79 GUILFORD ST

WC1N 1DF

Basement Impact Assessment,
Engineering Method Statement and
Construction Management Plan

Project Ref: J001413

April 2019

Revision 0

Unit 5, Quayside, Sir William Morris Way, Fulham SW6 2UZ
Registered in England Number 06533911

REVISION HISTORY

Rev	Purpose	Date	Issued By	Approved
Rev 0	Initial report	26-04-2019	VF	PB

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1. INTRODUCTION

Green Structural Engineering (GSE) has been involved in the design of a significant number of successful basements in a number of London boroughs on behalf of private clients, developers and contractors. We also undertake the temporary works design and sequencing for a number of contractors who operate across London.

Basement projects previously undertaken successfully have been of a similar size to that proposed in this application and quite often on a much larger scale and complexity.

This experience has positioned GSE at the forefront of basement design and indeed temporary works design for basement construction. This experience has led to an in-depth understanding and appreciation of the design parameters that should be considered for all basement construction projects.

GSE holds is a member of the ACE.

This report has been prepared as part of the planning application for 79 Guilford St on behalf of our client and is not to be used by any other parties or for any other purpose without the express written consent of GSE.

2. SCOPE OF REPORT

This report deals with the structural aspects of the proposed basement construction to 79 Guilford St and is to be read in conjunction with the following reports:

- Geotechnical report by GabrielGeo Consulting (enclosed in Appendix D)

This report is produced for submission to the London Borough of Camden as part of a planning application for works to 79 Guilford St and should not be used for any other purposes, e.g. construction or Party Wall Awards.

3. SCOPE OF WORKS

The proposal includes extending the existing basement further backwards and also reconfiguring the existing basement. A new rear light well will also be created to provide natural light and ventilation to the new basement rooms.

Investigation works on site to confirm the existing arrangement of the ground floor and the detailed design of the new permanent structure to basement and ground floor will be carried out as part of the detailed design process and are not included within this BIA report.

4. DESCRIPTION OF EXISTING SITE

The existing properties along this section of Guilford Street were constructed around 1874-1882. The footprint of the existing property resembles that shown on historical maps in late 1874s and shows signs of construction at the back of the rear garden.

Guilford Street is located on a broadly east/south-east facing slope which leads down to the base of a very shallow valley which was formed by the River Fleet, one of the 'lost' rivers of London. The location of this valley is defined by the 15m and 20m contours to the east of No.79. The contours on "Ordinance Survey map" indicate an overall slope across the site of approximately 0.4° towards the east, calculated between the 25m contour to the west and 20m contour to the east. Using the spot heights from Figure 1, the Guilford Street carriageway outside No.79 falls north-eastwards with a slope angle of 0.13°. Thus, the proposed basement excavation raises no concerns in relation to the overall stability of the slope, subject to normal precautions in supporting the ground around the basement.

All utilities and services assumed to be located within the adjacent street.

Russel Square underground station is located approximately 150m west from the site. Environmental search carried by GabrielGeo consulting confirmed underground tunnels within 250m from the site. Precisely, 112m towards west. Also, historical railway and tunnel features have been identified within 250m of the study side boundary (pag.56)

5. DESCRIPTION OF 79 GUILFORD STREET AND ADJOINING PROPERTIES

The property is part of a terrace of 23 houses, all constructed in the same period and of typical construction with timber floors and roof, supported off masonry walls. No.79 Guilford Street is a five-storey (including basement) terraced house with a single-storey rear extension at ground floor level. Beneath the footway at the front of the property there is a single vault which can be accessed via the front lightwell

The property is in a sound condition structurally. The adjoining properties are of similar construction and look to be in sound condition from an external non – intrusive visual examination.

No 79 shares party walls with No 78 (west side) and No 80 (east side).

The depth of the foundations of the existing building has been confirmed through trial pit excavation and found to be approximately 0.45m below cellar floor level.

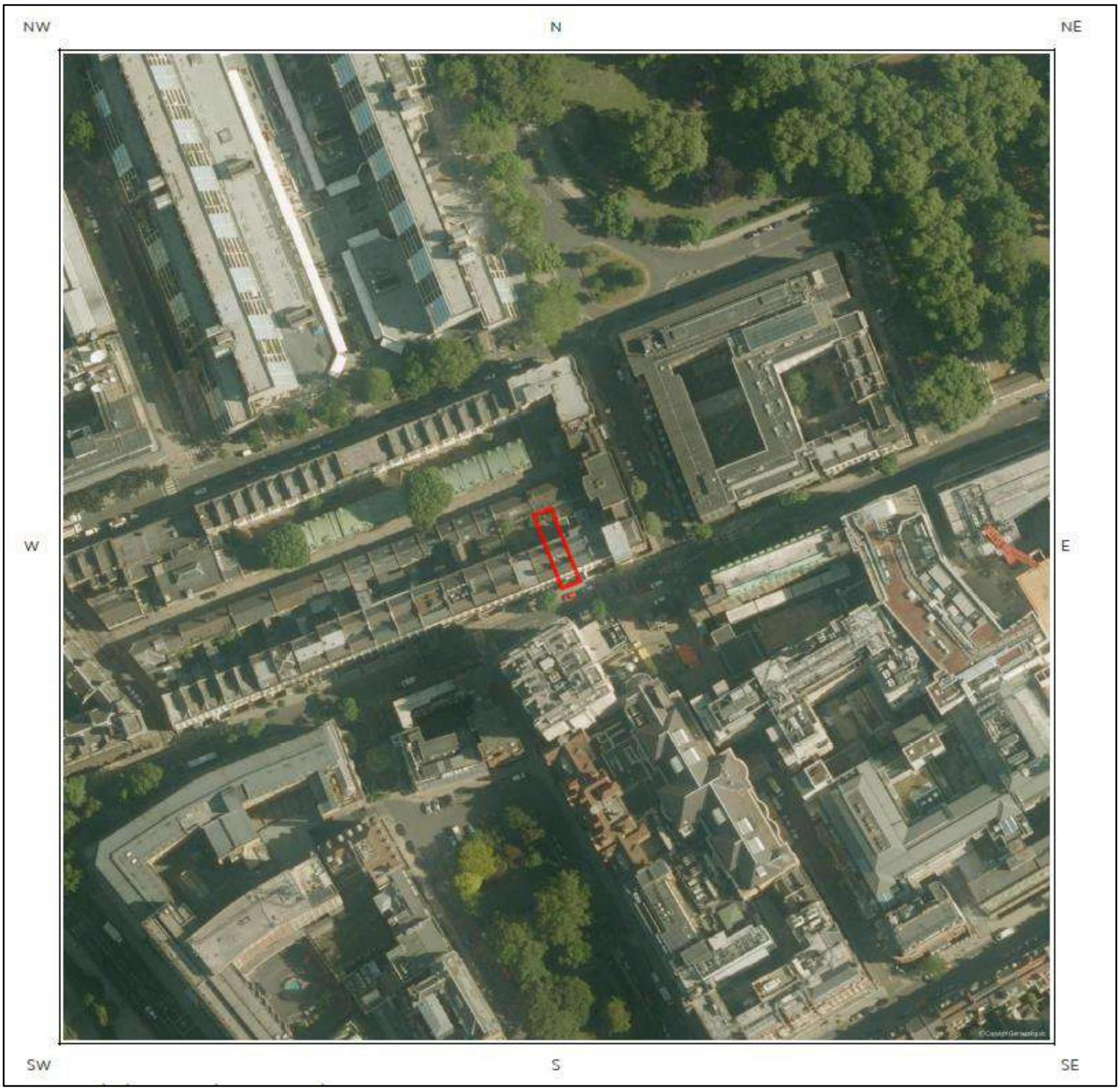

Figure 1. 79 Guilford Street

6. GEOLOGY AND HYDROLOGY CONDITIONS

The British Geological Survey website indicates that the site is underlain by the Lynch Hill Gravel Member over London Clay Formation. This has been confirmed by the site investigation carried out which has confirmed the ground conditions to comprise of firm to brownish grey to grey clay immediately below the sand and gravel layer found at the surface.

A copy of the site-specific boreholes carried by GabrielGeo Consulting Investigations is included in the Appendices to this report.

The GabrielGeo BIA report covers the groundwater, surface water and slope stability issues more fully but the engineering related issues are summarised below.

Surface Water and Subterranean (Groundwater) Flow

The site Investigation did encounter ground water in boreholes at 2.50m depth.

Perched groundwater would typically be expected in any overlying Made Ground, and possibly also in any Head Deposits which may be present, in at least the winter and early spring seasons.

Trial pit 6 of GabrielGeo report notes that some perched water was found at the bottom of the trial pits.

The Environmental Agency (EA) modelling indicates that the risk of flooding from surface water at 79 Guilford Street is 'Very Low' (which is the lowest, national background level of risk).

An uplift pressure is to be allowed for in the basement slab design. Moreover, to allow for potential burst water mains the retaining wall design will include ground water pressure to the existing cellar level.

Slope and Ground Stability

GabrielGeo's BIA report covers the geotechnical aspects of slope and ground stability and report no issues as the slope of the site is less than 7deg.

The temporary condition during the works is dealt with in the temporary works section below.

7. STRUCTURAL CALCULATIONS

GSE have carried out an outline structural design for the new basement to confirm the feasibility and buildability of the scheme.

The new retaining walls are designed as cantilever walls to reduce the amount of propping required during construction.

On the party wall line the retaining wall sections only require a bottom prop to maintain stability against sliding as the weight of the wall above resists the overturning.

The lightwell retaining walls will require temporary propping at high level and low level. At high level this will provided during construction by Multiprops propped off the central berm and in the permanent condition by a reinforced concrete wall forming a box section with opening on one side tying all walls together.

See calculation sheets in Appendix B for the retaining wall calculations prepared as part of this report. Retaining wall design has been prepared using the following London Clay parameters:

Angle of internal friction $\varphi' = 21$ deg
Unit weight $\gamma' = 20$ kN/m^3
Cohesion - ignore

The detailed design of the new basement will be undertaken at the start of works, once the house is unoccupied, to confirm that the existing structural arrangement are as allowed for in the outline design.

These operations will be carried out as part of the normal design process once planning has been obtained and will be submitted for checking by Building Control or an Approved Inspector.

8. STRUCTURAL DRAWINGS

The following structural drawings for the proposed basement are included in Appendix A and E:

J001413 – GA/01	General arrangement plan of the proposed basement indicating the proposed construction method using a 'hit and miss' sequence.
J001413 – S/01 and S/02	Typical section details through the party wall and external wall.
J001413 – MS/01 to MS/04	Construction sequence for typical underpinning section

9. CONSTRUCTION METHOD AND TEMPORARY WORKS REQUIREMENTS

GSE have considered the outline temporary works design to confirm the feasibility of the proposed basement construction.

