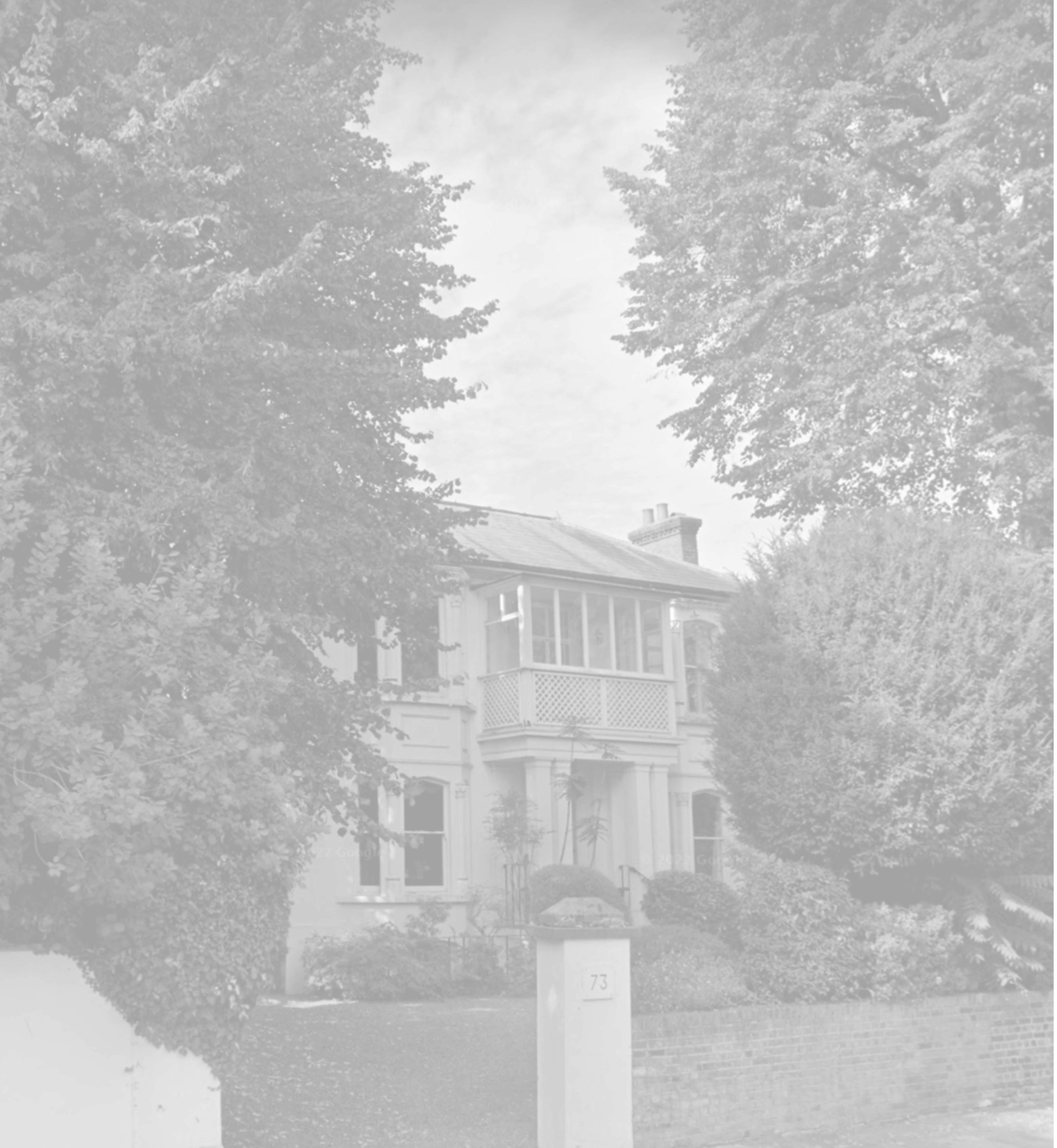

Basement Impact Assessment

Proposed Pool House Construction

73 Castelnau

Jan 2023

Job no. JH1751

JENSENHUNT | DESIGN

Contents

Status: Planning

Date: Jan 2023

Revision: P2

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Prepared by: Alex Hunt C.Eng M.IStruct.E

Checked by: Matt Turner C.Eng M.IStruct.E

LBRUT Policy Requirements for Basements- Adopted Supplementary Planning Document

	RBRUT Planning Requirements	Response and/or reference location
A	Basement Screening Assessment A brief exploration into the impacts of the proposed basement works regarding subterranean characteristics, land and slope stability and flood risks and drainage.	See separate document
B	Basement Impact Assessment Extracting key areas of concern identified in Basement Screening Assessment and exploring them further. Particularly on their impacts to neighbouring properties and flooding risks.	Included in this document.

1. Introduction

Jensen Hunt Design have been appointed by the owner of 73 Castelnau as the Chartered Structural Engineers and have been instructed to prepare a Basement Impact Assessment (BIA) for the proposed construction of a new pool house. The purpose of this BIA is to support the planning application to the London Borough of Richmond Upon Thames (RBRUT).

The BIA has been developed in line with the requirements of the RBRUT outlined in the Basement Assessment User Guide and Planning Advice Note - *Good Practice Guide on Basement Developments* (May 2015). A Basement Screening Assessment has preceded this BIA and highlighted key areas of concern that require further exploration in this BIA. These areas are:

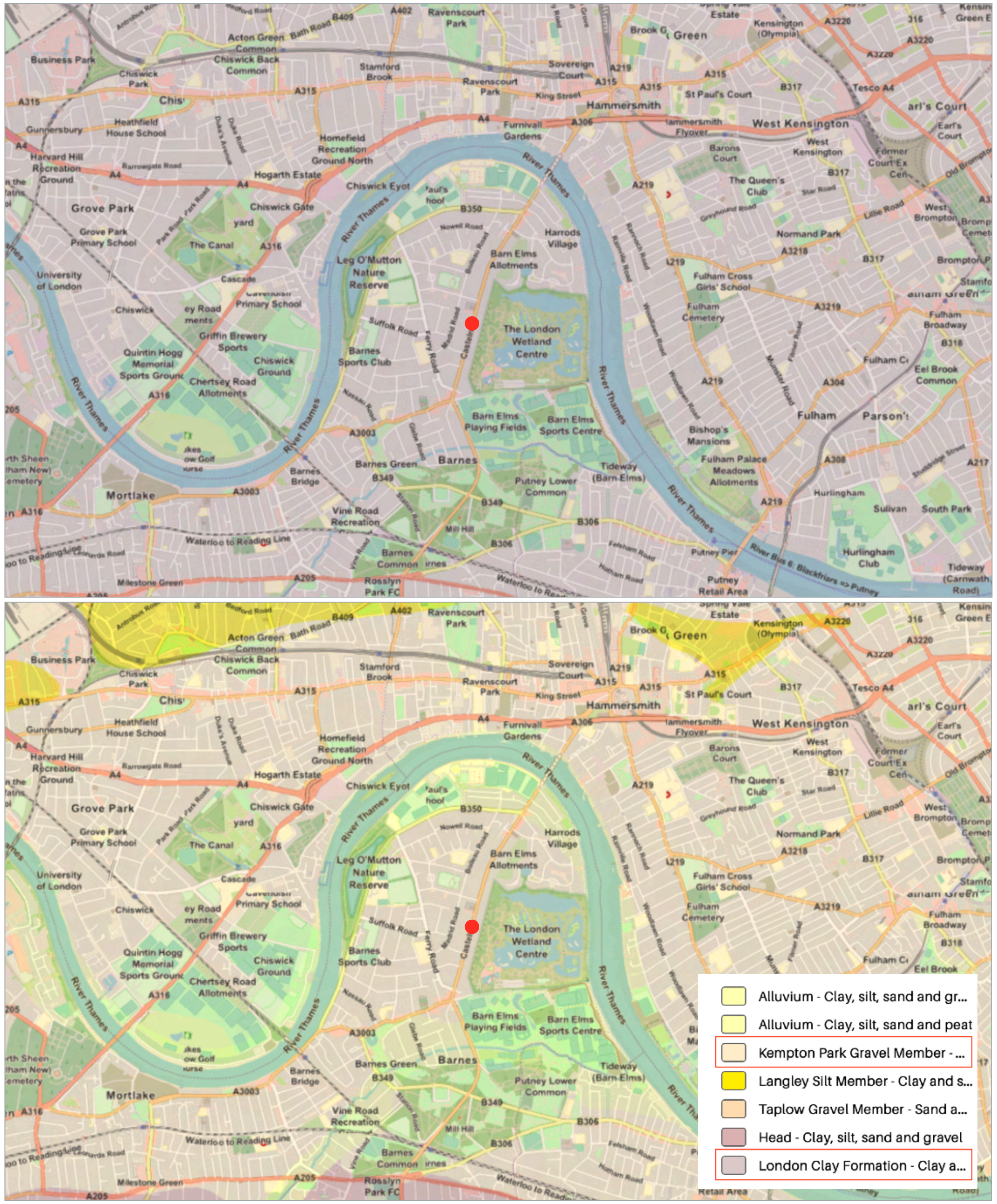
- Fluvial/tidal flooding
- Surface water flooding
- Groundwater flooding
- Reservoir flooding
- Proposed drainage strategy
- Impacts on neighbouring properties
- Trees

Several areas were briefly explored in Basement Screening Assessment and found to be of no concern for the proposed works and will not be taken further in this BIA. These areas are:

- Existing watercourses & spring lines
- Existing & proposed topography
- Influence of underground infrastructure

Street View - 73 Castelnau. Source: Google Maps

Aerial Plan View - 73 Castelnau. Source: Google Maps



Colour map showing bedrock geology (top) and superficial deposits (bottom) within the area of 73 Castelnau.
Source: BGS Online

2. The Project

The proposed scheme is comprised of a new-build pool house located in the rear garden of the existing property at 73 Castelnau. The pool house is approximately 8.0 x 4.6m in plan and 3.0m in height and contains a pool of 1.5m depth below ground. The depth of excavation is not expected to exceed 2.0m. The proposed scheme also includes works to the main house. However, these works do not include basement works and are not discussed further in this report.

The purpose of this basement impact assessment is to explore the impacts of the proposed substructure works, particularly on the neighbouring properties and the flooding risk. This report is intended to contribute to a planning application made by Locksley Architects.

2.1 The Existing Site

The site is situated in the north-east region of the Richmond borough and the west-side of Castelnau, the main road running south of Hammersmith Bridge. It is located within the Castelnau Conservation Area. Key features within the area include the London Wetland Centre to the east of the site and Barns Elms and Barnes Common to the south of the site. No. 73 is characterised as a Building of Townscape Merit. The site is typically flat and lies at approx. 9m AOD. As discussed in the Basement Screening Assessment, the topography of the site is generally flat and therefore slope instability is not deemed to be of concern during the construction of the new pool house.

2.2 Proposed Works

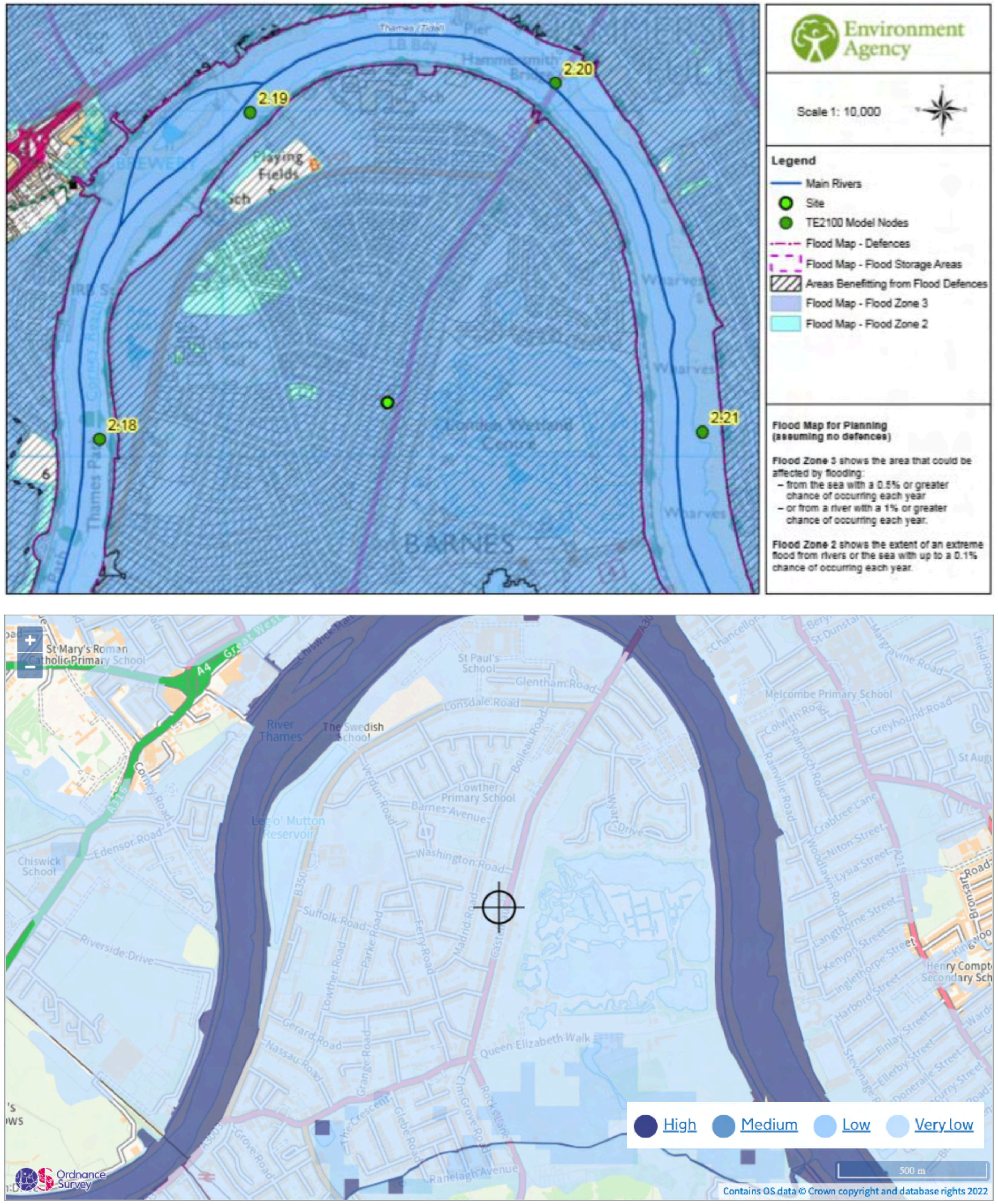
The proposal is for a new pool house in the rear garden of the main property at 73 Castelnau. Within the pool house, the scheme proposes a pool of approx. 5.8 x 3.0m plan area. Architectural drawings of the proposed poolhouse can be seen in Appendix A.

