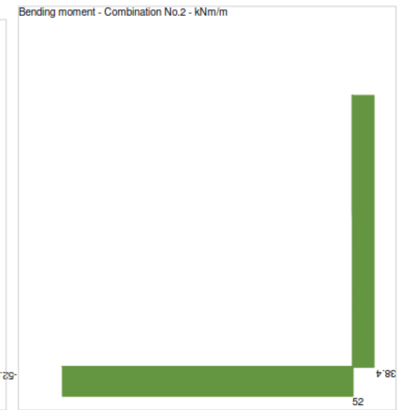












Check stem design at base of stem

Depth of section	h = **250** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **47.4** kNm/m
Depth to tension reinforcement	$d = h - c_{sr} - \phi_{sr} / 2$ = **169** mm
	$K = M / (d^2 \times f_{ck})$ = **0.055**
	$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = **0.207**
	K' > K - No compression reinforcement is required
Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **160** mm
Depth of neutral axis	$x = 2.5 \times (d – z)$ = **22** mm
Area of tension reinforcement required	$A_{sr.req} = M / (f_{yd} \times z)$ = **681** mm²/m
Tension reinforcement provided	12 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr})$ = **754** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **255** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{sr.max} = 0.04 \times h$ = **10000** mm²/m
	$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov}$ = **0.903**

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

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio	$\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000$ = **0.005**
Required tension reinforcement ratio	$\rho = A_{sr.req} / d$ = **0.004**
Required compression reinforcement ratio	$\rho' = A_{sr.2.req} / d_2$ = **0.000**
Structural system factor - Table 7.4N	K_b = **0.4**
Reinforcement factor - exp.7.17	$K_s = \min(500\ N/mm^2 / (f_{yk} \times A_{sr.req} / A_{sr.prov}), 1.5)$ = **1.108**
Limiting span to depth ratio - exp.7.16.a	$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b)$ = **11.5**
Actual span to depth ratio	h_{stem} / d = **16.6**

FAIL - Span to depth ratio exceeds deflection control limit

In reality the stem will be propped mid height during construction and the ground slab will provide a permanent prop at the top. This simplistic analysis also neglects the stiffening due to the return walls. Overall the deflection will be very low from experience. Considered OK

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.3**
Serviceability bending moment	M_{sls} = **22.3** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{sr.prov} \times z)$ = **184.2** N/mm²
Load duration	Long term
Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **76079** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff} = A_{sr.prov} / A_{c.eff}$ = **0.010**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**

Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff}$ = **461** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.255** mm
	w_k / w_{max} = **0.849**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **66.5** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2)$ = **2.000**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{sr.prov} / d, 0.02)$ = **0.004**
	$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5}$ = **0.542** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **96.3** kN/m
	$V / V_{Rd.c}$ = **0.690**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req} = \max(0.25 \times A_{sr.prov}, 0.001 \times t_{stem})$ = **250** mm²/m
Maximum spacing of reinforcement – cl.9.6.3(2)	s_{sx_max} = **400** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{sx.prov} = \pi \times \phi_{sx}^2 / (4 \times s_{sx})$ = **393** mm²/m

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

Check base design at toe

Depth of section	h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **64.4** kNm/m
Depth to tension reinforcement	$d = h - c_{bb} - \phi_{bb} / 2$ = **217** mm
	$K = M / (d^2 \times f_{ck})$ = **0.046**
	$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d$ = **206** mm
Depth of neutral axis	$x = 2.5 \times (d - z)$ = **27** mm
Area of tension reinforcement required	$A_{bb.req} = M / (f_{yd} \times z)$ = **719** mm²/m
Tension reinforcement provided	16 dia.bars @ 150 c/c
Area of tension reinforcement provided	$A_{bb.prov} = \pi \times \phi_{bb}^2 / (4 \times s_{bb})$ = **1340** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d$ = **327** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max} = 0.04 \times h$ = **12000** mm²/m
	$\max(A_{bb.req}, A_{bb.min}) / A_{bb.prov}$ = **0.536**

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

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width	w_{max} = **0.3** mm
Variable load factor - EN1990 – Table A1.1	ψ_2 = **0.3**
Serviceability bending moment	M_{sls} = **45.4** kNm/m
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{bb.prov} \times z)$ = **164.3** N/mm²
Load duration	Long term

Load duration factor	k_t = **0.4**
Effective area of concrete in tension	$A_{c.eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$
	$A_{c.eff}$ = **90958** mm^2/m
Mean value of concrete tensile strength	$f_{ct.eff} = f_{ctm}$ = **2.9** N/mm^2
Reinforcement ratio	$\rho_{p.eff} = A_{bb.prov} / A_{c.eff}$ = **0.015**
Modular ratio	$\alpha_e = E_s / E_{cm}$ = **6.091**
Bond property coefficient	k_1 = **0.8**
Strain distribution coefficient	k_2 = **0.5**
	k_3 = **3.4**
	k_4 = **0.425**
Maximum crack spacing - exp.7.11	$s_{r.max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p.eff}$ = **440** mm
Maximum crack width - exp.7.8	$w_k = s_{r.max} \times \max(\sigma_s - k_t \times (f_{ct.eff} / \rho_{p.eff}) \times (1 + \alpha_e \times \rho_{p.eff}), 0.6 \times \sigma_s) / E_s$
	w_k = **0.217** mm
	w_k / w_{max} = **0.722**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