As normal on projects of this type, where basements are constructed under an existing property, the method used will be an underpinning approach, with the individual underpins constructed in sections no wider than 1000 mm, sequenced such that no adjacent underpins are constructed within a 48-hour period.

This method of construction mitigates the potential ground movement and so minimizes any effects of settlement on the adjacent structures. Refer to appendix E - GSE Structural drawings for proposed basement, MS/01 and MS/02

10. CONSTRUCTION SEQUENCE OF THE NEW BASEMENT

1. Install basement/cellar floor waling frame and propping as required above the existing cellar floor slab, providing fixing details to the existing masonry wall
2. The cellar floor will be broken out locally and removed from site.
3. Batter back and reduce dig across the site and blind with an oversite concrete.
4. A conveyor belt will be set up through the front room of the existing lightwell to convey the spoil from the excavation to the skip placed at the front of the property for disposal. The conveying will be done using a method that does not impair the safety of pedestrians.
5. Underpin front elevation and cast new lightwell installing strip footing and vertical stem in a hit and miss underpinning sequence as per J001413 Basement Layout GA/01.
6. Once complete underpinning construction to party walls may commence.
7. Underpin the rear elevation and construct rear lightwell and prop off berm.

Note: local needle and propping to wall may be required due to openings and will comprise 152UC needles and Multiprops based on temporary footings.

Note: Reinforced concrete retaining walls will be formed as follows:

- ~ Excavate locally and shore excavation as required, installing sacrificial back board to external face. Excavated face to be propped off central berm behind.
- ~ Fix reinforcement to base and cast.
- ~ Fix reinforcement to wall and cast.
- ~ Dry pack between top of underpin and existing foundation
- ~ Re-prop wall off berm.

8. Form local excavation to install pad foundations for the columns
9. Lower berm level and install propping to low level of new underpins.
10. As excavation progresses, any existing foundations discovered will be broken out and removed from site to make way for the new basement construction.
11. Where new columns are required to support structure above, needle and prop wall over, cast new base and install column ground floor steelwork.
12. Both high- and low-level propping will be required to the rear lightwell underpins.

    Initially this will comprise propping off the central berm with Multi-props at high level and RMD Slim-shores (or similar designed by appointed Temporary Works Engineer) at low level. The high-level props can be removed once the permanent reinforced concrete ring beam is cast to tie the top of the retaining wall sections together.

    At low level the propping is to remain in place until the basement slab is cast.
13. Reduce the depth of the berm to formation level and cast basement slab.
14. After the new basement slab has cured, a drained – cavity layer will be laid to the slab and walls.
15. A layer of insulation will be placed on top of the drained – cavity layer on the slab, and in front of the drained – cavity layer on the walls.
16. Finally, a layer of screed will be laid to form the finished basement floor.

11. CONSTRUCTION SEQUENCE OF FRONT VAULT

1. Install propping above the existing lower ground floor slab. Once installed, break out existing lower ground floor slab and remove all debris from site
2. Complete mass concrete underpin section in areas shown in a hit and miss sequence.
3. Once, 1 and 2 are complete, form underpin section to chamber in a hit and miss sequence starting with section along the party wall. Ensure propping is in place until all construction is complete
4. Cast new ground bearing slab. *NOTE: propping to underpin section can be removed once all underpin section are complete.*

12. POTENTIAL IMPACT ON 78 AND 80 GUILFORD STREET AND ADJOINING PROPERTIES

The construction of the new deeper basement to 79 Guilford Street will affect No 79 and No 80 with which it shares a party walls. The zone of influence of the excavations will extend some distance but as set out in the GabrielGeo BIA report but the impact will be negligible.

The adjoining properties of No.80 and No.78 Guilford Street have very similar layouts to No.79, so both have existing single-storey basements. The footprint of No.80's basement extends beyond the proposed rear extension of No.79 on the 79/80 party wall, and there are no internal walls in No.80 alongside the proposed basement. The rear projection of No.78's basement is along the No.77/78 party wall, so there is no structure adjoining No.79's rear basement extension on the 78/79 boundary, other than the garden boundary wall (which should be separated structurally from the main wall of the house by insertion of a movement joint)

During trial pit investigation no evidence of underpinning of the 78/80 party wall was found, and it is assumed that the floor level in that basement is unlikely to be deeper than 2.70-3.05m below the ground floor level.

The damage assessment for the rear party walls, defined in geotechnical report by GabrielGeo Consulting, is within category 1 and therefore the impact will be minimal provided a suitably experienced contractor is appointed and a designed temporary works methodology is developed and followed on site.

BRE Digest 251 'Assessment of damage in low-rise buildings' describes category 0 to be Fine cracks which can be treated easily using normal decoration. Damage generally restricted to internal wall finishes; cracks rarely visible in external brickwork. Typical crack widths up to 1 mm.'.

The critical stage of the works in relation to the effect on the neighbouring properties will be during the construction of the basement. The major risk of movement during this stage of the works can be reduced and controlled by the appointment of a contractor with previous experience of basement construction that follows the agreed method of working incorporating all necessary temporary works.

The contractor will be required to produce traffic management, detailed method statements and provide detailed temporary works proposals for approval prior to the start of any works.

The temporary works, in accordance with the outline temporary works intent, as described above, will maintain the stability of the new basement during the construction and prevent rotation or slipping of the retaining walls during this stage of the works.

One of the major sources of movement in basement construction is differential settlement of the new foundations when bearing onto different geological strata. The site investigations carried out reveals that the underlying ground strata comprises London Clay to depth, and any movement of the existing walls during the works will be governed by any settlement which occurs during the construction of the proposed underpinning.

The new RC retaining wall will be designed as free-standing cantilevered walled, ignoring propping from ground floor level.

The proximity of the proposed basement to the neighbouring properties means that Party Wall Agreements will be required, and the Schedule of Conditions undertaken in this process will allow any inherent defects in the existing structures to be assessed and accommodated in the detailed design stage.

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

13. REDUCTION OF NOISE, DUST AND VIBRATION IMPACT ON NEIGHBOURING OCCUPIERS

The main environmental impacts are noise, vibration and dust. Contractors will always be expected to have considered noise and dust impacts related to their operations and to use Best Practicable Means (BPM) to minimize them, e.g. adjust working times, consider use of quieter methods.

The appointed contractor will be a member of the Considerate Contactors scheme.

The appointed contractor will comply with the following standards and practices.

- British Standard BS 5228 (noise and vibration control on construction and open sites).
- BS 6472:2008 (guide to evaluation of human exposure to vibration in buildings).
- Mayor's guidance on 'The control of dust and emissions during construction and demolition'.
- Principles set out within Section 61 of the Control of Pollution Act 1974.

Liaison with neighbors likely to be affected by works is an essential element of BPM and will be undertaken. The contractors will be expected to respond to complaints and resolve where practicable.

Impact on neighbors from vehicle movement will be addressed in the attached traffic management plan.

As residents are likely to be disturbed by noise, the permitted times of operation, including ancillary activities such as deliveries, will be restricted to standard hours:

- 8am – 6.00pm (Monday to Friday);
- 8am - 1pm (Saturday);
- No working is permitted on Sundays, bank holidays or other national holidays.

The appointed contractors will employ quiet working methods and noise generating equipment where practicable. Plant and activities to be employed should be reviewed to ensure that they are the quietest available for the required purpose e.g. 'super silenced' compressors. Work and sound reducing equipment should be regularly maintained to minimise noise emissions.

The contractors will make use of acoustic barriers or enclosures where there is likely to be significant disturbance to residents (subject to safety considerations).

The contractor's management team will employ the following actions to minimise the impact of noise, dust and vibration on the neighbours;

- All site operatives should be briefed and trained in the correct use of equipment and BPM measures in order to minimise noise impacts.
- Site surveys should take place to identify potential problems and facilitate work scheduling, the need for noise control measures, working hours and minimal delay and noise / dust impacts.

- Effective arrangements for the timely communication of site-specific noise control measures to site teams should be in place.

To reduce air pollution the appointed contractors will be expected to employ the methods listed below.

- Ensuring that fumes and/or dust do not escape from the site to affect members of the public and the surrounding environment;
- Burning of materials on site is not permitted under any circumstances;
- Dusty activity should be undertaken away from sensitive receptors, with wind direction taken into consideration;
- The site should be regularly inspected for spillages of cement and other powders which should be removed to prevent off-site deposition;
- Dusty material and activities should be dampened down in dry weather. The use of groundwater should be investigated, and water should be reused wherever possible.
- Rubber chutes should be used and drop heights minimised;
- Off-site fabrication, or cutting to size, shall be employed to avoid cutting materials on site whenever possible; and
- Careful consideration should be given to the location and temperature control of tar and asphalt burners.

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

Any local services on the property's land will be maintained during construction and rerouted if necessary. The exact location of these services will not be known until the works commence. However, the impact will be negligible as these services will be maintained. If it is necessary to relocate or divert any utilities, the Contractor and Design Team will be under a statutory obligation to notify the utility owner prior to any works. This will be so that they can assess the impact of the works and grant or refuse their approval.

The method of constructing the front retaining wall, along with the presence of the front garden area means that services in the street should not be affected by these works.

15. POTENTIAL IMPACT ON DRAINAGE, SEWAGE, SURFACE AND GROUND WATER LEVELS AND FLOWS INCLUDING SUDS

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

The proposed works will not alter the current state of the property, which will remain as part of a single residence; therefore, there will be no significant change in discharge to the existing drainage and sewage systems and there will be little or no impact on the foul drainage.

Surface water will not be altered as the proposed works are underground and there will be no additional proposed 'hard surfaces' formed at ground level.

Ground water will not be greatly altered as the proposed works will be carried out in London Clay, which itself is a highly impermeable material, and the proposed lightwell extensions are within generally within existing hardstanding resulting in negligible change to the property's 'hard surfaces'.

The geotechnical investigations and research carried out confirm that the new formation will be into London Clay and ground water is not expected to be an issue.

16. POTENTIAL IMPACT ON EXISTING AND PROPOSED TREES

No existing trees will be felled during the construction of the proposed works and no trees are affected by the proposed works nor are any trees protected by Tree Preservation Orders in the vicinity of the proposed works that will be damaged by the construction works.

With the above considerations, the impact on existing and proposed trees is negligible.