A reinforced concrete (RC) substructure will be formed to create the new pool. A preliminary design for the pool retaining walls can be seen in Appendix B.

In accordance with BS 8102:2009, the RC box will be designed to provide two forms of protection against water from the ground which could include barrier protection (i.e. membranes / layers / renders), structurally integral protection (i.e. waterproof concrete or designed concrete - crack control) or drained cavity protection. The design of any proprietary protection systems will be the responsibility of a specialist Contractor.

3. Impacts & Mitigations

3.1 Geology and Ground Conditions



Tidal and River flood maps for proposed site from Environmental Agency showing Flood Zone 3 prior to flood defences (top) and very low risk after benefitting from existing flood defences (bottom). Source: EA & gov.uk

The British Geological Survey (BGS) maps of the area identifies the superficial ground deposits as Kempton Park Gravel Member - Sand And Gravel, overlaying London Clay Formation - Clay And Silt.

Borehole investigations conducted as part of the planning application for No. 130 Castelnau (22/0901/FUL), approximately 0.4 km from the proposed site, closely corroborated the conclusions drawn from the BGS maps. Made ground was overlaid on Kempton Park Gravel which, in turn, was underlain by London Clay formation. A summary of the findings of the trial pits can be seen below.

Stratum	Top Depth (m, BGL)	Base Depth (m, BGL)	Thickness (m)
MADE GROUND	0.00	0.70	0.70
KEMPTON PARK GRAVEL MEMBER (CLAY)	0.70	1.20 - 1.70	0.50 - 1.00
KEMPTON PARK GRAVEL MEMBER (SAND)	1.20 - 1.70	> 5.00 - 6.70	> 3.30 - 5.50
KEMPTON PARK GRAVEL MEMBER (GRAVEL)	6.70	7.80	1.10
LONDON CLAY FORMATION	7.80	> 8.45	> 0.65

Summary of findings from trial pit investigations conducted for No. 130 Castelnau planning application.

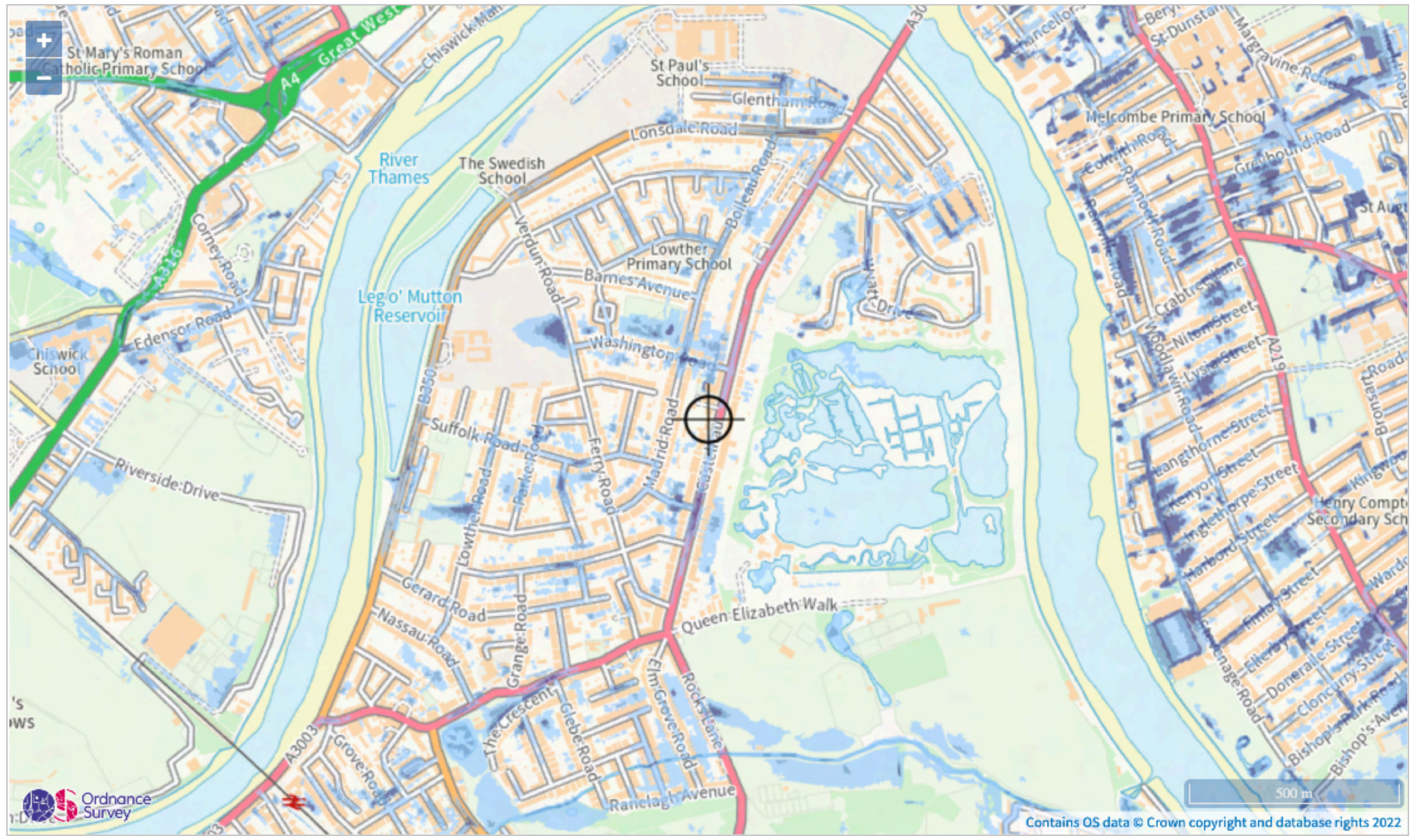
The findings from these ground investigations suggest that the pool house substructure may potentially lie upon the Kempton Park Gravel Member (Clay). However, due to the distance between Nos. 73 and 130 Castelnau and the resulting unreliability of these results, geotechnical site investigations have been proposed by Jensen Hunt Design to be undertaken by a geotechnical specialist and will include a desktop study.

With the potential presence of water, either perched or groundwater, the soil stability of the clay layer may be significantly compromised. Efforts should be made to prevent any groundwater ingress into the excavation during construction and kept dry for the duration of construction using a sump or other means where applicable. Where the structural design permits, efforts will be made to ensure the pool retaining walls surpass the cohesive Kempton Park Gravel Member layers and settle on the granular stratum instead.

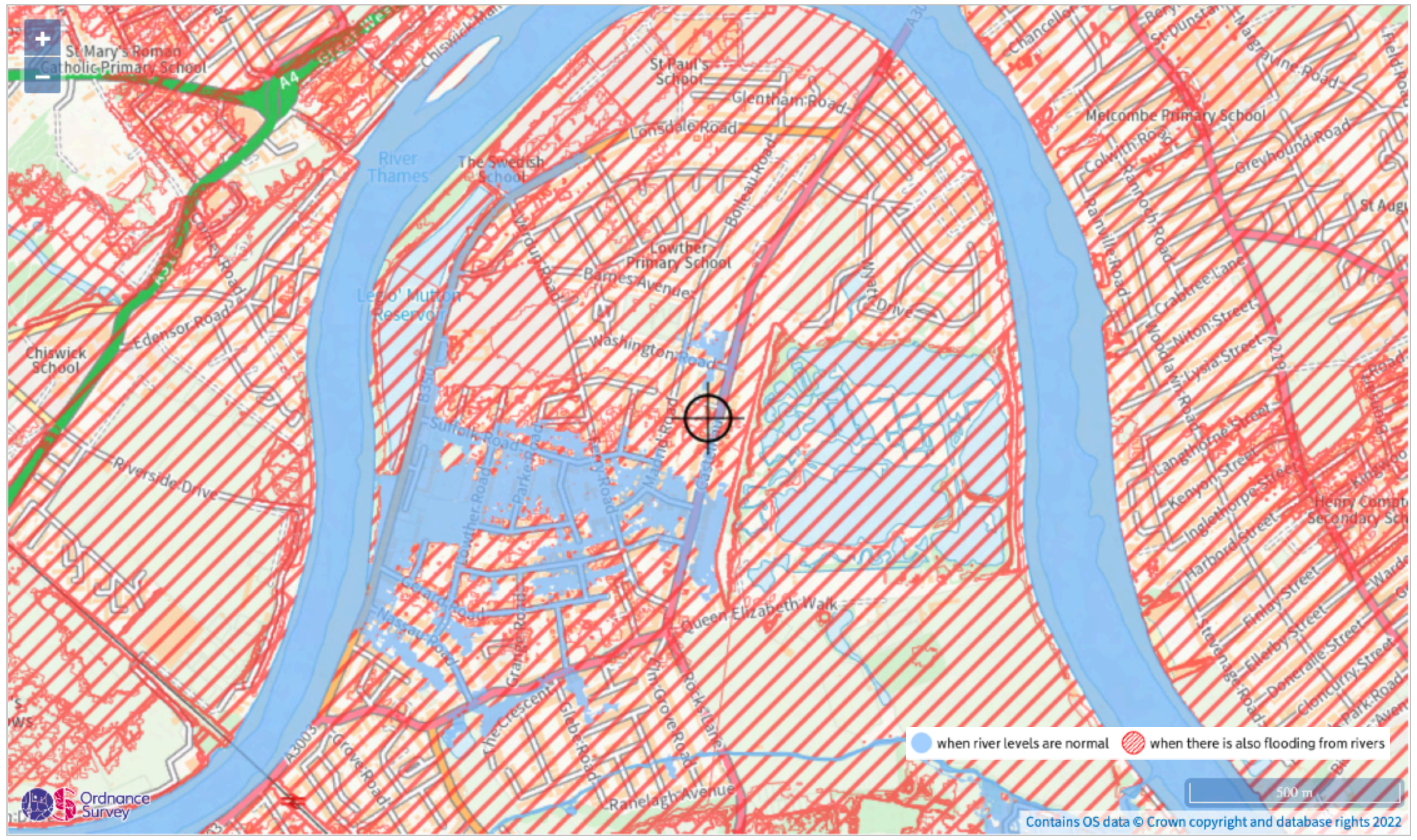
From the two trial pit investigations conducted as part of the No. 130 Castelnau planning application, groundwater was only detected in one - at a depth of approximately 3.80m below ground. With the proximity of the trial pits to each other, we would expect the groundwater table to be detected in both trial pits. This suggests that the geotechnical findings for these investigations are undependable. Ground investigations and monitoring at 73 Castelnau will need to be conducted to ensure the local water table behaviour and the soil conditions are captured and accounted for in the design of the pool house substructure.

3.2 Site Hydrology

As noted above the site is located in Kempton Park Gravel and London Clay. The Kempton Park Gravels are considered a Secondary ‘A’ Aquifer, which refers to permeable layers



Flood map for surface water flooding. Source: gov.uk



Flood map for reservoir flooding. Source: gov.uk

capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers.

The London Clay is classified as Unproductive Strata, which refers to rock layers or drift deposits with low permeability that have negligible significance for water supply or river base flow.

This suggests that the presence of groundwater should be expected across the site.

3.3 Groundwater Flooding

The London Borough of Richmond Upon Thames: Strategic Flood Risk Assessment Online Maps find that the site is within an area characterised with at least 75% susceptibility to groundwater flooding but does not fall within the four throughflow and groundwater catchment areas located within the borough. Consequently, the proposed pool house substructure is vulnerable to groundwater flooding. Also, the presence of perched water may be encountered during the construction of the pool house, especially after periods of heavy rainfall.

To mitigate this risk, flood resistant measures should be implemented such as tanking of the pool house substructure. The pool retaining walls will also be designed conservatively to account for the presence of ground water table at a low depth below ground level as well as the potential presence of perched water.