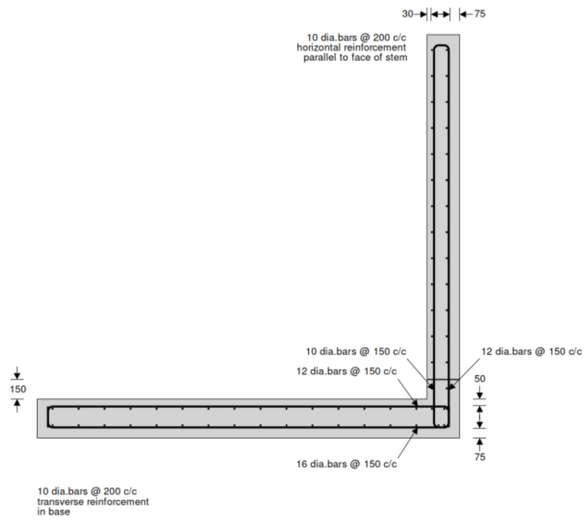
Design shear force	V = **46.5** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	$k = \min(1 + \sqrt{(200\ mm / d)}, 2)$ = **1.960**
Longitudinal reinforcement ratio	$\rho_l = \min(A_{bb.prov} / d, 0.02)$ = **0.006**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.526** N/mm^2
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **135.1** kN/m
	$V / V_{Rd.c}$ = **0.344**

PASS - Design shear resistance exceeds design shear force

Secondary transverse reinforcement to base - Section 9.3

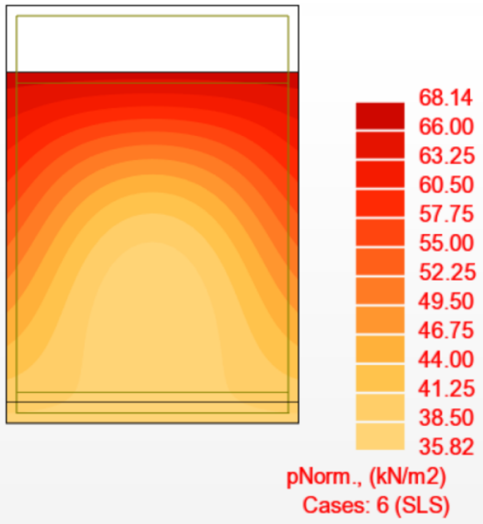
Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **268** mm^2/m
Maximum spacing of reinforcement – cl.9.3.1.1(3)	s_{bx_max} = **450** mm
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx.prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx})$ = **393** mm^2/m

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

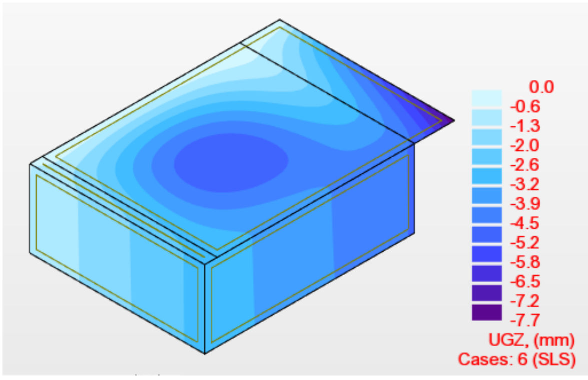


Reinforcement details

Basement 3D analysis



Bearing Pressure < 100kPa



Deformation < 10mm

Base Slab

300 thick slab. Strength OK as max stresses due to retained loads.
Approximate long term heave potential:
= 0.5 x depth of excavation x density of soil = 0.5*2.2x18 = 19.8kN/m^2

Weight of basement RC slabs + screeds = (.3+.2+.15)*25 = 16.5kN/m^2

Weight of basement walls = 20*25*2.5*0.25/5/10 = 6.25kN/m^2

Weight of superstructure = 1.5kN/m^2

Total new structure = 24.25kN/m^2 > 19.8kN/m^2

∴OK

Ground Slab

200 thick two-way spanning slab, spanning 5.5m. From Concrete Centre Economic Concrete Elements guide, extract below, the overall depth required ~140mm < 200mm

∴OK

Table 3.6a
Data for two-way solid slabs: single span

SINGLE span, m	4.0	5.0	6.0
Overall depth, mm			
IL = 2.5 kN/m^2	125	129	153
IL = 5.0 kN/m^2	125	144	170
IL = 7.5 kN/m^2	128	156	183
IL = 10.0 kN/m^2	138	168	197