Prepared By:

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MSc

Checked By:

Paul Bennett

B.Sc. (Hons), C.Eng., MICE

Green Structural Engineering Ltd

April 2019

APPENDICES

The following appendices are included with this report:

Appendix A - GSE Structural drawings for proposed basement

Appendix B - GSE Calculation sheets for design of basement retaining walls

Appendix C - GSE Underpinning Specification

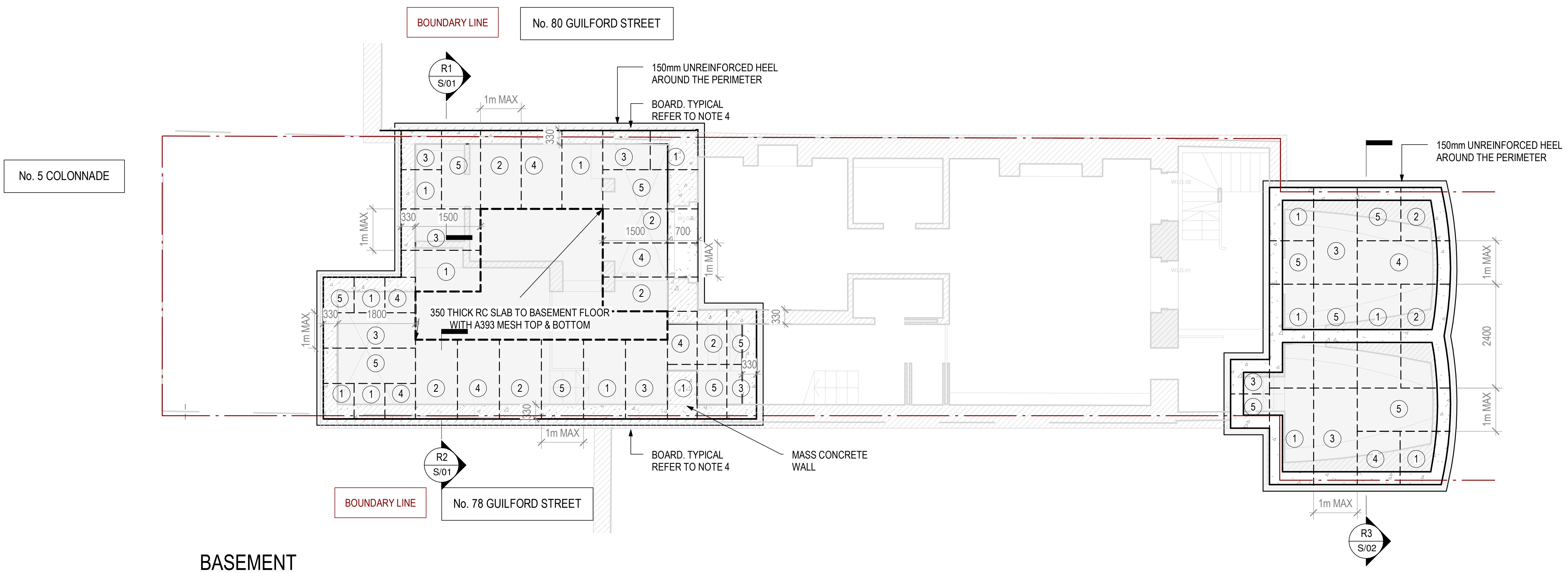
Appendix D - Site Investigation Report (extract)

Appendix E - Temporary works sequence`

APPENDIX A

GSE STRUCTURAL DRAWINGS FOR PROPOSED BASEMENT

Unit 5, Quayside, Sir William Morris Way, Fulham SW6 2UZ
Registered in England Number 06533911



BASEMENT
1 : 100

KEY:	
	EXISTING STRUCTURE
	NEW LOAD BEARING CONCRETE
	NEW LOAD BEARING BRICKWORK (20N/mm²)
	NEW LOAD BEARING BLOCKWORK (7N/mm²)
	NEW LOAD BEARING STUD PARTITION WALL
	NEW NON-LOAD BEARING STUD WALL
— — —	AREA TO BE UNDERPINNED

NOTES:
1. CATNIC STRONGHOLD SWC STAINLESS STEEL WALL STARTER KITS. POSITION AT JUNCTIONS OF EXISTING MASONRY WALL AND NEW MASONRY. INSTALL IN ACCORDANCE WITH THE MANUFACTURERS SPECIFICATION. FULLY EMBED TIES IN MORTAR JOINTS.
2. BELOW GROUND WATERPROOFING AND DRAINAGE BY OTHERS.
3. UNDERPINS WILL NOT BE STABLE WHILST UNDER CONSTRUCTION. CONTRACTOR MUST PROVIDE ADEQUATE LATERAL SUPPORT TO ALL PINS UNTIL BASEMENT SLAB HAS BEEN CAST.
4. NON COMPRESSIBLE WATER RESISTANT CEMENTITIOUS BOARD LINER TO BACK OF ALL UNDERPIN SUPPORTING PARTY WALLS.
5. RAISE RETAINING WALL LOCALLY (440 LONG x 100 WIDE) TO SUPPORT NEW STEEL WORK.

J001413

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79 GUILFORD STREET

GREEN STRUCTURAL ENGINEERING
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William Morris Way,
Fulham, SW6 2UZ
020 3405 3120

BASEMENT GA

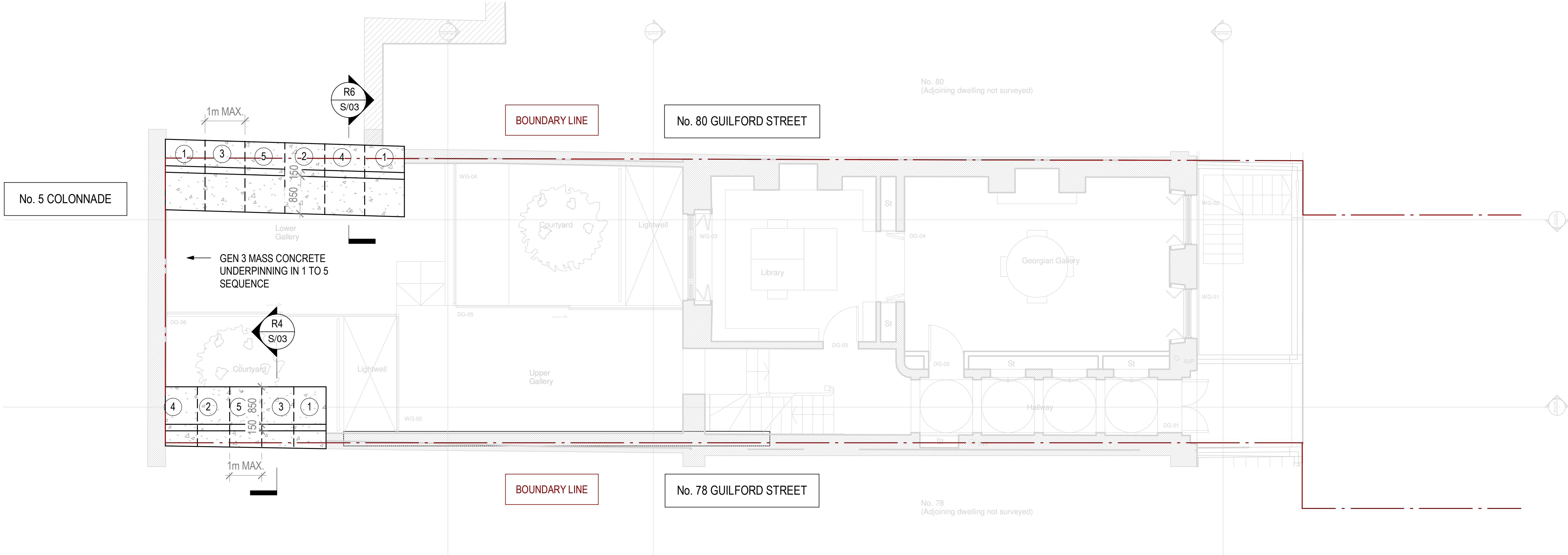
PRELIMINARY

DRAWN	CHECKED	DATE	PAPER SIZE	SCALE	DRAWING NO.	REV.
J.W	M.E	03.04.2019	A3	As indicated	GA/01	P1

KEY:	
	EXISTING STRUCTURE
	NEW LOAD BEARING CONCRETE
	NEW LOAD BEARING BRICKWORK (20N/mm²)
	NEW LOAD BEARING BLOCKWORK (7N/mm²)
	NEW LOAD BEARING STUD PARTITION WALL
	NEW NON-LOAD BEARING STUD WALL
	STRAPS - SEE NOTE 2 ABOVE
	END PLATE TO BEAM

NOTES:

1. CATNIC STRONGHOLD SWC STAINLESS STEEL WALL STARTER KITS. POSITION AT JUNCTIONS OF EXISTING MASONRY WALL AND NEW MASONRY. INSTALL IN ACCORDANCE WITH THE MANUFACTURERS SPECIFICATION. FULLY EMBED TIES IN MORTAR JOINTS.
2. RECESS ALL TIMBER JOISTS & STRAP TO BEAM USING 30x5x900 LONG GALVANIZED STRAPS @ 1200 CRS MAX. SCREW FIX TO JOISTS IN ACCORDANCE WITH MANUFACTURERS SPECIFICATION.
3. DO NOT BEAR NEW STEELWORK INTO EXISTING CHIMNEY FLUES. STRUCTURAL ENGINEER TO BE INFORMED IF THIS ISSUE IS ENCOUNTERED ON SITE. CCTV SURVEY TO BE UNDERTAKEN TO CONFIRM POSITION OF EXISTING FLUES AND ALTERNATIVE SOLUTION TO BE AGREED.