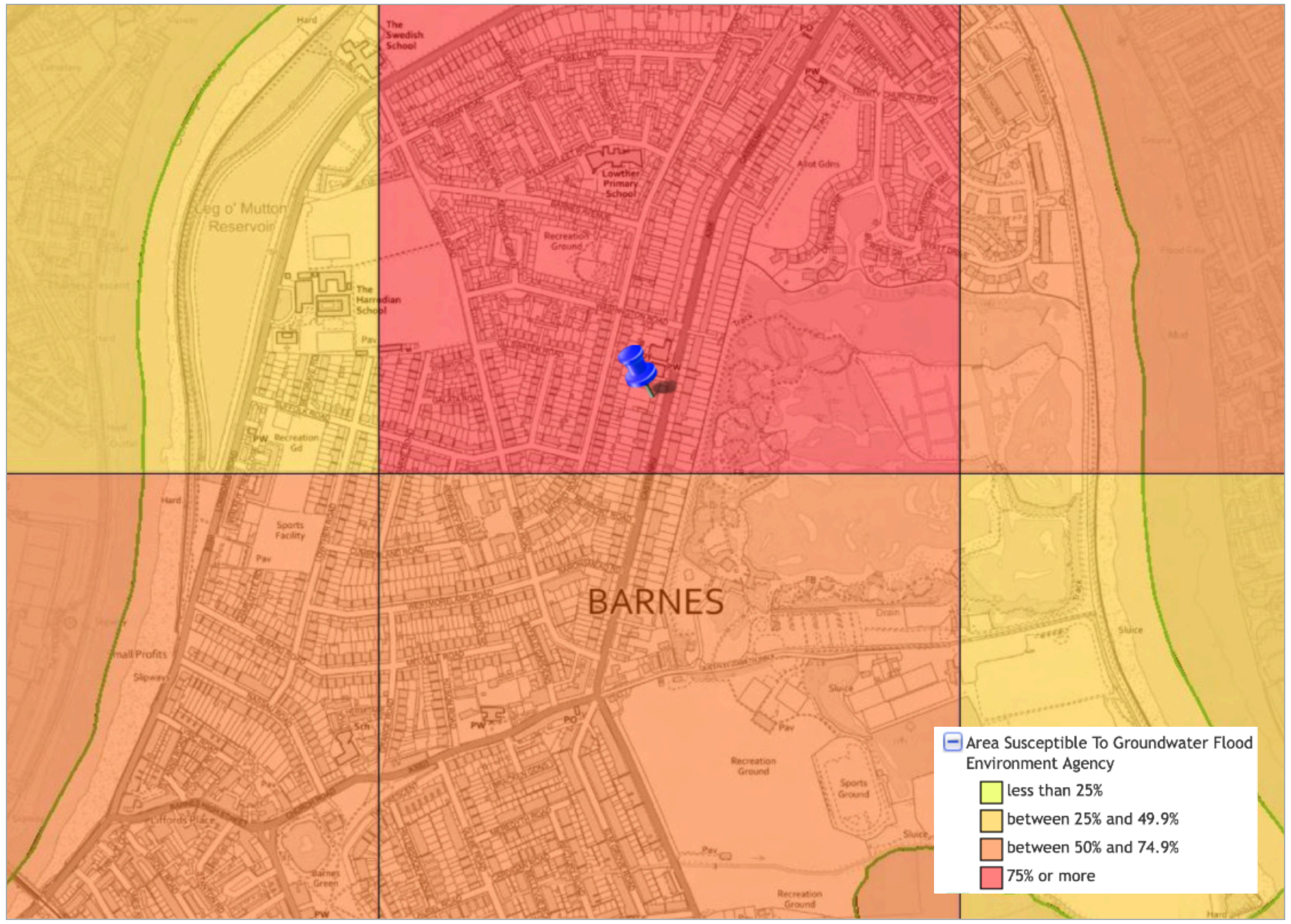
The Alan Baxter Residential Basement Study Report (2012) states that excavations conducted in sand or gravels wholly above the upper aquifer should not have an impact on the ground water unless the construction cuts off the ground water by extending downwards below or close to the aquifer. With excavations proposed at depths of approx. 2.0m, and historic boreholes detecting groundwater at depths of approx. 4.0m below ground level, it is unlikely that this applies.

It is anticipated that the groundwater will be able to follow a pathway beneath and around the new pool (Scenario B in below figure). This would also be applicable in the future if a basement structure was constructed beneath the adjacent property (Scenario D).

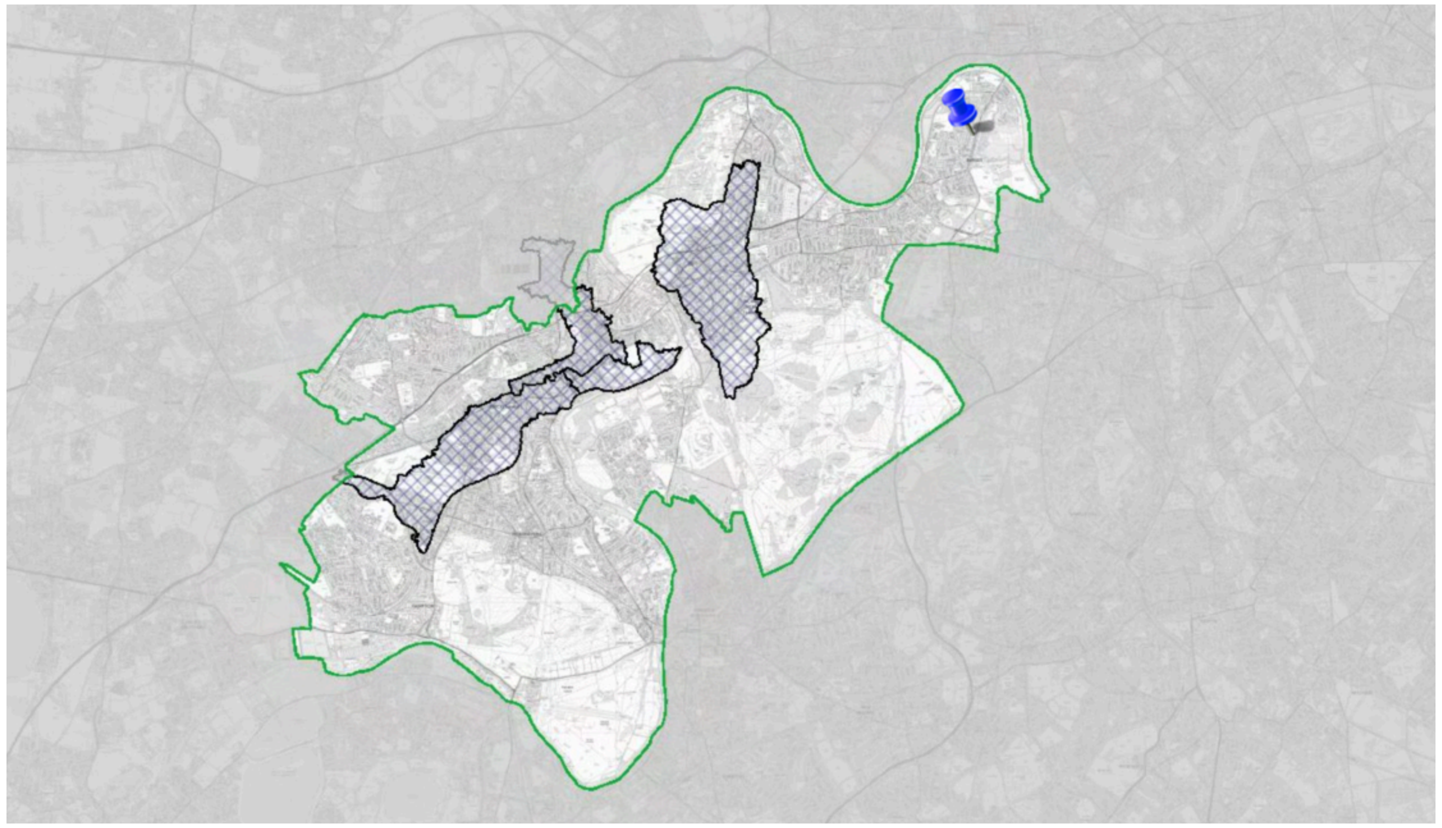
Any potential effects of damming or restriction of ground water flow are considered minimal. Therefore, the proposed pool construction is unlikely to significantly impact the hydrology of the area.

Since historic boreholes identified the groundwater depth to fall below the anticipated excavation depth, it is unlikely that groundwater will be encountered during the excavation of the pool house substructure. However, if this is not the case, there may be a requirement to use sump pumps during construction to draw out groundwater ingress but we do not expect this to affect the local hydrology.

3.4 Tidal and River (Fluvial) Flooding



Strategic Flood Risk Assessment (SFRA) map showing very high risk of groundwater flooding at the site location. Source: London Borough of Richmond Upon Thames.


Strategic Flood Risk Assessment (SFRA) map showing through flow and groundwater catchment areas with in the borough relative to the site location. Source: London Borough of Richmond Upon Thames.

The site is northbound by the Thames river at an approximate 1.0km radius from the site. The London lost rivers maps indication that the Beverley Brook river runs approximately 0.8km south of the site. This river is now culverted and not expected to be at risk of flooding. No other watercourses exist in proximity to the site. The Environmental Agency (EA) flood maps for rivers and sea show that the site is located in a Flood Zone 3. This means that, in any given year, there is a 1% chance of fluvial flooding (rivers) and 0.5% chance of tidal flooding (seas). However, the site falls within the area benefitting from flood defences such as a Thames barrier. These flood defences reduce the risk of tidal and river flooding to a very low degree with a chance of flooding of less than 0.1% in a given year. Additionally, no records of historic tidal or river flooding to the site have been identified.

3.5 Surface Water Flooding

The risk of surface water flooding at the proposed site is very low. This means there is less than 0.1% chance each year of surface water flooding. While there is an increase in the area of hardstanding landscape in the proposed scheme, this is of a very small area and unlikely to significantly influence the surface water flood risk.

3.6 Reservoir Flooding

A risk of flooding from reservoirs has been identified at the proposed site. The reservoirs that contribute to this risk are the Queen Mother, Queen Elizabeth II and the Queen Mary reservoirs which are owned by Thames Water. Additionally, the London Wetland Centre, located approximately 0.2km from the proposed site, comprises four unused reservoirs.

However, generally flooding from reservoirs is extremely unlikely and the risk only suggests that people's lives could be threatened in the event of a dam or a reservoir failure. Therefore, no mitigation plan is necessary.

3.7 Sewer Flooding

Basement structures can be susceptible to sewer flooding, which can occur due to exceeded sewer capacity and during periods of heavy rainfall. To reduce this risk, the pool house sewerage network should be installed with a non-return value device. Additionally, if the pool house substructure is likely to fall below the level of the gravity outfall drainage system, a pumped device shall be provided to lift the basement foul drainage to ground level before out falling by gravity to the combined public sewer.

3.8 Proposed Drainage Strategy

A CCTV survey of the existing drainage will be conducted by drainage contractors. It is expected that the main outfall joins a communal sewer in Castelnau. The existing ground foul and surface water drainage network will be retained where possible, damaged runs or runs needing diversion will be replaced. Where applicable, infiltration methods will form part of the drainage strategy and possibly including permeable paving, water butts, etc.

3.9 Impact on Neighbouring Properties

The location of the pool house in the rear garden of the property is in close proximity (approx. 0.5m) to a boundary with a neighbouring property, No. 73B Castelnau. This boundary is likely to be marked by either wooden fencing panels or a solid masonry wall.

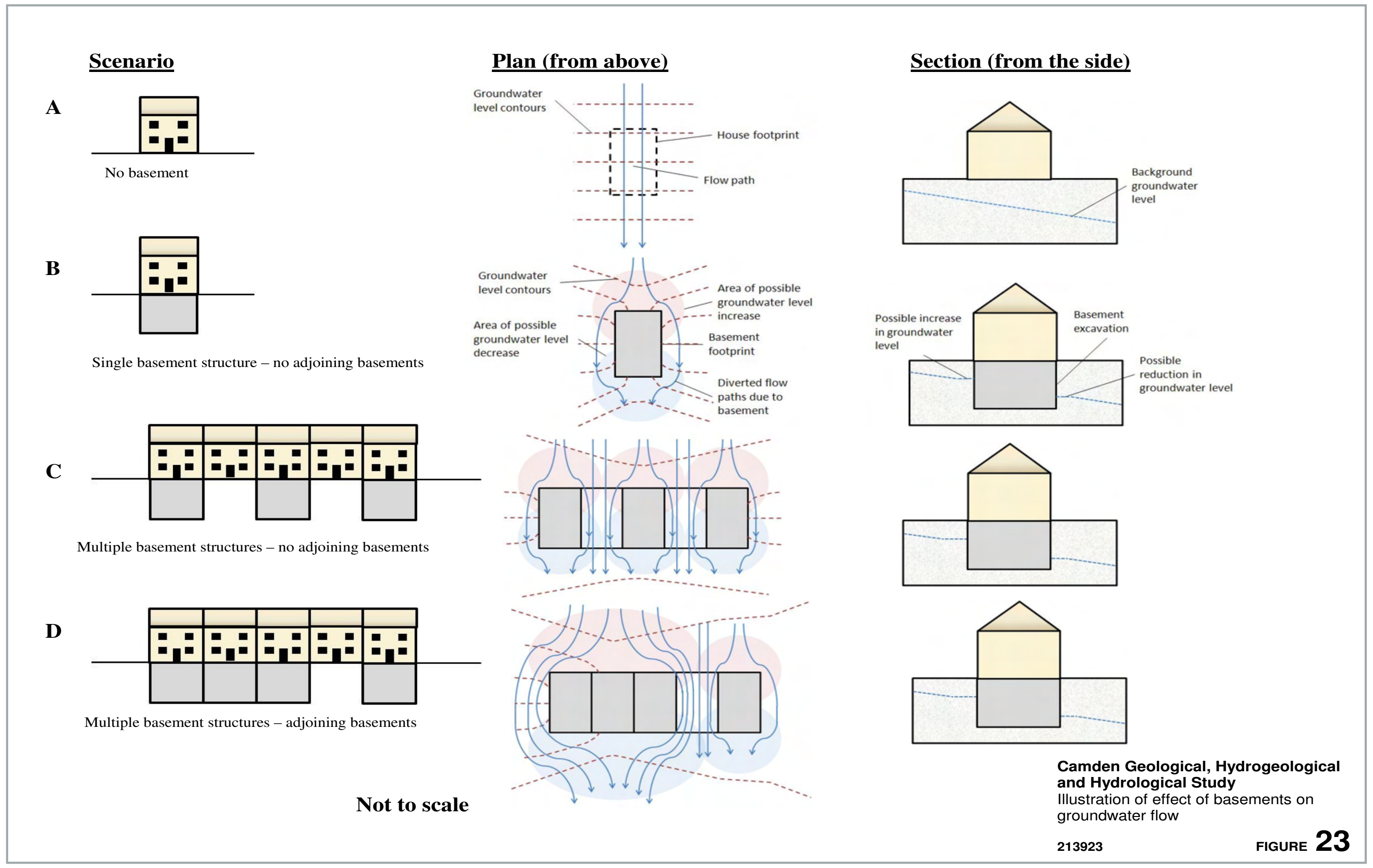
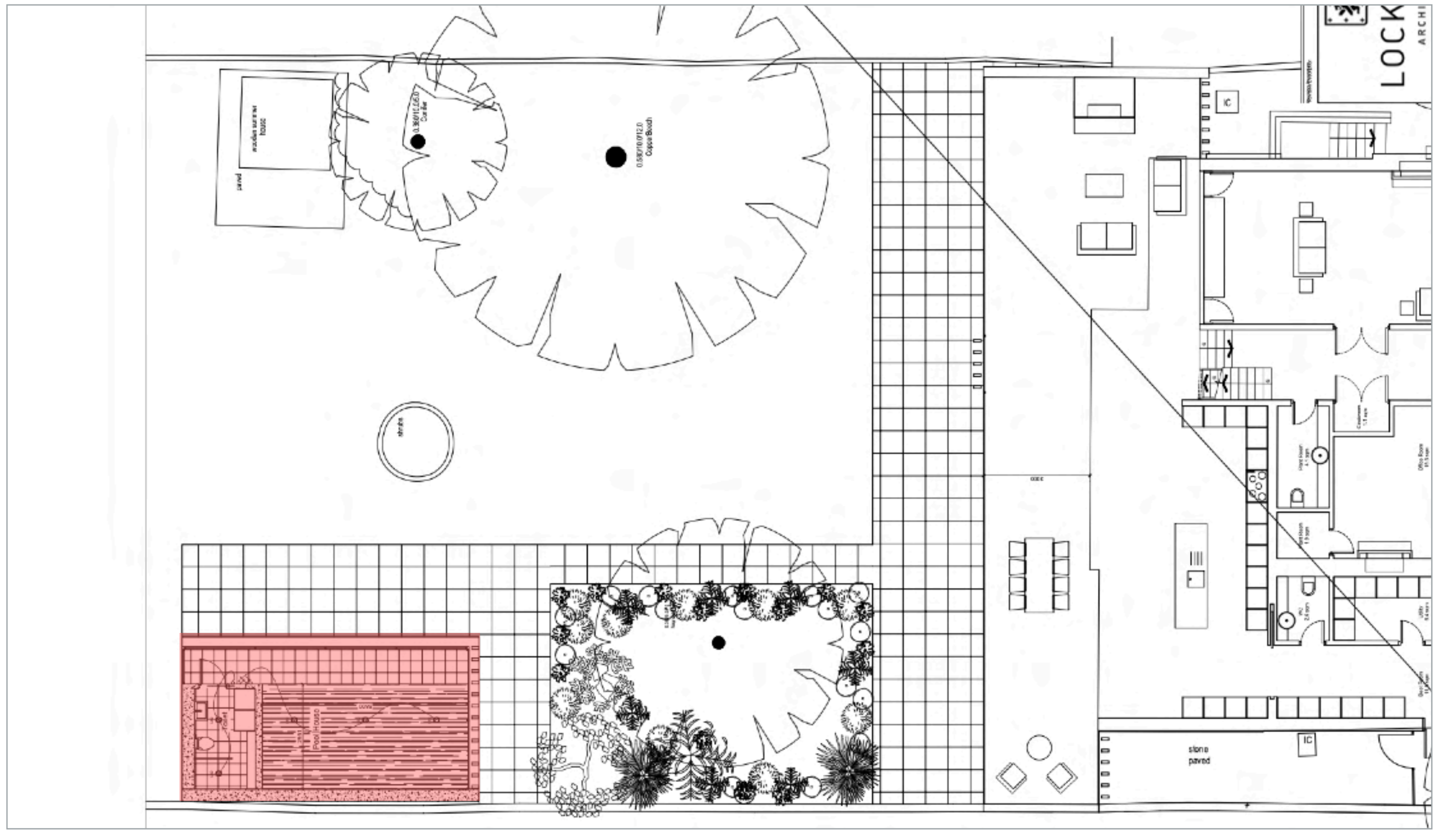


Illustration of the effect of basements on groundwater flow- Extract from 'Camden Geological, Hydrogeological and Hydrological Study / November 2010 - Arup'



Proposed landscape plan showing location of new pool house. Source: Locksley Architects

The proposed substructure of the pool house is very likely to extend below the footings of the wall. Depending on the construction of the boundary wall as well as its distance to the proposed pool house, a party wall agreement and underpinning works may be required for the construction of the pool house.

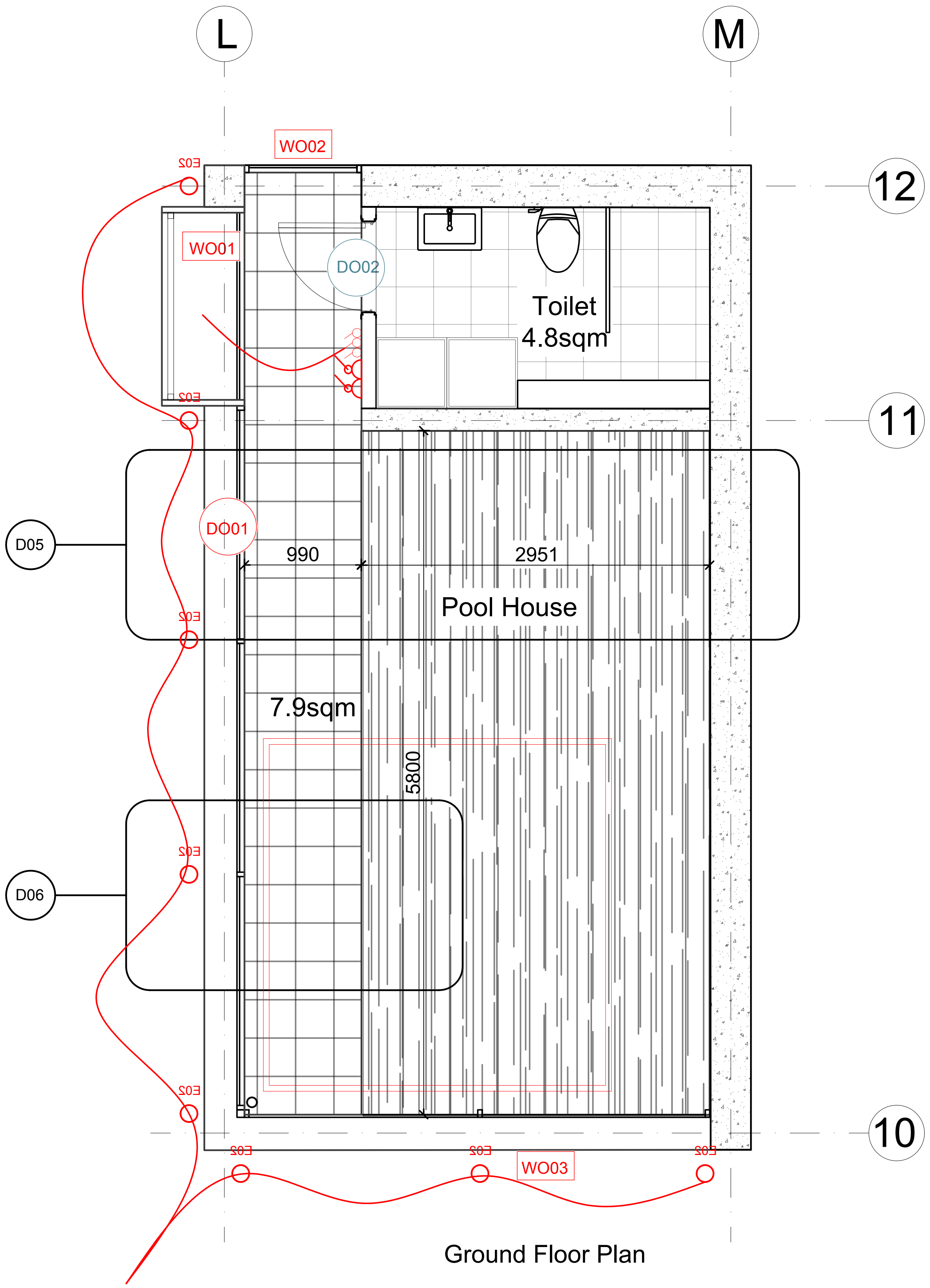
The substructure will be designed such that the stability of the boundary wall is not compromised both during the construction and in-use phases. This will primarily be achieved through the use of propping and sequential construction of the pool retaining walls. Additionally, the RC underpins (if required) and retaining walls will be designed as sufficiently stiff to minimise any lateral movement of the ground materials to within acceptable limits. The substructure will be constructed in sections each no wider than 1000mm, with no adjacent underpins constructed within a 48 hour period. This method of construction reduces the amount of potential ground movement and minimises the effects of settlements of the adjacent structures.

The construction of the pool house should not induce slope instability both due to the nature of the works and the existing topography.

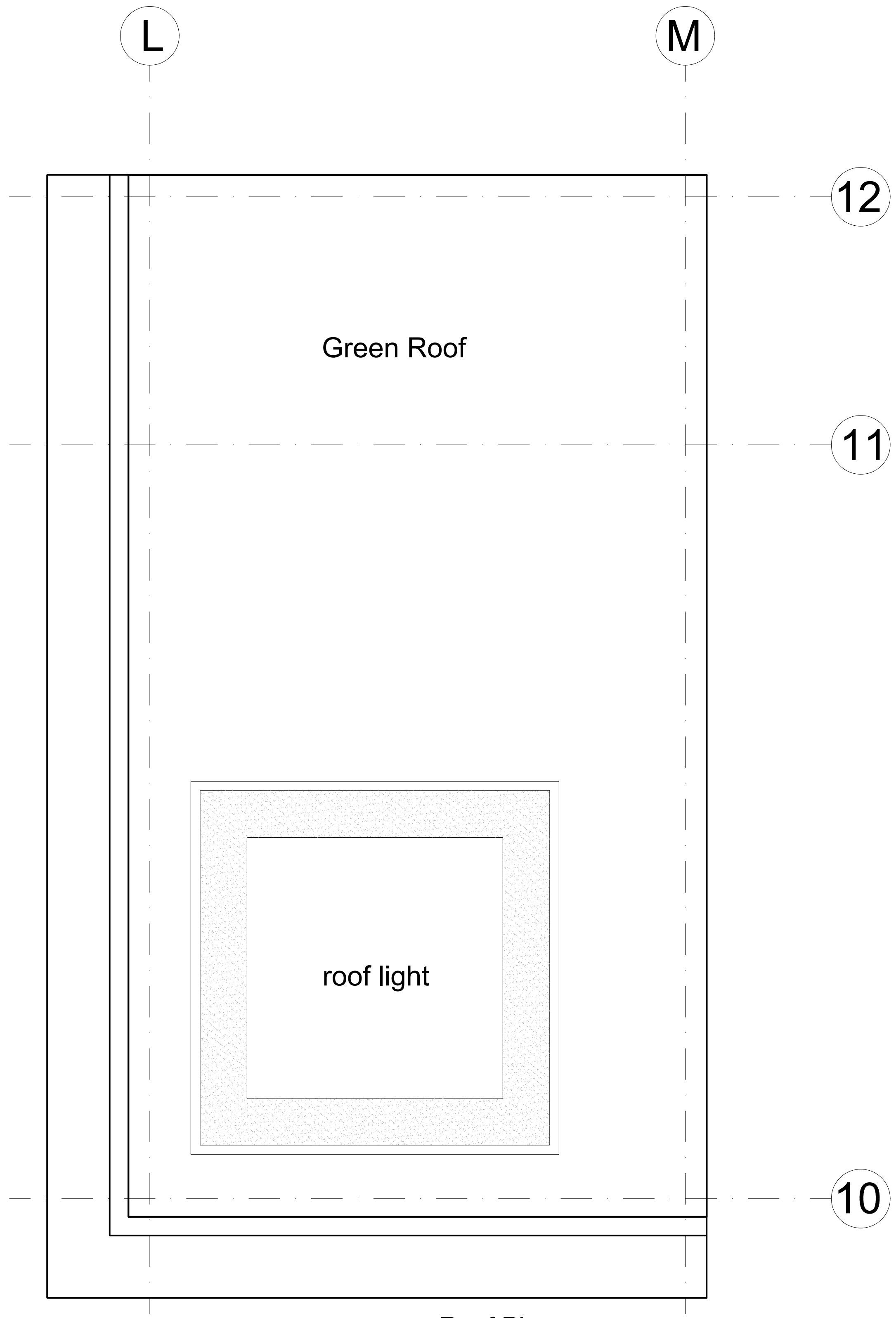
4.0 Trees

The site is located within the Castelnau Conservation Area. Consequently, a tree works planning application must be submitted prior to any works to the existing trees. Some trees in the borough are protected by the tree preservation order (TPO). Trees in conservation areas are also protected by law. According with TPO List from RBRUT, no protected trees exist on the site. The proposed pool house scheme does not require the removal of any trees as indicated by the Tree Constraints Plan and Proposed Landscape Plan in Appendices C and D, respectively.