GROUND FLOOR

1 : 100

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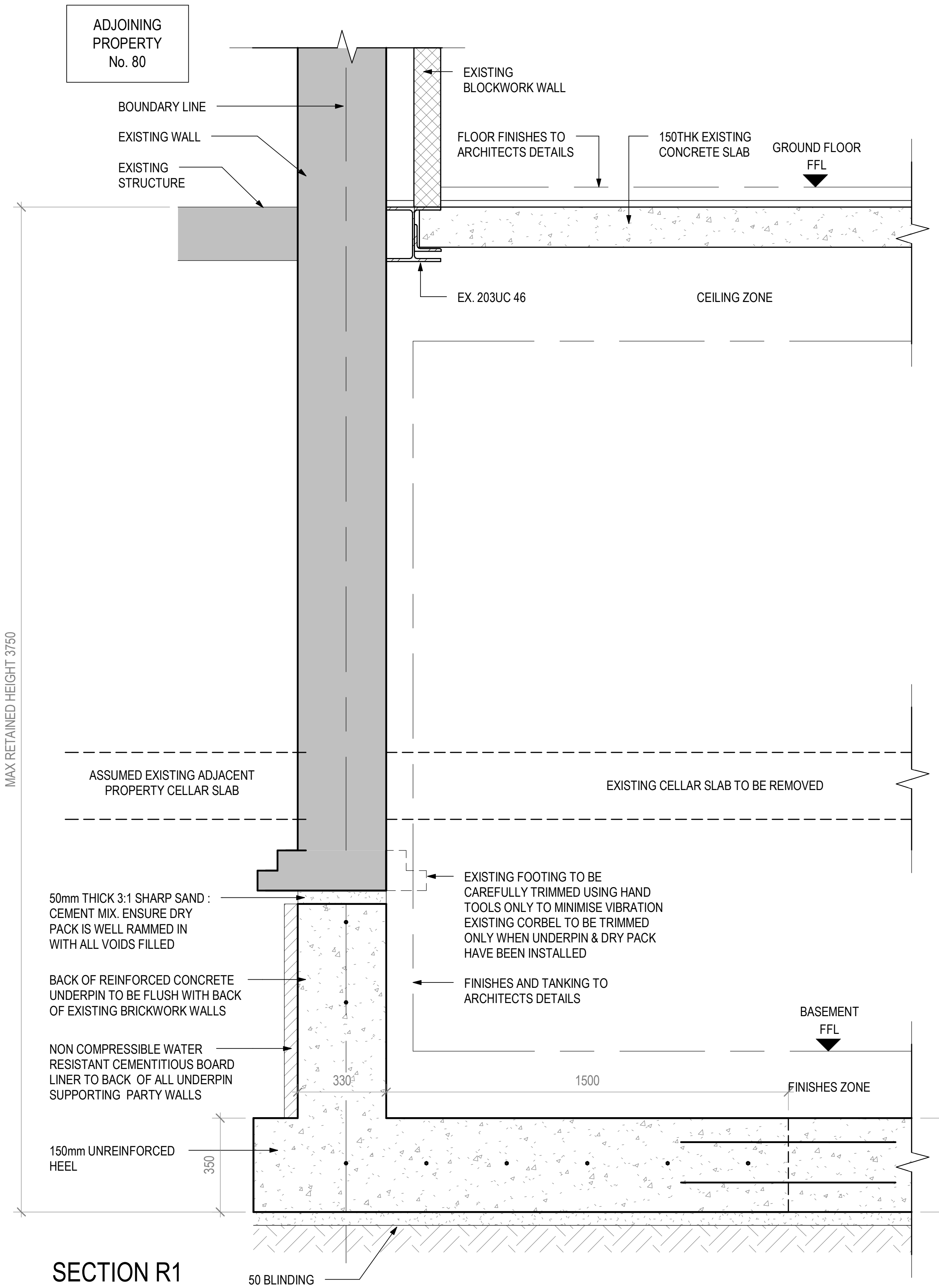

SECTION R1
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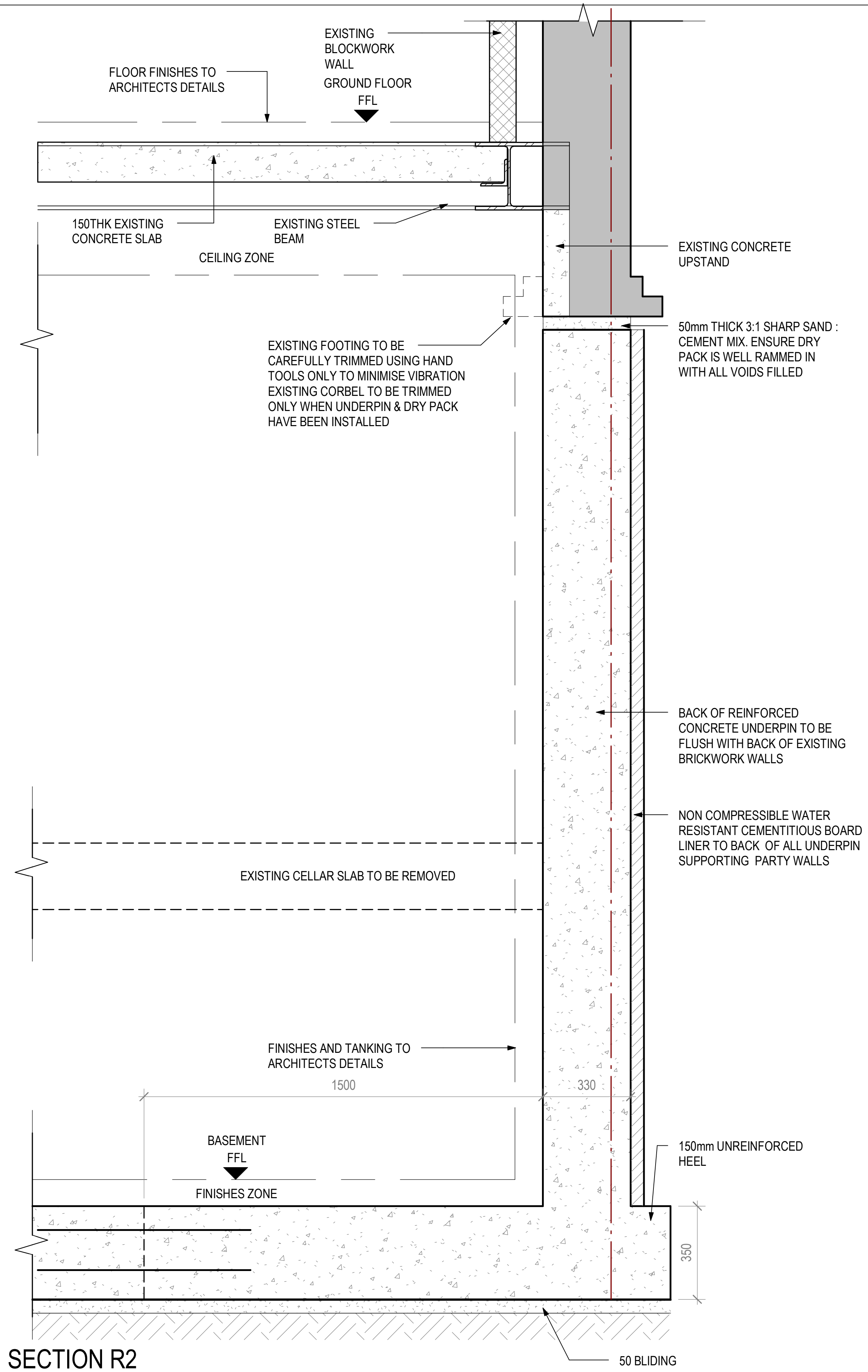

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SETTING OUT TO BE CONFIRMED ON SITE.

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SECTION SHEET 1

PRELIMINARY

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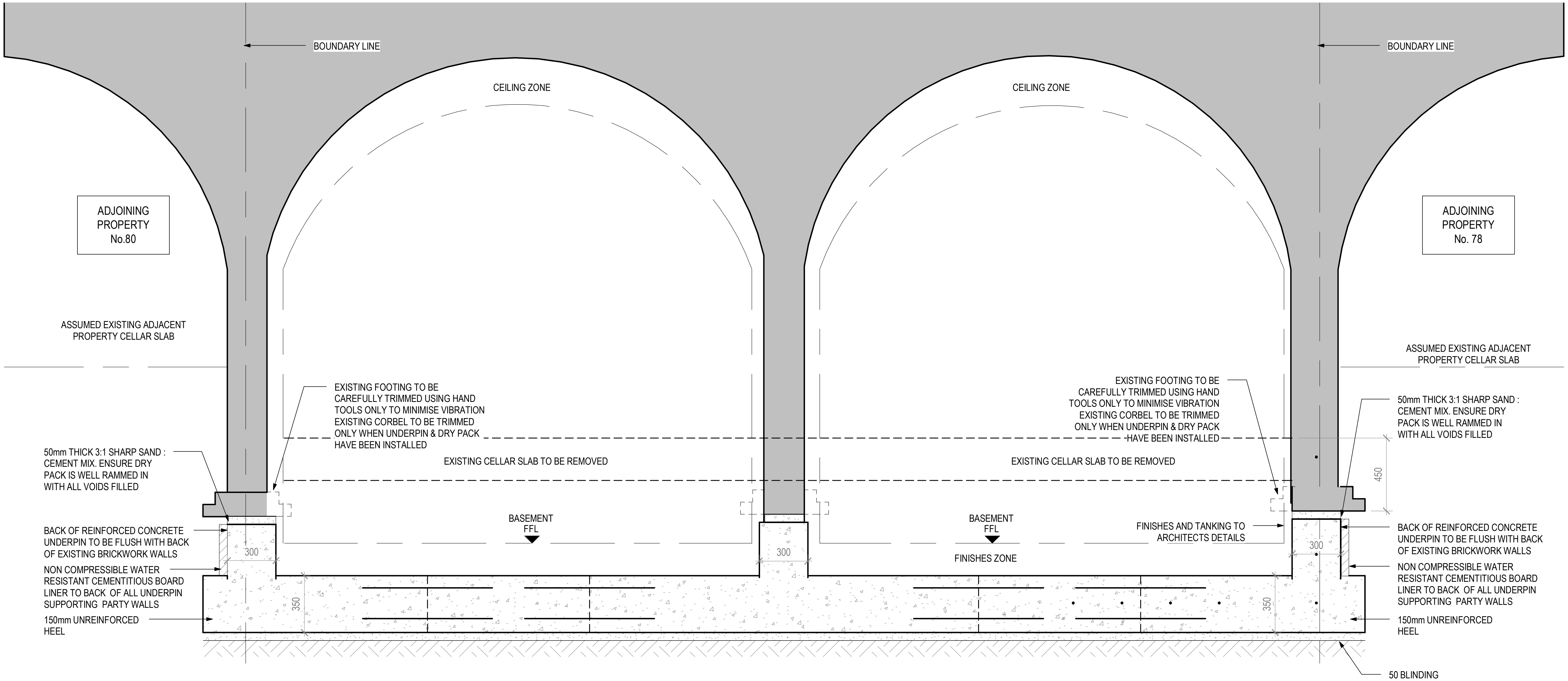

SECTION R3

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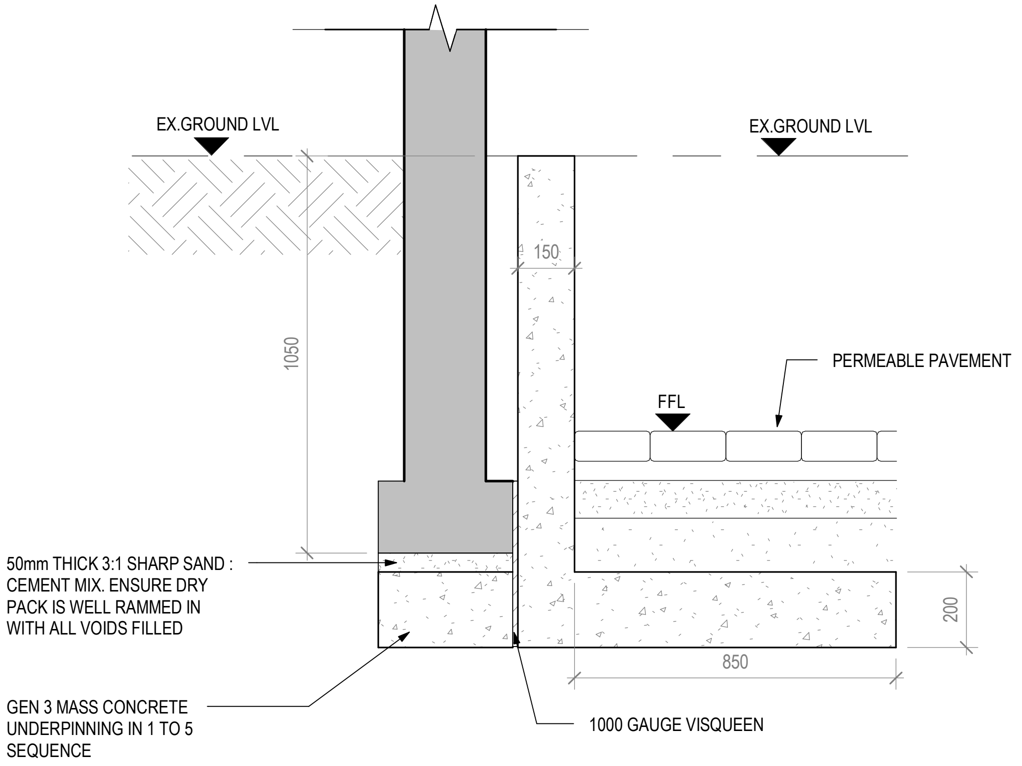

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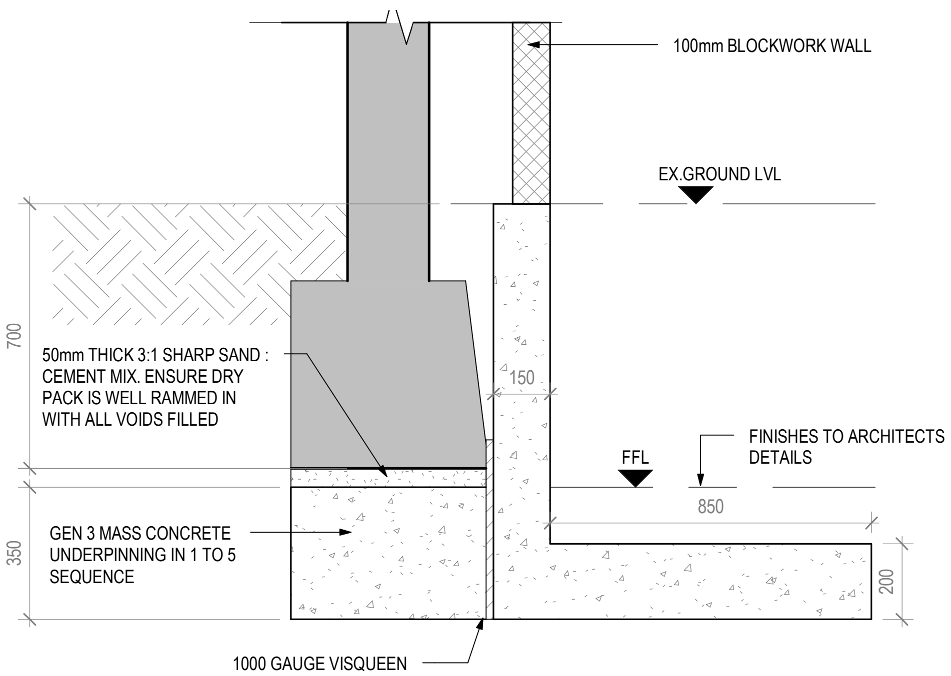

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APPENDIX B

GSE CALCULATIONS FOR DESIGN OF BASEMENT RETAINING WALLS

Unit 5, Quayside, Sir William Morris Way, Fulham SW6 2UZ
Registered in England Number 06533911

	Project		Job Ref	
GSE GREEN STRUCTURAL ENGINEERING	79 Guilford Street		J001413	
	Drawing Ref	Calculations by	Checked by	Sheet No
		V. Ferraiuolo	M. Egan	
	Part of Structure		Date	
	Basement design		March 2019	

CALCULATE VERTICAL LOAD ON VAULT WALL
• CASE 1 --- MIDDLE WALL
WEIGH OF SOIL OVER VAULT

→ SURCHARGE $20 KN/m^2$ (LIVE LOAD)

→ SOIL $20 KN/m^3$

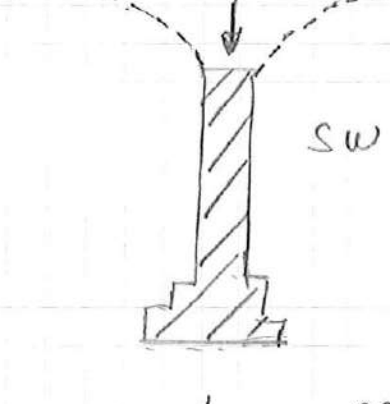
$H_{ave} = (1,54 + 0,48)/2 = 1,01 m$

→ BRICKWORK VAULT THK = 330mm → $7,40 KN/m^2$

$DL = 27,60 KN/m^2$
$LL = 20 KN/m^2$

span load = 3,20 m → $DC = 88,32 KN/m$; $LL = 20 KN/m$

DC = 88,32 KN/m
LC = 20,00 KN/m



SW BRICKWALL

THK = 330mm + PL
DL = $7,40 KN/m^2$ | DC = 11,12 KN/m
H_{AVE} = 1,50 m

TOTAL
DC 99,44 KN/m
LL = 20,00 KN/m

	Project		Job Ref	
GSE GREEN STRUCTURAL ENGINEERING	79 Guilford Street		J001413	
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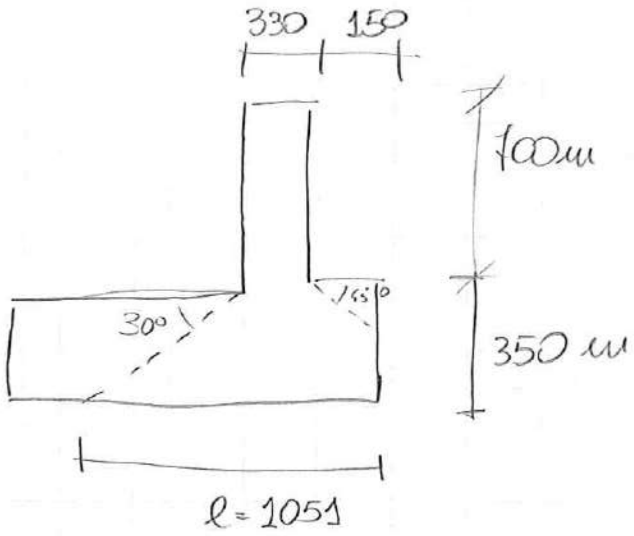
	Part of Structure		Date	
	Basement design		March 2019	

CASE 2 - - - FRONT WALL

THK WALL = 440 mm → 8,40 KN/m²

H = 1,60 m

DC = 13,44 KN/m



F = 99,44 + 20 = 119,44 KN

P = 119,44 / 1,05 = 113,75 KN/m²

Allowable bearing pressure = 130 KN/m²
(MEDIUM DENSE N = 13)

Verified

	Project		Job Ref	
GSE Green Structural Engineering	79 Guilford Street		J001413	
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5 COLONNADE LOAD TAKE DOWN

ROOF

$g_k = 1.0 kN/m^2$

$q_k = 0.6$

CEILING

$g_k = 0.6 kN/m^2$

$q_k = 0.5 kN/m^2$

FLOORS =

$g_k = 1.0 kN/m^2$, $q_k = 2.5 kN/m^2$

(ASSUMED TIMBER FLOORS)

ASSUME G.F. SUSPENDED

WALL

ASSUMED 215 THICK 7.0m HIGH ABOVE GROUND

$g_k = 5.3 kN/m^2$

BRICKWORK LOAD = $[1.0 + 0.6 + 0.6 + 0.5 + ((1.0 + 2.5) \times 2)] \times 3] + (8 \times 5.3)$

$7 + 1.2$

$= 73 kN/m$

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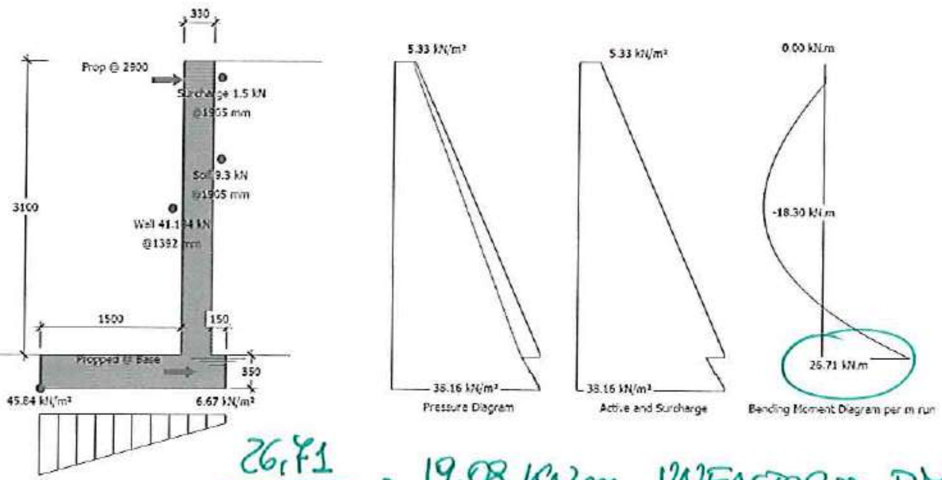
Job Ref	: J001413
Sheet	: URW2 - Temporary condi /
Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
Checked	: M. Egan
Approved	:

MasterKey : Retaining Wall Design to BS 8002 and BS 8110 : 1997
Load Case 2 - Temporary Case
Reinforced Concrete Retaining Wall with Reinforced Base



$\frac{26,71}{1,40} = 19,08$ KNm UNFACTORED BY

Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Dry Soil 20.00, Saturated Soil 22.00, Submerged Soil 12.00, Concrete 24.00
Concrete grade	fcu 35 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 460 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Water table level 0 mm
Unplanned excavation depth	Front of wall 345 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Additional Wall Prop	Prop @ 2.9 m
† Dimensions	All props are measured from the top of the base
	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Soil bearing pressure	Allowable pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	90.759/134.382	0.675	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(18.758+0.000)	0.000	OK
Prop Reactions Case 2 (Service)	53.8 kN @ Base, 17.4 kN @ 3.250 m		

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Approved :

Soil Pressure

Virtual Back (No uplift)	Max(38.256/100, 14.253/100) kN/m²	0.383	OK
Wall Back (No uplift)	Max(45.837/100, 6.672/100) kN/m²	0.458	OK

Structural Design

At Rest Earth Pressure

At rest earth pressures magnification	$(1+\mathrm{Sin}(\phi))\ \mathrm{x}\sqrt{\mathrm{OCR}} = (1+\mathrm{Sin}(17.74)\mathrm{x}\sqrt{1}$	1.3