However, two trees marked as Nos. 14 and 15 in the Tree Constraints Plan are rooted in close proximity to the proposed pool house location.
According to the Arboricultural Survey conducted by Arbtech Consulting (dated 02 Sept 2022), tree No. 14 is a Lawson Cypress of approximately 7m height and of Category C while No. 15 is Weeping Beech tree 6m in height and of Category B. Category B and C trees indicates trees of moderate and low quality, respectively. The presence of cohesive Kempton Park Gravel Member strata is anticipated as suggested by the historic boreholes and the BGS maps. These soils have volume change potential which can be problematic in the presence of the tree roots belonging to trees. No. 14 and 15. It is advised that foundations are constructed outside the zones comprising root-penetrated soils with volume change potential. NHBC Standards Chapter 4.2 will be used to identify the tree influence rings and the pool house substructure will be designed accordingly. Additionally, the volume change potential of cohesive soils i.e. heave/uplift will also be taken into account in the structural design of the pool retaining walls and base slab. Measures may include heave protection, for example.



Ground Floor Plan



Roof Plan

NORTH:

SCALE at A3:

0m 5m.

GENERAL NOTES:

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PROJECT NOTES:

Revision Description	Date	Rev
Issued for Client Approval	10/01/2023	C
Issued for Planning	01/11/2022	B
Issued for Client Approval	21/10/2022	A

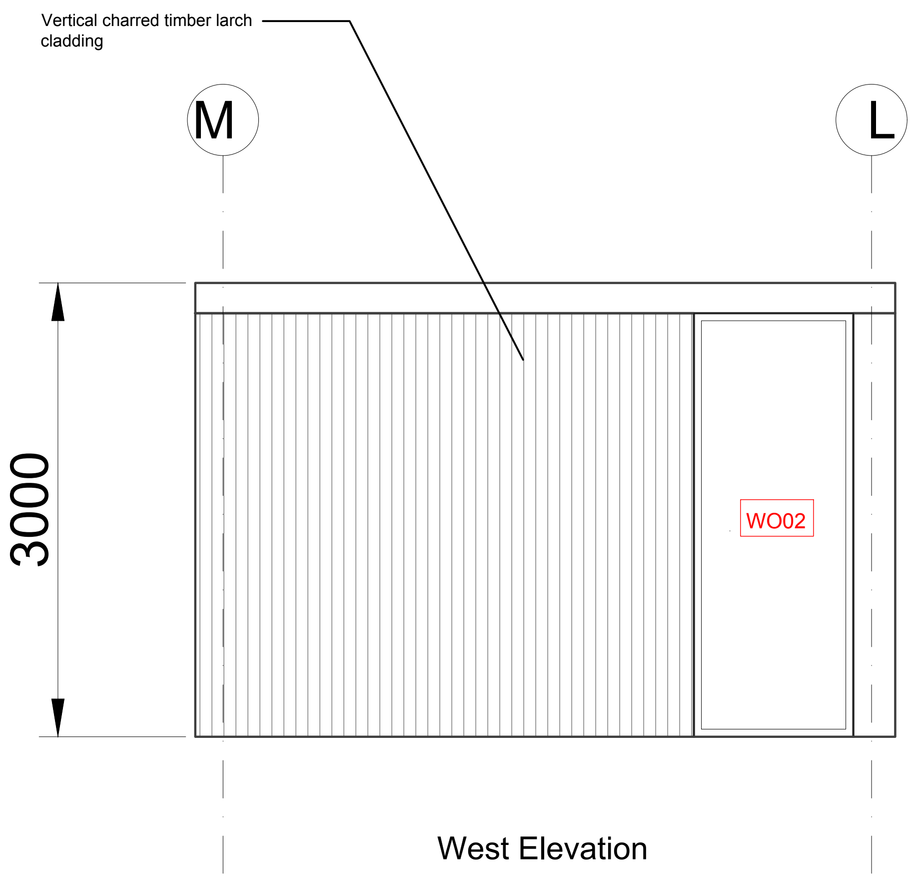
LOCKSLEY
ARCHITECTS

Architect Details
118 Campden Hill Road, Kensington, London W8 7AR
info@locksleyarchitects.com
0207 206 2727

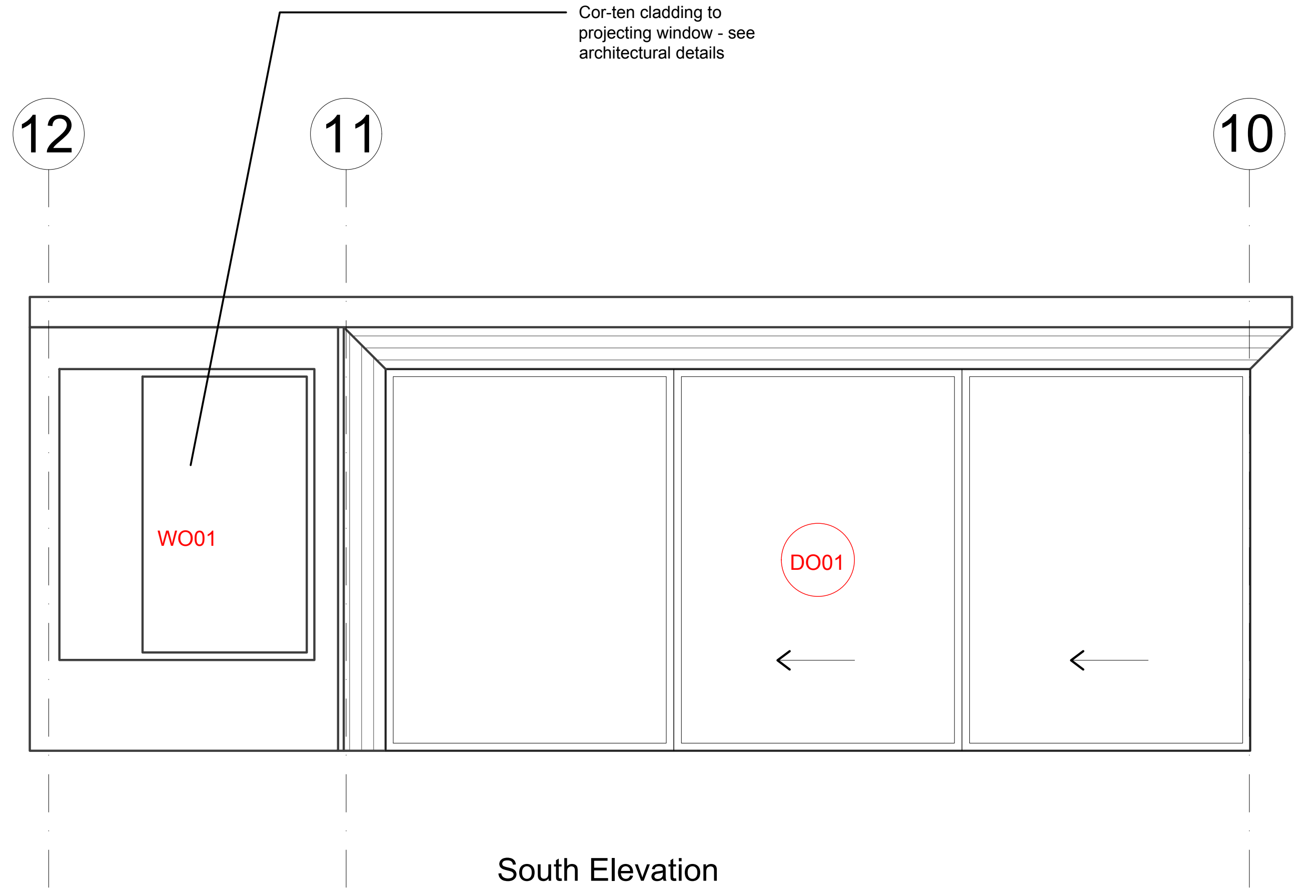
Project
73 Castelnau

Title
Pool House Drawings

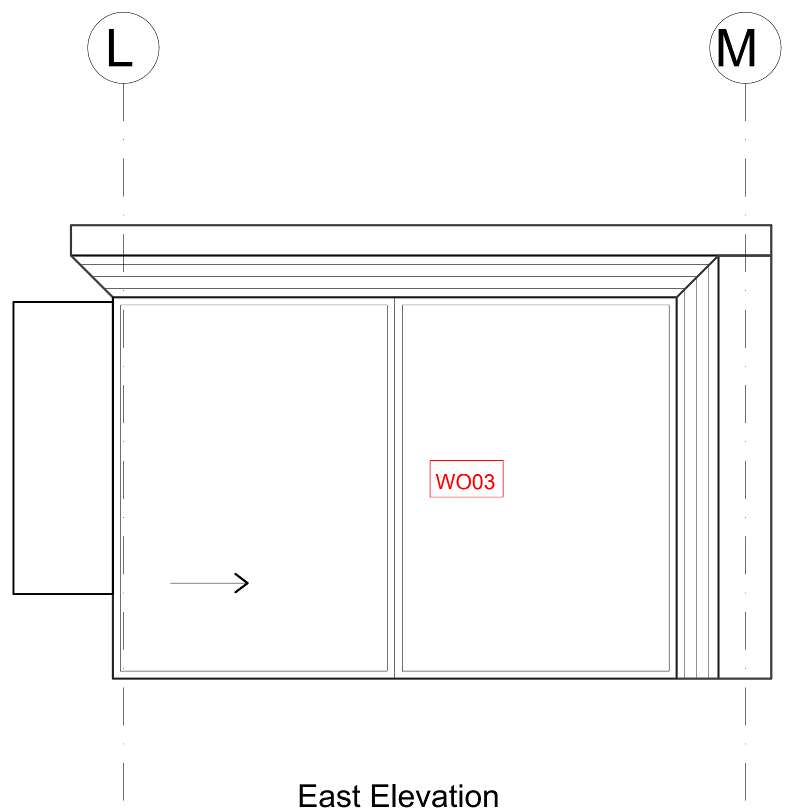
Job No	Drg No	Revision
1087	P11	C



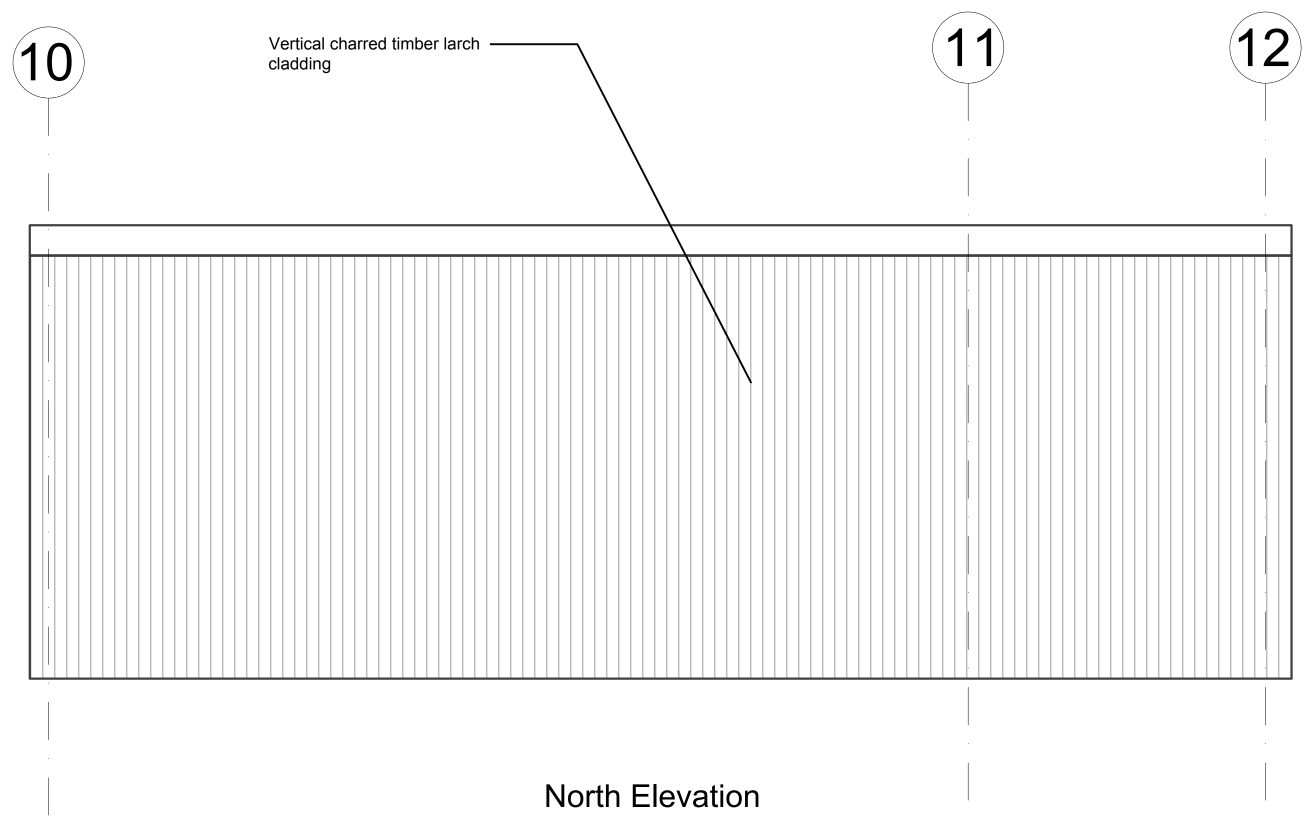
West Elevation



South Elevation



East Elevation



North Elevation

NORTH:

SCALE at A3:

0m 5m.

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LOCKSLEY ARCHITECTS

Architect Details
118 Campden Hill Road, Kensington, London W8 7AR
info@locksleyarchitects.com
0207 206 2727

Project
73 Castelnau

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Pool House Drawings

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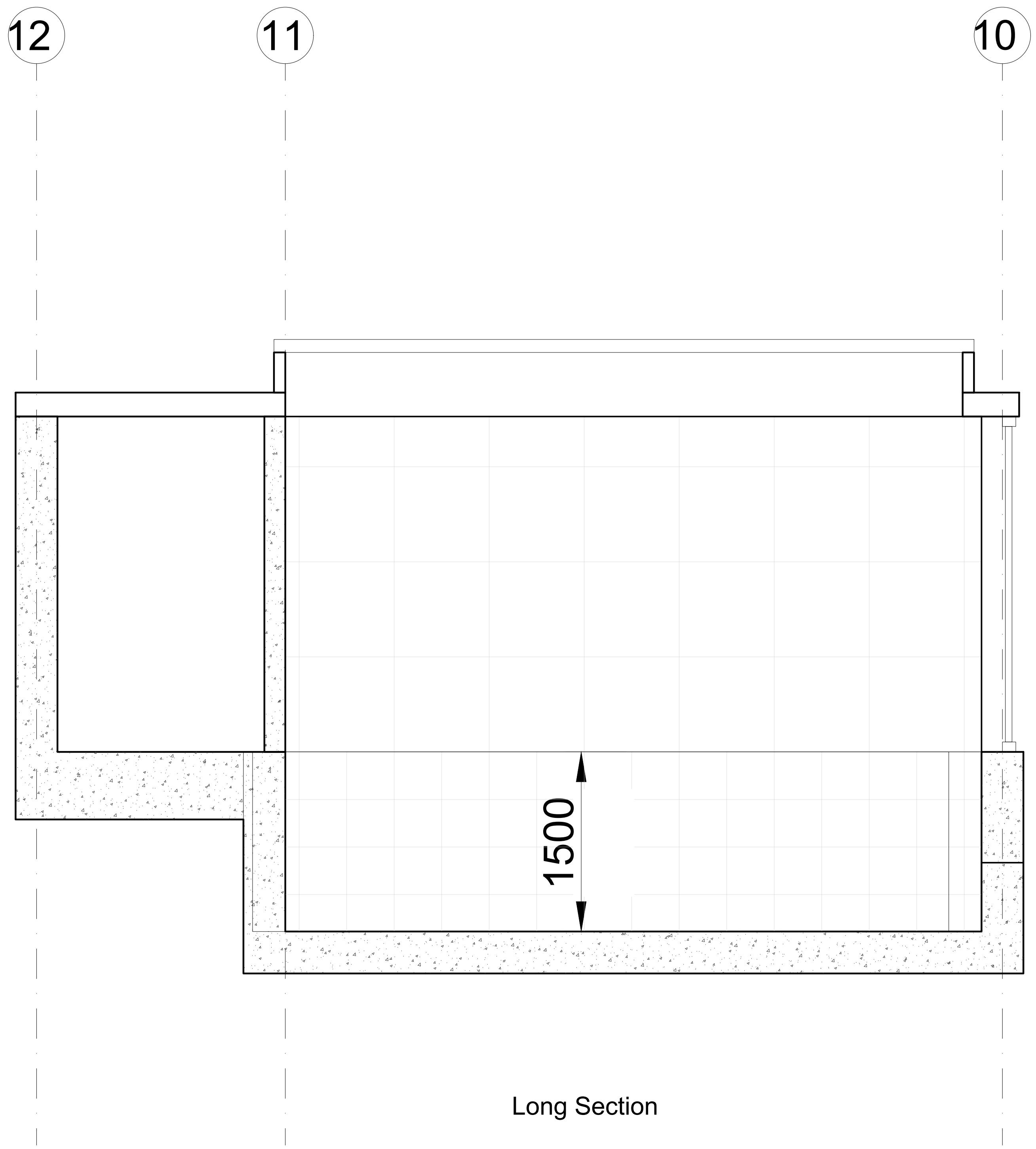
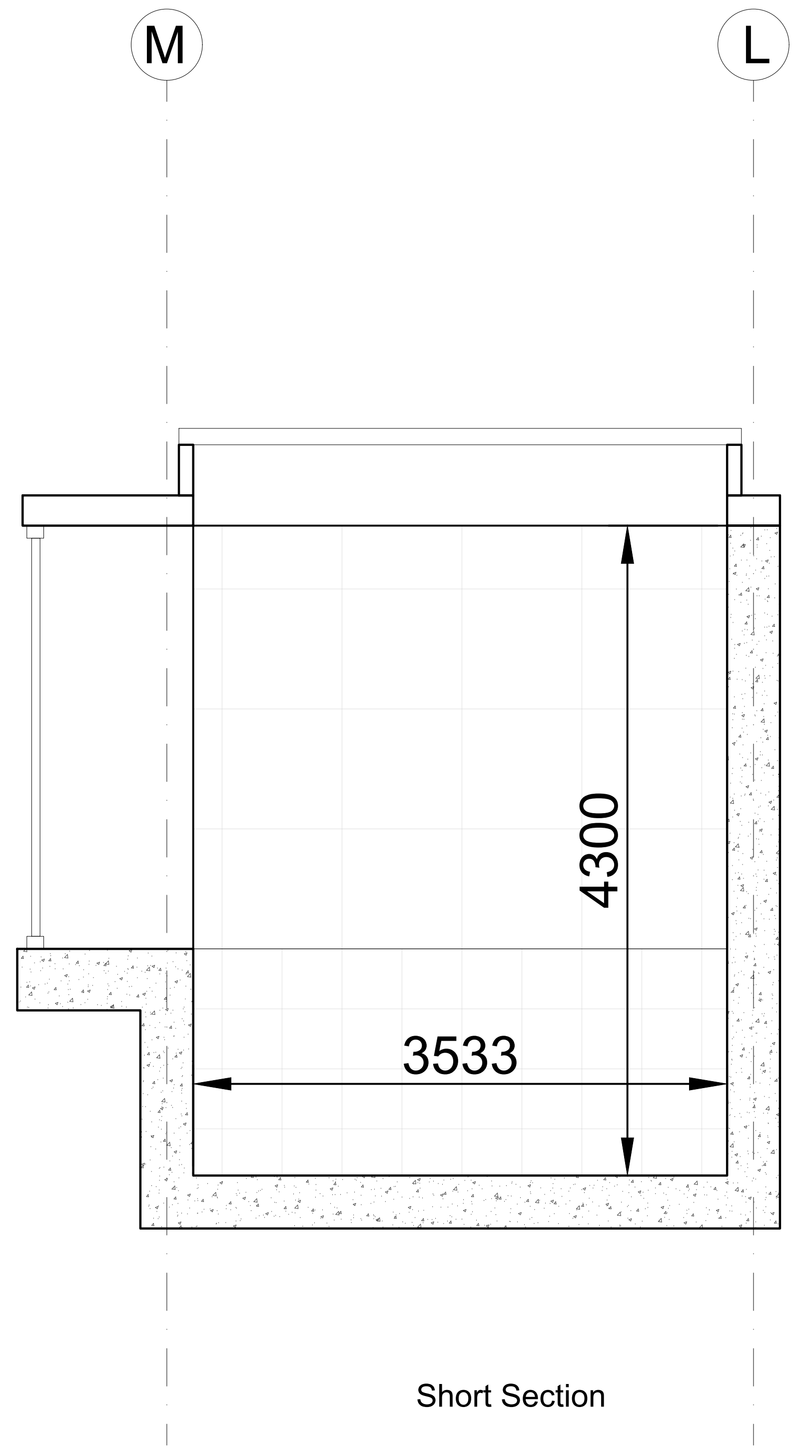
1087	P12	C



Long Section



Short Section

NORTH:

SCALE at A3:

0m
5m.

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LOCKSLEY
ARCHITECTS

Architect Details
118 Campden Hill Road, Kensington, London W8 7AR
info@locksleyarchitects.com
0207 206 2727

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Pool House Drawings

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1087	P13	C

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 2.9.17

Retaining wall details

Stem type	Cantilever
Stem height	h_{stem} = **1500** mm
Stem thickness	t_{stem} = **250** mm
Angle to rear face of stem	α = **90** deg
Stem density	γ_{stem} = **25** kN/m^3
Toe length	l_{toe} = **1000** mm
Heel length	l_{heel} = **200** mm
Base thickness	t_{base} = **300** mm
Base density	γ_{base} = **25** kN/m^3
Height of retained soil	h_{ret} = **1500** mm
Angle of soil surface	β = **0** deg
Depth of cover	d_{cover} = **0** mm
Height of water	h_{water} = **700** mm
Water density	γ_w = **9.8** kN/m^3

Retained soil properties

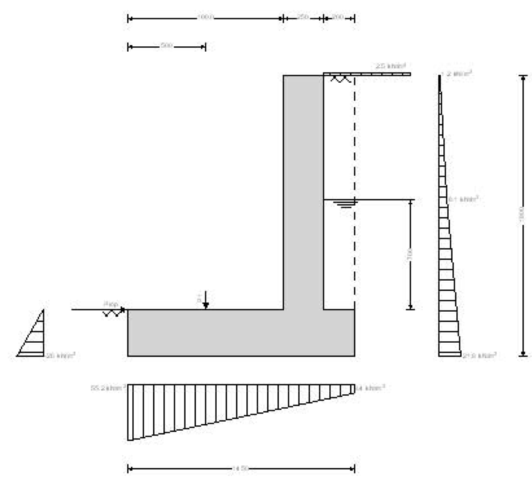
Soil type	Firm clay
Moist density	γ_{mr} = **18** kN/m^3
Saturated density	γ_{sr} = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{r.k}$ = **18** deg
Characteristic wall friction angle	$\delta_{r.k}$ = **9** deg

Base soil properties

Soil type	Medium dense well graded sand
Soil density	γ_b = **18** kN/m^3
Characteristic effective shear resistance angle	$\phi'_{b.k}$ = **30** deg
Characteristic wall friction angle	$\delta_{b.k}$ = **15** deg
Characteristic base friction angle	$\delta_{bb.k}$ = **30** deg
Presumed bearing capacity	$P_{bearing}$ = **100** kN/m^2

Loading details

Variable surcharge load	$Surcharge_Q$ = **2.5** kN/m^2
Vertical line load at 500 mm	P_{G1} = **20** kN/m