Prop Reactions

Maximum Prop Reactions (Ultimate)	78.3 kN @ Base, 26.2 kN @ 2.900 m

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	OK
Compression Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 449 mm², 460 N/mm², 35.0 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	449 mm², 35 mm, 14 mm, 0.05	55.0 kN.m	
Moment Capacity Check (M/Mr)	M 26.7 kN.m, Mr 55.0 kN.m	0.486	OK
Shear Capacity Check	F 62.1 kN, vc 0.407 N/mm², Fvr 120.1 kN	0.52	OK

Wall Design (Outer Steel)

Critical Section	Critical @ 1605 mm from base, Case 2		
Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	OK
Compression Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 449 mm², 460 N/mm², 35.0 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	449 mm², 35 mm, 14 mm, 0.05	55.0 kN.m	
Moment Capacity Check (M/Mr)	M 18.3 kN.m, Mr 55.0 kN.m	0.333	OK
Shear Capacity Check	F 0.2 kN, vc 0.407 N/mm², Fvr 120.1 kN	0.00	OK

Base Top Steel Design

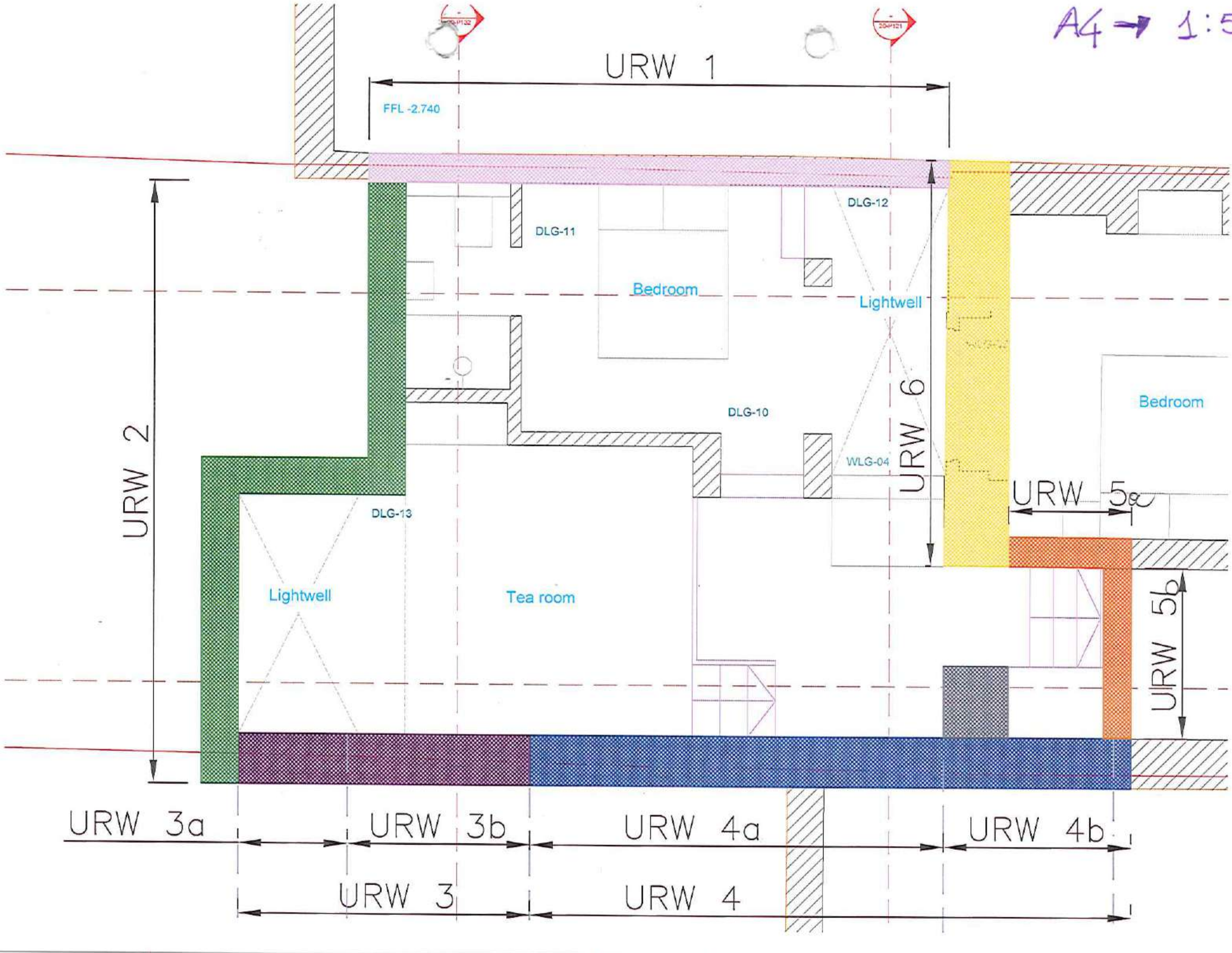
Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 1.1 kN.m, Mr 64.1 kN.m	0.017	OK
Shear Capacity Check	F 14.6 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.12	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 40.1 kN.m, Mr 64.1 kN.m	0.626	OK
Shear Capacity Check	F 45.5 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.36	OK

DESIGN AS RAFT

A4 → 1:50


URW 1
FFL -2.740
DLG-11
Bedroom
DLG-12
Lightwell
URW 6
DLG-10
WLG-04
Bedroom
URW 5a
URW 5b
URW 2
DLG-13
Lightwell
Tea room
URW 3a
URW 3b
URW 4a
URW 4b
URW 3
URW 4

	Project		Job Ref	
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	Part of Structure		Date	
	Basement design		March 2019	

UNDERPINNING RETAINING WALL 1 H= 1,60m

LOAD BRICKWALL

H= 3,00 m
215mm +pl = 5,30 KN/m²

DC = 15,90 KN/m

H = 3,00 m
330mm+pl = 7,40 KN/m²

DC = 22,20 KN/m

NO. 80 property

ROOF TERRACE DC: 1,00 KN/m²
CC: 1,50 KN/m²
span load: $\frac{2,80}{2}$ = 1,40m

DC = 1,40 KN/m
CC: 2,10 KN/m

LOWER GROUND FLOOR
DC = 0,80 KN/m²
CC = 1,50 KN/m²
span load: $\frac{2,80}{2}$ = 1,40 m

DC = 1,12 KN/m
CC = 2,10 KN/m

	Project		Job Ref	
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N° P01 EXTERNAL COURTYARD - STONE FLOOR

DL = 5,80 KN/m²
LL = 5,60 KN/m²
span load 3.4/2 = 1,70 m

DL = 9,86 KN/m
LL = 9,52 KN/m

TOTAL LOAD
DL = 58,40 KN/m
LL = 13,72 KN/m

length 6,27 m → spreader on 13,00 m

$$DL = \frac{58,40 \times 6,27}{13} = 28,17 \text{ KN/m}$$

$$LL = \frac{13,72 \times 6,27}{13} = 6,62 \text{ KN/m}$$

SW REINFORCEMENT STEM WALL (APPLIED @ BOTTOM)

H = 1,60 m
THK = 0,33 m
SW: 24 KN/m³

DL = 12,92 KN/m

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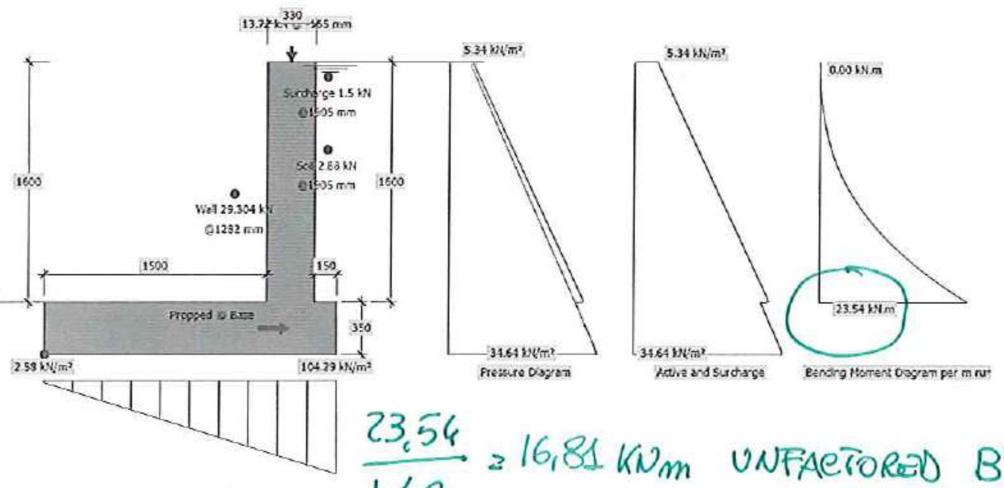
Job Ref	: J001413
Sheet	: RCW1 - Permanent condi /
Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
Checked	: M. Egan
Approved	:

MasterKey : Retaining Wall Design to BS 8002 and BS 8110 : 1997
Load Case 1 - Permanent Case
Reinforced Concrete Retaining Wall with Reinforced Base



$\frac{23,54}{1,40}$ = 16,81 KNm UNFACTORED BM

Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Back Soil - Dry 20.00, Saturated 22.00, Submerged 12.00
	Front Soil - Dry 18.00, Saturated 20.80, Submerged 10.80, Concrete 24.00
Concrete grade	fcu 35 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 460 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Water table level 1600 mm
Unplanned excavation depth	Front of wall 195 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Vertical Line Loads	58.40 kN/m @ X -165 mm and Y 0 mm - Load type Dead
	13.72 kN/m @ X -165 mm and Y 0 mm - Load type Live
† Dimensions	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Soil bearing pressure	Allowable pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(22)/1.2) = 18.61°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	28.014/165.988	0.169	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(38.179+0.375)	0.000	OK
Prop Reaction Case 2 (Service)	38.5 kN @ Base		

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Checked	: M. Egan
Approved	:

Soil Pressure

Virtual Back (No uplift)	Max(2.582/100, 104.290/100) kN/m²	1.043	**Warning**
Wall Back (No uplift)	Max(6.004/100, 100.869/100) kN/m²	1.009	**Warning**

Structural Design

At Rest Earth Pressure

At rest earth pressures magnification	$(1+\text{Sin}(\phi))\ x\sqrt{OCR} = (1+\text{Sin}(17.74)x\sqrt{1}$		1.3

Prop Reaction

Maximum Prop Reaction (Ultimate)	53.1 kN @ Base		

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	OK
Compression Steel Provided (Cover)	Main B10@300 (30 mm) Dist. B10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 449 mm², 460 N/mm², 35.0 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 14 mm, 0.05	55.0 kN.m	
Moment Capacity Check (M/Mr)	M 23.5 kN.m, Mr 55.0 kN.m	0.428	OK
Wall Axial Design (N/Ncap)	N 121.5 kN, Ncap 4620.0 kN	0.026	OK
Wall Slenderness λ	Leff/tk =2.00x1600.0/330.0	9.7	OK
Wall Axial-Mom Design (M/M_{rAxial})	M 23.5 kN, Mr_{Axial} 75.5 kN.m	0.312	OK
Shear Capacity Check	F 38.5 kN, vc 0.407 N/mm², Fvr 120.1 kN	0.32	OK