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length	$l_{base} = l_{toe} + t_{stem} + l_{heel}$ = **1450** mm
Saturated soil height	$h_{sat} = h_{water} + d_{cover}$ = **700** mm
Moist soil height	$h_{moist} = h_{ret} - h_{water}$ = **800** mm
Length of surcharge load	$l_{sur} = l_{heel}$ = **200** mm
- Distance to vertical component	$x_{sur\_v} = l_{base} - l_{heel} / 2$ = **1350** mm
Effective height of wall	$h_{eff} = h_{base} + d_{cover} + h_{ret}$ = **1800** mm
- Distance to horizontal component	$x_{sur\_h} = h_{eff} / 2$ = **900** mm
Area of wall stem	$A_{stem} = h_{stem} \times t_{stem}$ = **0.375** m^2
- Distance to vertical component	$x_{stem} = l_{toe} + t_{stem} / 2$ = **1125** mm
Area of wall base	$A_{base} = l_{base} \times t_{base}$ = **0.435** m^2
- Distance to vertical component	$x_{base} = l_{base} / 2$ = **725** mm
Area of saturated soil	$A_{sat} = h_{sat} \times l_{heel}$ = **0.14** m^2
- Distance to vertical component	$x_{sat\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat}$ = **1350** mm
- Distance to horizontal component	$x_{sat\_h} = (h_{sat} + h_{base}) / 3$ = **333** mm
Area of water	$A_{water} = h_{sat} \times l_{heel}$ = **0.14** m^2
- Distance to vertical component	$x_{water\_v} = l_{base} - (h_{sat} \times l_{heel}^2 / 2) / A_{sat}$ = **1350** mm
- Distance to horizontal component	$x_{water\_h} = (h_{sat} + h_{base}) / 3$ = **333** mm
Area of moist soil	$A_{moist} = h_{moist} \times l_{heel}$ = **0.16** m^2
- Distance to vertical component	$x_{moist\_v} = l_{base} - (h_{moist} \times l_{heel}^2 / 2) / A_{moist}$ = **1350** mm

- Distance to horizontal component — $x_{moist\_h} = (h_{moist} \times (t_{base} + h_{sat} + h_{moist} / 3) / 2 + (h_{sat} + t_{base})^2/2) / (h_{sat} + t_{base} + h_{moist} / 2) = \mathbf{719}$ mm

Using Coulomb theory

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

Passive pressure coefficient — $K_P = \sin(90 - \phi'_{b.k})^2 / (\sin(90 + \delta_{b.k}) \times [1 - \sqrt{[\sin(\phi'_{b.k} + \delta_{b.k}) \times \sin(\phi'_{b.k}) / (\sin(90 + \delta_{b.k}))]}]^2) = \mathbf{4.977}$

Bearing pressure check

Vertical forces on wall

Wall stem	$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{9.4}$ kN/m
Wall base	$F_{base} = A_{base} \times \gamma_{base} = \mathbf{10.9}$ kN/m
Surcharge load	$F_{sur\_v} = Surcharge_Q \times l_{heel} = \mathbf{0.5}$ kN/m
Line loads	$F_{P\_v} = P_{G1} = \mathbf{20}$ kN/m
Saturated retained soil	$F_{sat\_v} = A_{sat} \times (\gamma_{sr} - \gamma_w) = \mathbf{1.1}$ kN/m
Water	$F_{water\_v} = A_{water} \times \gamma_w = \mathbf{1.4}$ kN/m
Moist retained soil	$F_{moist\_v} = A_{moist} \times \gamma_{mr} = \mathbf{2.9}$ kN/m
Total	$F_{total\_v} = F_{stem} + F_{base} + F_{sur\_v} + F_{P\_v} + F_{sat\_v} + F_{water\_v} + F_{moist\_v} = \mathbf{46.2}$ kN/m

Horizontal forces on wall

Surcharge load	$F_{sur\_h} = K_A \times \cos(\delta_{r.k}) \times Surcharge_Q \times h_{eff} = \mathbf{2.1}$ kN/m
Saturated retained soil	$F_{sat\_h} = K_A \times \cos(\delta_{r.k}) \times (\gamma_{sr} - \gamma_w) \times (h_{sat} + h_{base})^2 / 2 = \mathbf{2}$ kN/m
Water	$F_{water\_h} = \gamma_w \times (h_{water} + d_{cover} + h_{base})^2 / 2 = \mathbf{4.9}$ kN/m
Moist retained soil	$F_{moist\_h} = K_A \times \cos(\delta_{r.k}) \times \gamma_{mr} \times ((h_{eff} - h_{sat} - h_{base})^2 / 2 + (h_{eff} - h_{sat} - h_{base}) \times (h_{sat} + h_{base})) = \mathbf{9.6}$ kN/m
Base soil	$F_{pass\_h} = -K_P \times \cos(\delta_{b.k}) \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-3.9}$ kN/m
Total	$F_{total\_h} = F_{sur\_h} + F_{sat\_h} + F_{water\_h} + F_{moist\_h} + F_{pass\_h} = \mathbf{14.7}$ kN/m

Moments on wall

Wall stem	$M_{stem} = F_{stem} \times x_{stem} = \mathbf{10.5}$ kNm/m
Wall base	$M_{base} = F_{base} \times x_{base} = \mathbf{7.9}$ kNm/m
Surcharge load	$M_{sur} = F_{sur\_v} \times x_{sur\_v} - F_{sur\_h} \times x_{sur\_h} = \mathbf{-1.3}$ kNm/m
Line loads	$M_P = P_{G1} \times p_1 = \mathbf{10}$ kNm/m
Saturated retained soil	$M_{sat} = F_{sat\_v} \times x_{sat\_v} - F_{sat\_h} \times x_{sat\_h} = \mathbf{0.9}$ kNm/m
Water	$M_{water} = F_{water\_v} \times x_{water\_v} - F_{water\_h} \times x_{water\_h} = \mathbf{0.2}$ kNm/m
Moist retained soil	$M_{moist} = F_{moist\_v} \times x_{moist\_v} - F_{moist\_h} \times x_{moist\_h} = \mathbf{-3}$ kNm/m
Total	$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{sat} + M_{water} + M_{moist} = \mathbf{25.3}$ kNm/m

Check bearing pressure

Propping force	$F_{prop\_base} = F_{total\_h} = \mathbf{14.7}$ kN/m
Distance to reaction	$\bar{x} = M_{total} / F_{total\_v} = \mathbf{547}$ mm
Eccentricity of reaction	$e = \bar{x} - l_{base} / 2 = \mathbf{-178}$ mm
Loaded length of base	$l_{load} = l_{base} = \mathbf{1450}$ mm
Bearing pressure at toe	$q_{toe} = F_{total\_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{55.2}$ kN/m^2
Bearing pressure at heel	$q_{heel} = F_{total\_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{8.4}$ kN/m^2

Factor of safety — $FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.811}$

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 2.9.17

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	$\lambda = \mathbf{0.80}$
Effective strength factor	$\eta = \mathbf{1.00}$
Bending coefficient k_1	$K_1 = \mathbf{0.40}$
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = \mathbf{1.00}$
Bending coefficient k_3	$K_3 = \mathbf{0.40}$
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\varepsilon_{cu2}) = \mathbf{1.00}$

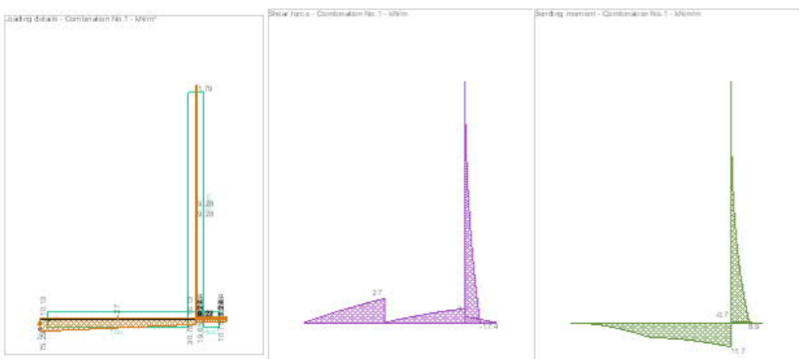
Reinforcement details

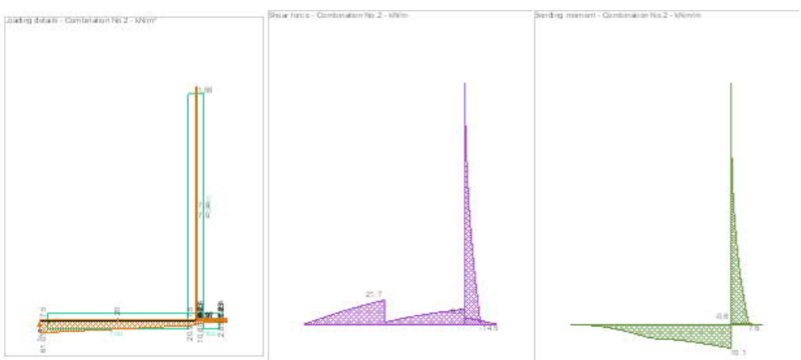
Characteristic yield strength of reinforcement	$f_{yk} = \mathbf{500}$ N/mm^2
Modulus of elasticity of reinforcement	$E_s = \mathbf{200000}$ N/mm^2
Partial factor for reinforcing steel - Table 2.1N	$\gamma_S = \mathbf{1.15}$
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S = \mathbf{435}$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

Cover to reinforcement

Front face of stem	$c_{sf} = \mathbf{40}$ mm
Rear face of stem	$c_{sr} = \mathbf{50}$ mm
Top face of base	$c_{bt} = \mathbf{50}$ mm
Bottom face of base	$c_{bb} = \mathbf{75}$ mm

Jensen Hunt Design	Project 73 Castelnau			Job Ref. 1751	
	Section Poolhouse Substructure			Sheet no./rev. 5	
	Calc. by AM	Date 11/30/2022	Chk'd by	Date	App'd by	Date





Check stem design at base of stem

Depth of section — $h = \mathbf{250}$ mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 — $M = \mathbf{8.9}$ kNm/m

Depth to tension reinforcement — $d = h - c_{sr} - \phi_{sr} / 2 = \mathbf{194}$ mm

$K = M / (d^2 \times f_{ck}) = \mathbf{0.008}$

$K' = (2 \times \eta \times \alpha_{cc}/\gamma_C)\times(1 - \lambda \times (\delta - K_1)/(2 \times K_2))\times(\lambda \times (\delta - K_1)/(2 \times K_2))$

$K' = \mathbf{0.207}$

K' > K - No compression reinforcement is required

Lever arm — $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_C))^{0.5}, 0.95) \times d = \mathbf{184}$ mm

Depth of neutral axis — $x = 2.5 \times (d – z) = \mathbf{24}$ mm

Area of tension reinforcement required — $A_{sr.req} = M / (f_{yd} \times z) = \mathbf{111}$ mm²/m

Jensen Hunt Design	Project 73 Castelnau			Job Ref. 1751	
	Section Poolhouse Substructure			Sheet no./rev. 6	
	Calc. by AM	Date 11/30/2022	Chk'd by	Date	App'd by	Date

Tension reinforcement provided — 12 dia.bars @ 200 c/c

Area of tension reinforcement provided — $A_{sr.prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = \mathbf{565}$ mm²/m

Minimum area of reinforcement - exp.9.1N — $A_{sr.min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{292}$ mm²/m

Maximum area of reinforcement - cl.9.2.1.1(3) — $A_{sr.max} = 0.04 \times h = \mathbf{10000}$ mm²/m

$\max(A_{sr.req}, A_{sr.min}) / A_{sr.prov} = \mathbf{0.517}$

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 — $\rho_0 = \sqrt{(f_{ck} / 1\ N/mm^2)} / 1000 = \mathbf{0.005}$

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

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

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

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

Limiting span to depth ratio - exp.7.16.a — $\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1\ N/mm^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = \mathbf{16}$

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

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

Crack control - Section 7.3

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

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

Serviceability bending moment — $M_{sls} = \mathbf{5.9}$ kNm/m

Tensile stress in reinforcement — $\sigma_s = M_{sls} / (A_{sr.prov} \times z) = \mathbf{56.9}$ N/mm²

Load duration — Long term

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

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

$A_{c.eff} = \mathbf{75250}$ mm²/m

Mean value of concrete tensile strength — $f_{ct.eff} = f_{ctm} = \mathbf{2.9}$ N/mm²

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

Modular ratio — $\alpha_e = E_s / E_{cm} = \mathbf{6.091}$

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

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

$k_3 = \mathbf{3.4}$

$k_4 = \mathbf{0.425}$

Maximum crack spacing - exp.7.11 — $s_{r.max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p.eff} = \mathbf{441}$ mm

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

$w_k = \mathbf{0.075}$ mm

$w_k / w_{max} = \mathbf{0.251}$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force — $V = \mathbf{17.4}$ kN/m

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

$k = \min(1 + \sqrt{(200\ mm / d)}, 2) = \mathbf{2.000}$

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

	v_{min} = 0.035 $N^{1/2}$/mm × $k^{3/2}$ × $f_{ck}^{0.5}$ = **0.542** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c}$ = max($C_{Rd.c}$ × k × (100 N²/mm⁴ × ρ_l × f_{ck})$^{1/3}$, v_{min}) × d
	$V_{Rd.c}$ = **105.2** kN/m
	V / $V_{Rd.c}$ = **0.166**

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1)	$A_{sx.req}$ = max(0.25 × $A_{sr.prov}$, 0.001 × t_{stem}) = **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	$A_{sx.prov}$ = π × ϕ_{sx}^2 / (4 × 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 = **11.7** kNm/m
Depth to tension reinforcement	d = h - c_{bb} - ϕ_{bb} / 2 = **219** mm
	K = M / (d² × f_{ck}) = **0.008**
	K' = (2 × η × α_{cc}/γ_C)×(1 - λ × (δ - K_1)/(2 × K_2))×(λ × (δ - K_1)/(2 × K_2))
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	z = min(0.5 + 0.5 × (1 - 2 × K / (η × α_{cc} / γ_C))$^{0.5}$, 0.95) × d = **208** mm
Depth of neutral axis	x = 2.5 × (d – z) = **27** mm
Area of tension reinforcement required	$A_{bb.req}$ = M / (f_{yd} × z) = **129** mm²/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{bb.prov}$ = π × ϕ_{bb}^2 / (4 × s_{bb}) = **565** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{bb.min}$ = max(0.26 × f_{ctm} / f_{yk}, 0.0013) × d = **330** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bb.max}$ = 0.04 × h = **12000** mm²/m
	max($A_{bb.req}$, $A_{bb.min}$) / $A_{bb.prov}$ = **0.583**

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

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 = **0.6**
Serviceability bending moment	M_{sls} = **8.5** kNm/m
Tensile stress in reinforcement	σ_s = M_{sls} / ($A_{bb.prov}$ × z) = **72.1** 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 × (h - d), (h - x) / 3, h / 2)
	$A_{c.eff}$ = **90875** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff}$ = f_{ctm} = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff}$ = $A_{bb.prov}$ / $A_{c.eff}$ = **0.006**
Modular ratio	α_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 × c_{bb} + k_1 × k_2 × k_4 × ϕ_{bb} / $\rho_{p.eff}$ = **583** mm
Maximum crack width - exp.7.8	w_k = $s_{r.max}$ × max(σ_s – k_t × ($f_{ct.eff}$ / $\rho_{p.eff}$) × (1 + α_e × $\rho_{p.eff}$), 0.6 × σ_s) / E_s
	w_k = **0.126** mm
	w_k / w_{max} = **0.42**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force	V = **27** kN/m
	$C_{Rd.c}$ = 0.18 / γ_C = **0.120**
	k = min(1 + √(200 mm / d), 2) = **1.956**
Longitudinal reinforcement ratio	ρ_l = min($A_{bb.prov}$ / d, 0.02) = **0.003**
	v_{min} = 0.035 $N^{1/2}$/mm × $k^{3/2}$ × $f_{ck}^{0.5}$ = **0.524** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c}$ = max($C_{Rd.c}$ × k × (100 N²/mm⁴ × ρ_l × f_{ck})$^{1/3}$, v_{min}) × d
	$V_{Rd.c}$ = **114.8** kN/m
	V / $V_{Rd.c}$ = **0.235**

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section	h = **300** mm

Rectangular section in flexure - Section 6.1

Design bending moment combination 1	M = **0.7** kNm/m
Depth to tension reinforcement	d = h - c_{bt} - ϕ_{bt} / 2 = **244** mm
	K = M / (d² × f_{ck}) = **0.000**
	K' = (2 × η × α_{cc}/γ_C)×(1 - λ × (δ - K_1)/(2 × K_2))×(λ × (δ - K_1)/(2 × K_2))
	K' = **0.207**

K' > K - No compression reinforcement is required

Lever arm	z = min(0.5 + 0.5 × (1 - 2 × K / (η × α_{cc} / γ_C))$^{0.5}$, 0.95) × d = **232** mm
Depth of neutral axis	x = 2.5 × (d – z) = **31** mm
Area of tension reinforcement required	$A_{bt.req}$ = M / (f_{yd} × z) = **7** mm²/m
Tension reinforcement provided	12 dia.bars @ 200 c/c
Area of tension reinforcement provided	$A_{bt.prov}$ = π × ϕ_{bt}^2 / (4 × s_{bt}) = **565** mm²/m
Minimum area of reinforcement - exp.9.1N	$A_{bt.min}$ = max(0.26 × f_{ctm} / f_{yk}, 0.0013) × d = **368** mm²/m
Maximum area of reinforcement - cl.9.2.1.1(3)	$A_{bt.max}$ = 0.04 × h = **12000** mm²/m
	max($A_{bt.req}$, $A_{bt.min}$) / $A_{bt.prov}$ = **0.65**

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 = **0.6**
Serviceability bending moment	M_{sls} = **0.5** kNm/m
Tensile stress in reinforcement	σ_s = M_{sls} / ($A_{bt.prov}$ × z) = **3.9** 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 × (h - d), (h - x) / 3, h / 2)
	$A_{c.eff}$ = **89833** mm²/m
Mean value of concrete tensile strength	$f_{ct.eff}$ = f_{ctm} = **2.9** N/mm²
Reinforcement ratio	$\rho_{p.eff}$ = $A_{bt.prov}$ / $A_{c.eff}$ = **0.006**
Modular ratio	α_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	$s_{r.max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p.eff}$ = **494** 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 = **0.006** mm
	w_k / w_{max} = **0.019**

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

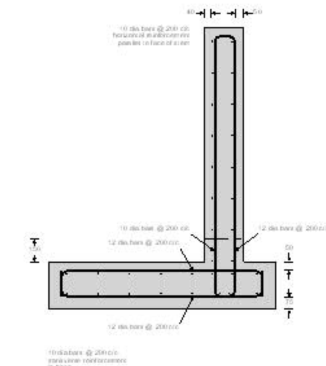
Design shear force	V = **7** kN/m
	$C_{Rd,c} = 0.18 / \gamma_C$ = **0.120**
	k = min(1 + √(200 mm / d), 2) = **1.905**
Longitudinal reinforcement ratio	ρ_l = min($A_{bt.prov}$ / d, 0.02) = **0.002**
	$v_{min} = 0.035\ N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5}$ = **0.504** N/mm²
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd.c} = \max(C_{Rd.c} \times k \times (100\ N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$
	$V_{Rd.c}$ = **123** kN/m
	V / $V_{Rd.c}$ = **0.057**

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)	$A_{bx.req} = 0.2 \times A_{bb.prov}$ = **113** mm²/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** mm²/m

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



Reinforcement details

Tree Categories

Trees are categorised in accordance with the cascade chart in Table 1 of the British Standard BS 5837:2012 'Trees in relation to design, demolition and construction - Recommendations'

Category 'U' - Trees in such condition that they cannot realistically be retained as living trees in context of the current land use for longer than 10 years.
Category 'A' - Trees of high quality with an estimated remaining life expectancy of at least 40 years.
Category 'B' - Trees of moderate quality with an estimated remaining life expectancy of at least 20 years.
Category 'C' - Trees of low quality with an estimated remaining life expectancy of at least 10 years, or young trees with a stem diameter below 150mm.

Root Protection Area

In order to avoid damage to the roots or rooting environment of retained trees, the Root Protection Areas (RPAs) should be plotted around each of the category A, B and C trees. This is a minimum area in m² which should be left undisturbed around each retained tree.

The RPA is calculated using the British Standard BS 5837:2012 'Trees in relation to design, demolition and construction - Recommendations.

The calculated RPA is capped to 707m², which is the equivalent to a circle with a radius of 15m. Where there appears to be restrictions to root growth the root protection area is reshaped to more accurately reflect the likely distribution of the roots.

Tree Survey Report

Please refer to Arbtech Consulting Ltd. Tree Survey Report and Tree Schedule for full details on all surveyed trees, hedgerows and major shrub groups.
All trees were surveyed and categorised in accordance with the guidance as set out in the British Standard BS5837:2012 Tree in relation to design, demolition and construction - Recommendations.

We make the following recommendation to ensure that no conditions relating to arboriculture are attached to any planning consent secured: obtain and arboricultural report to include:
a) An arboricultural impact assessment (AIA);
b) An arboricultural method statement (AMS); and
c) A tree protection plan (TPP).

arbtech
Unit 3, Well House Barns, Chester, CH4 0DH
https://arbtech.co.uk, 01244 661170

Project: 73 Castelnau, London, SW13 9RT.

Client: Locksley Architects

Drawing: Tree Constraints Plan

Based on: OS Tile & E01 A

Drawing No:	Rev:	
Arbtech TCP 01		
Date: Sept 2022	Scale: 1:100 @ A0	Drawn: MGM

Key:

Tree Nos.:	1	Tree Canopies:		Trunks:	
RPAs:		Category 'U' trees:		Category 'B' trees:	
Category 'C' trees:		Category 'C' groups:			

All dimensions should be checked on site. No dimensions are to be scaled from this drawing.
Please notify us of any discrepancies found. Arbtech Consulting Ltd. cannot be held responsible for inaccuracies in the base drawing in which this plan is based.
This drawing is designed to reflect the principles of the layout or design only, and relates only to the protection of retained trees.
This drawing is not to be read as a definitive part of the engineering or construction designs or method statement. An architect or structural engineer should be contacted over any matters of construction, detailing or specification and for any standards or regulatory requirements relating to proposed structures, hard surfacing or underground services.
This drawing was produced in colour - a monochrome copy should not be relied upon.

NORTH:

SCALE at A1:

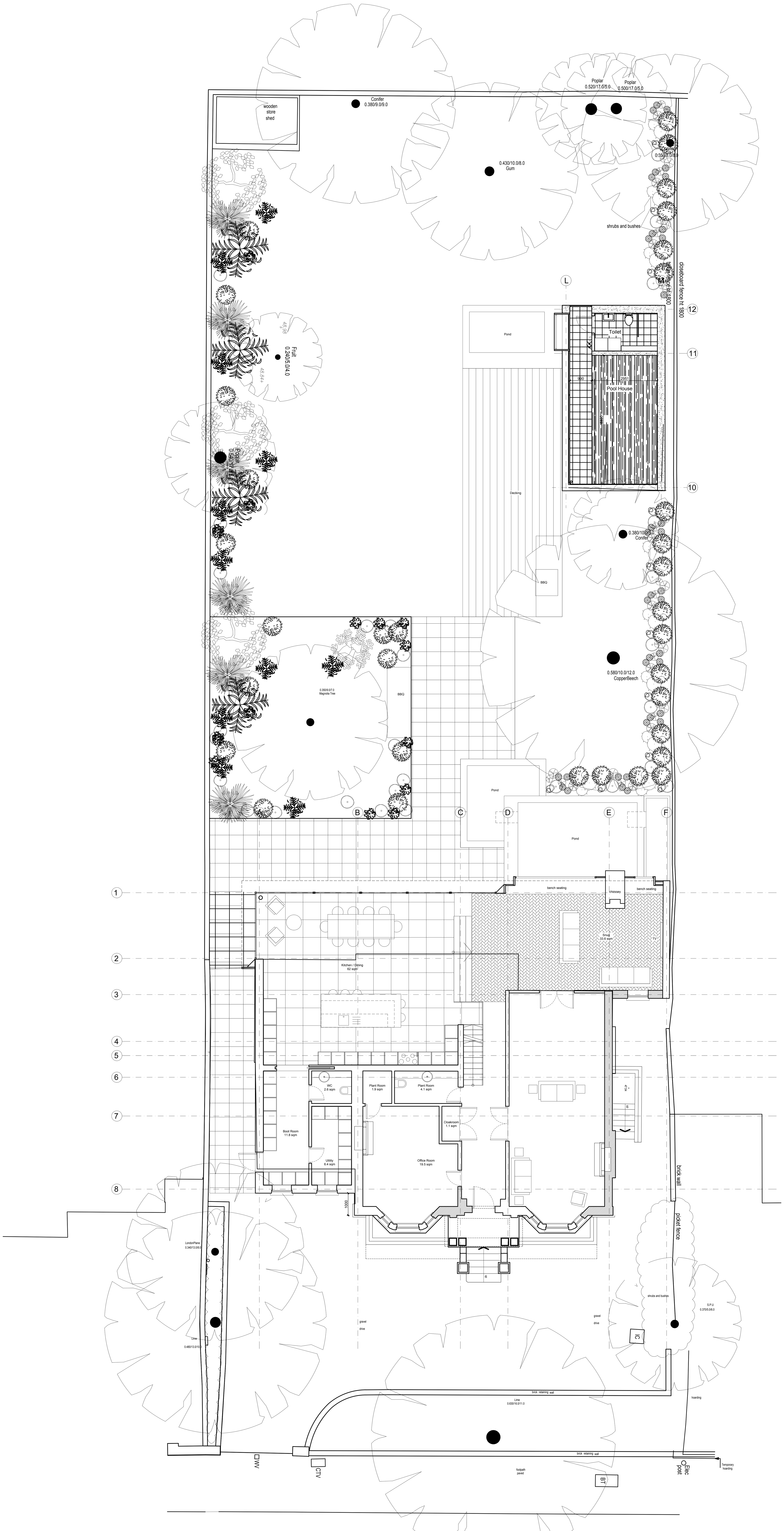
0m 5m.

GENERAL NOTES:

This drawing remains the property of Locksley Architects Ltd, reproduction shall only be allowed with written permission.

This drawing is not to be scaled for construction purposes, use written dimensions only. Any discrepencies to be report ed to the architect. All dimensions to be checked on site. The contractor is responsible for all dimensions and setting out of the work on site. The contractor must ensure that all elements of the work comply with current building regulations.

PROJECT NOTES:



Revision Description	Date	Rev
Issued for Client Approval	10/01/2023	C
Issued for Planning	01/11/2022	B
Issued for Client Approval	21/10/2022	A

LOCKSLEY
ARCHITECTS

Architect Details
118 Campden Hill Road, Kensington, London W8 7AR
info@locksleyarchitects.com
0207 206 2727

Project
73 Castelnau

Title
Proposed Landscape Plan

Job No	Drg No	Revision
1087	P10	C