Base Top Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 0.1 kN.m, Mr 64.1 kN.m	0.002	OK
Shear Capacity Check	F 1.7 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.01	OK

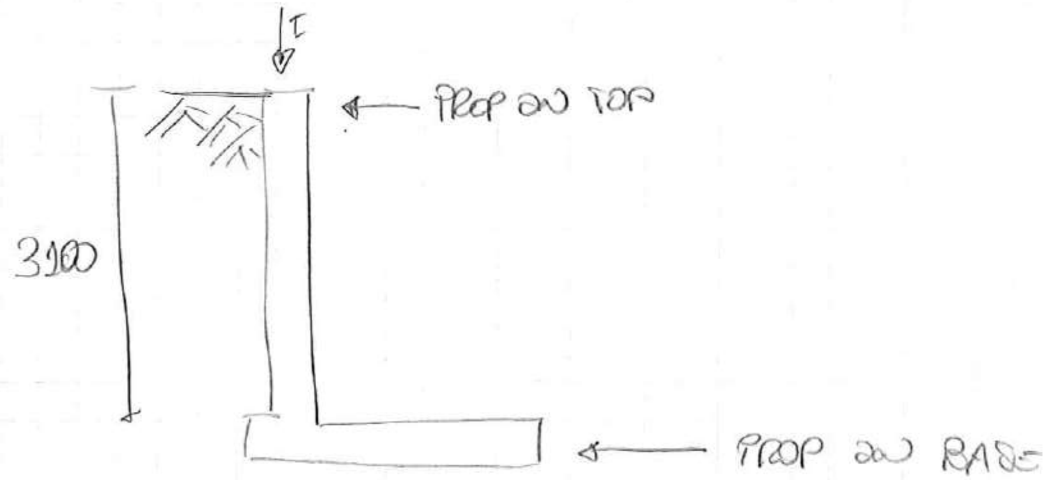
Base Bottom Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 31.4 kN.m, Mr 64.1 kN.m	0.490	OK
Shear Capacity Check	F 70.0 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.55	OK

⚠ DESIGN AS RAFT

GSE GREEN STRUCTURAL ENGINEERING	Project 79 Guilford Street		Job Ref J001413	
	Drawing Ref	Calculations by V. Ferraiuolo	Checked by M. Egan	Sheet No
	Part of Structure Basement design		Date March 2019	

UNDERPINNING RETAINING WALL 2 (and 3a)



F = ∅

SW STEP WALL (@ bottom level)

$\gamma = 24\ kN/m^3$

THK 330 mm

H = 3,10 m

DL = 24,55 kN/m

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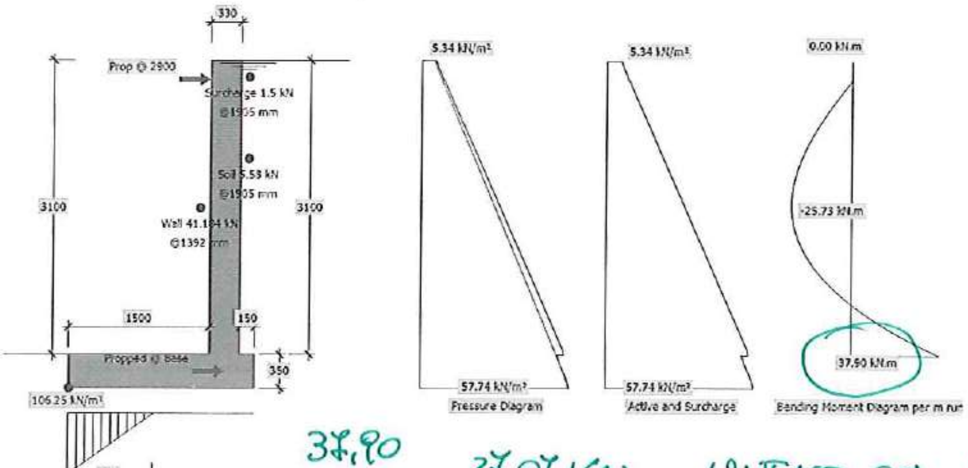
Checked	: M. Egan
Approved	:

MasterKey : Retaining Wall Design to BS 8002 and BS 8110 : 1997
Load Case 1 - Permanent Case
Reinforced Concrete Retaining Wall with Reinforced Base



$\frac{37.90}{1.40} = 27.07$ KNm UNFACTORED BM

Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Dry Soil 20.00, Saturated Soil 22.00, Submerged Soil 12.00, Concrete 24.00
Concrete grade	fcu 35 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 460 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Water table level 3100 mm
Unplanned excavation depth	Front of wall 345 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Additional Wall Prop	Prop @ 2.9 m
† Dimensions	All props are measured from the top of the base
	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Soil bearing pressure	Allowable pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	134.043/151.988	0.882	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(17.416+0.000)	0.000	OK
Prop Reactions Case 2 (Service)	83.4 kN @ Base, 25.0 kN @ 3.250 m		

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Approved	:

Soil Pressure

Virtual Back	86.537/100 kN/m², Length under pressure 1.115 m	0.865	OK
Wall Back	106.247/100 kN/m², Length under pressure 0.909 m	1.062	Warning
Note:	Length under pressure is less than 75% of the base width		Warning

Structural Design

At Rest Earth Pressure

At rest earth pressures magnification	$(1+\mathrm{Sin}(\phi)) \times \sqrt{OCR} = (1+\mathrm{Sin}(17.74) \times \sqrt{1}$		1.3

Prop Reactions

Maximum Prop Reactions (Ultimate)	114.0 kN @ Base, 35.1 kN @ 2.900 m

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	OK
Compression Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 449 mm², 460 N/mm², 35.0 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	449 mm², 35 mm, 14 mm, 0.05	55.0 kN.m	
Moment Capacity Check (M/Mr)	M 37.9 kN.m, Mr 55.0 kN.m	0.690	OK
Shear Capacity Check	F 89.1 kN, vc 0.407 N/mm², Fvr 120.1 kN	0.74	OK

Wall Design (Outer Steel)

Critical Section	Critical @ 1605 mm from base, Case 2		
Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	OK
Compression Steel Provided (Cover)	Main B10@175 (30 mm) Dist. B10@175 (40 mm)	449 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 449 mm², 460 N/mm², 35.0 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	449 mm², 35 mm, 14 mm, 0.05	55.0 kN.m	
Moment Capacity Check (M/Mr)	M 25.7 kN.m, Mr 55.0 kN.m	0.468	OK
Shear Capacity Check	F 0.9 kN, vc 0.407 N/mm², Fvr 120.1 kN	0.01	OK

Base Top Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 0.9 kN.m, Mr 64.1 kN.m	0.014	OK
Shear Capacity Check	F 12.0 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.09	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 46.6 kN.m, Mr 64.1 kN.m	0.727	OK
Shear Capacity Check	F 40.3 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.32	OK

⚠ DESIGN AS RAFT

	Project		Job Ref	
GSE GREEN STRUCTURAL ENGINEERING	79 Guilford Street		J001413	
	Drawing Ref	Calculations by	Checked by	Sheet No
		V. Ferraiuolo	M. Egan	
	Part of Structure		Date	
	Basement design		March 2019	

UNDERPINNING RETAINING WALL 3b H = 3100 mm

WALL → GARDEN WALL H=1,50
215 mm + pl = 5,30 KN/m²

| DC = 7,95 KN/m

0,10 m internal blockwork (600 kg/m³) = 0,6 KN/m²
H = 3,80

| DC = 2,28 KN/m

ROOF TERRACE

DC = 1,25 KN/m²
LL = 3,00 KN/m²
Span load = 2,60 m

| DC = 3,25 KN/m
| LL = 7,80 KN/m

UPPER GALLERY

DC = 5,70 KN/m²
LL = 5,60 KN/m²
Span load = 2,60 m

| DC = 14,82 KN/m
| LL = 14,56 KN/m

DL = 28,30 KN/m
LL = 22,36 KN/m

Spread the load from 1971 to 11323 mm

$$DC = \frac{28,30 \cdot 1,97}{11,32} = 4,93 \text{ KN/m}$$

$$LL = \frac{22,36 \cdot 1,97}{11,32} = 3,89 \text{ KN/m}$$

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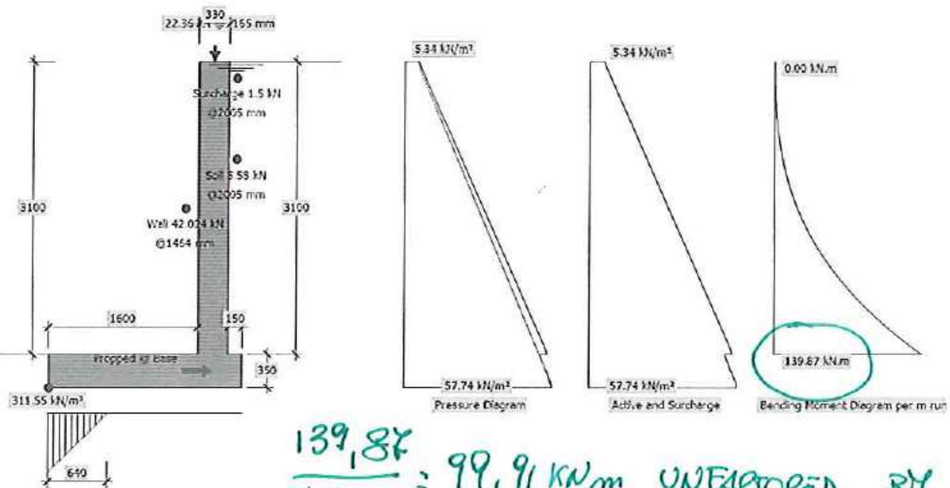
Job Ref	: J001413
Sheet	: URW3b - Permanent condi /
Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
Checked	: M. Egan
Approved	:

MasterKey : Retaining Wall Design to BS 8002 and BS 8110 : 1997
Load Case 1 - Permanent Case
Reinforced Concrete Retaining Wall with Reinforced Base



$\frac{139,87}{1,40}$ = 99,91 kNm UNFACTORED BM

Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Dry Soil 20.00, Saturated Soil 22.00, Submerged Soil 12.00, Concrete 24.00
Concrete grade	fcu 35 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 460 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Water table level 3100 mm
Unplanned excavation depth	Front of wall 345 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Vertical Line Loads	28.30 kN/m @ X -165 mm and Y 0 mm - Load type Dead
	22.36 kN/m @ X -165 mm and Y 0 mm - Load type Live
† Dimensions	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Soil bearing pressure	Allowable pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	134.043/165.115	0.812	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(35.999+0.000)	0.000	OK
Prop Reaction Case 2 (Service)	108.3 kN @ Base		

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Job Ref	: J001413
Sheet	: URW3b - Permanent condi /
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Checked	: M. Egan
Approved	:

Soil Pressure

Virtual Back	213.541/100 kN/m², Length under pressure 0.934 m	2.135	**Warning**
Wall Back	311.548/100 kN/m², Length under pressure 0.64 m	3.115	**Warning**
Note:	Length under pressure is less than 75% of the base width		**Warning**

Structural Design

At Rest Earth Pressure

At rest earth pressures magnification	$(1+\text{Sin}(\phi))\,x\sqrt{OCR} = (1+\text{Sin}(17.74)x\sqrt{1}$		1.3

Prop Reaction

Maximum Prop Reaction (Ultimate)	149.2 kN @ Base		

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main B16@150 (30 mm) Dist. B10@175 (46 mm)	1340 mm²	OK
Compression Steel Provided (Cover)	Main B10@300 (30 mm) Dist. B10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	292 mm, 1000 mm, 1340 mm², 460 N/mm², 35.0 N/mm²	273 mm	
Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 42 mm, 0.14	160.1 kN.m	
Moment Capacity Check (M/Mr)	M 139.9 kN.m, Mr 160.1 kN.m	0.874	OK
Shear Capacity Check	F 124.3 kN, vc 0.590 N/mm², Fvr 172.3 kN	0.72	OK

Base Top Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 0.9 kN.m, Mr 64.1 kN.m	0.014	OK
Shear Capacity Check	F 12.0 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.09	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 15.5 kN.m, Mr 64.1 kN.m	0.242	OK
Shear Capacity Check	F 16.2 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.13	OK

! DESIGN AS RAFT

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	Part of Structure		Date	
	Basement design		March 2019	

UNDERPINNING RETAINING WALL 4a

H = 3,10m

WALL

330mm + pl = 8,40 KN/m² | DL = 22,20 KN/m
H = 3,00

215 mm + pl = 5,30 KN/m² | DL = 15,90 KN/m
H = 3,00 m =

100 m light weight BLOCKWORK = 0,60 KN/m² | DL = 2,28 KN/m
H = 3,80m

ROOF TERRACE

DL : 1,25 KN/m² | DL = 3,25 KN/m
LL = 3,00 KN/m² | LL = 7,80 KN/m
span load = 2,60m

UPPER GALLERY

DL = 5,70 KN/m² | DL = 14,82 KN/m
LL = 5,60 KN/m² | LL = 14,56 KN/m
span load = 2,60 m

TOTAL LOAD | DL = 58,45 KN/m
| LL = 22,36 KN

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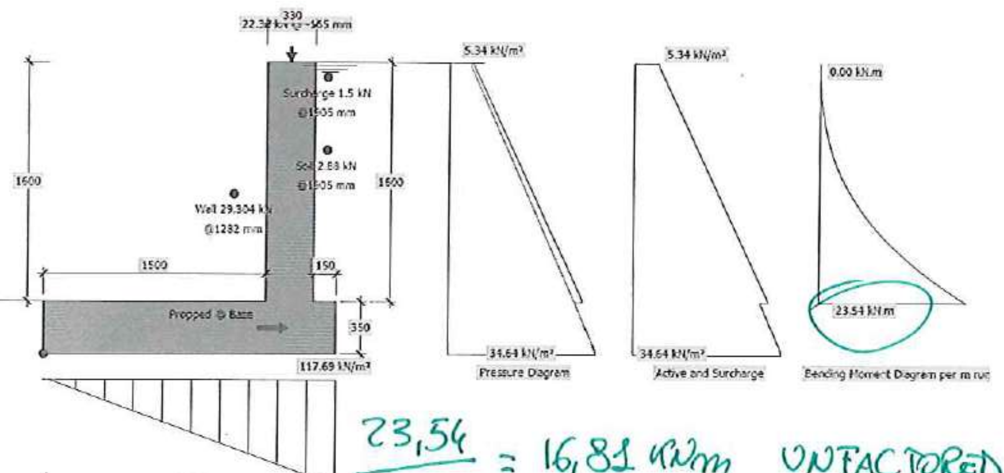
Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
Checked	: M. Egan
Approved	:

MasterKey : Retaining Wall Design to BS 8002 and BS 8110 : 1997
Load Case 1 - Permanent Case
Reinforced Concrete Retaining Wall with Reinforced Base



$\frac{23,54}{1,40}$ = 16,81 kNm UNFACTORED BY

Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Dry Soil 20.00, Saturated Soil 22.00, Submerged Soil 12.00, Concrete 24.00
Concrete grade	fcu 35 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 460 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Water table level 1600 mm
Unplanned excavation depth	Front of wall 195 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Vertical Line Loads	58.45 kN/m @ X -165 mm and Y 0 mm - Load type Dead
	22.36 kN/m @ X -165 mm and Y 0 mm - Load type Live
† Dimensions	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Soil bearing pressure	Allowable pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	28.014/180.457	0.155	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(41.314+0.395)	0.000	OK
Prop Reaction Case 2 (Service)	38.5 kN @ Base		

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Job Ref	: J001413
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Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
Checked	: M. Egan
Approved	:

Soil Pressure

Virtual Back	117.692/100 kN/m², Length under pressure 1.946 m	1.177	**Warning**
Wall Back (No uplift)	Max(1.414/100, 114.237/100) kN/m²	1.142	**Warning**

Structural Design

At Rest Earth Pressure

At rest earth pressures magnification	$(1+\mathrm{Sin}(\phi)) \times \sqrt{OCR} = (1+\mathrm{Sin}(17.74) \times \sqrt{1}$		1.3

Prop Reaction

Maximum Prop Reaction (Ultimate)	53.1 kN @ Base		

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main B12@100 (30 mm) Dist. B10@175 (42 mm)	1131 mm²	OK
Compression Steel Provided (Cover)	Main B10@300 (30 mm) Dist. B10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	294 mm, 1000 mm, 1131 mm², 460 N/mm², 35.0 N/mm²	278 mm	
Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 35 mm, 0.12	137.5 kN.m	
Moment Capacity Check (M/Mr)	M 23.5 kN.m, Mr 137.5 kN.m	0.171	OK
Wall Axial Design (N/Ncap)	N 135.3 kN, Ncap 4620.0 kN	0.029	OK
Wall Slenderness λ	Leff/tk =2.00x1600.0/330.0	9.7	OK
Wall Axial-Mom Design (M/Mr_{Axial})	M 23.5 kN, Mr_{Axial} 155.0 kN.m	0.152	OK
Shear Capacity Check	F 38.5 kN, vc 0.555 N/mm², Fvr 163.3 kN	0.24	OK

Base Top Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B16@150 (50 mm) Dist. B10@150 (66 mm)	1340 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	1340 mm², 58 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 0.4 kN.m, Mr 64.1 kN.m	0.006	OK
Shear Capacity Check	F 3.5 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.03	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main B16@150 (50 mm) Dist. B10@150 (66 mm)	1340 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	292 mm, 1000 mm, 1340 mm², 460 N/mm², 35 N/mm²	273 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 42 mm, 0.14	160.1 kN.m	
Moment Capacity Check (M/Mr)	M 31.3 kN.m, Mr 160.1 kN.m	0.196	OK
Shear Capacity Check	F 75.2 kN, vc 0.590 N/mm², Fvr 172.3 kN	0.44	OK

⚠ DESIGN AS RAFT

	Project		Job Ref	
GSE GREEN STRUCTURAL ENGINEERING	79 Guilford Street		J001413	
	Drawing Ref	Calculations by	Checked by	Sheet No
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	Part of Structure		Date	
	Basement design		March 2019	

UNDERPINNING RETAINING WALL 4b

WALL THICKNESS AND SELF WEIGHT

DC

- BASEMENT TO GF → H = 3,00 m | 37,80 kN/m
  THK 660 mm → 12,6 kN/m²

- GF to 1ST H = 3,50 m | 33,25 kN/m
  THK = 500 mm → 9,50 kN/m²

- 1ST to 2ND Floor H = 3,70 m | 31,08 kN/m
  THK = 440 mm → 8,40 kN/m²

- 2nd to 3rd H = 6,70 m | 47,57 kN/m
  3rd to roof THK = 375 mm → 7,1 kN/m²

TOTAL SW WALL 149,70 kN/m

	Project		Job Ref	
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	Basement design		March 2019	

ROOF LOAD

PITCHED ROOF DL = 9,80 kN/m²
LL = 0,60 kN/m²
Span load: 6,40 / 2 = 3,20 m
N° FLOORS 2

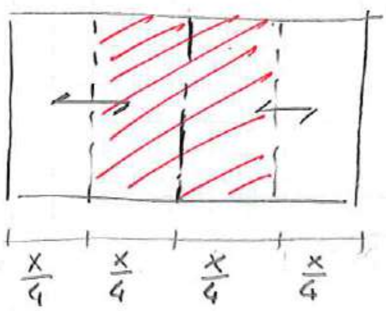
DL = 5,12 kN/m
LL = 3,84 kN/m

FLOORS

N° FLOORS 2 x 4 = 8
DL = 1,00 kN/m²
LL = (1,50 + 2,10)/2 = 1,80 kN/m²
Span load = 3,20 m

DL = 25,60 kN/m
LL = 46,08 kN/m

PARTIAL DL = 30,72
LL = 49,92



HALF OF THE LOAD WILL BE TAKEN BY BEAM AND SPREADED ON PARTY WALL

THE OTHER HALF PART WILL BE TAKEN BY FRONT AND REAR WALL

DC = 15,36 kN/m
LL = 24,96 kN/m

TOTAL LOAD

DL = 165,06 kN/m
LL = 24,96 kN/m

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TOTAL LOAD TO BE SPREADED ON THE BOTTOM OF THE URW

4a $DC = 56,17$ KN/m $L = 4,50$m

$L = (22,36 \cdot 0,9) = 20$ KN/m

[10% LIVE LOAD REDUCTION]

Spread on 11,20m

$DC = \frac{58,65 \times 4,50}{11,20} = 23,48$ KN/m

$L = \frac{20,00 \times 4,50}{11,20} = 8,04$ KN/m

4b $DC = 165,06$ KN/m $L = 2,00$m

$L = (24,96 \cdot 0,7) = 17,48$ KN/m

[30% LIVE LOAD REDUCTION]

Spread on 8,80 m

$DC = \frac{165,06 \times 2}{8,80\text{m}} = 37,51$ KN/m

$L = \frac{17,48 \cdot 2}{8,80} = 3,97$ KN/m

TOTAL LOAD

$DC = 60,99$ KN/m

$L = 12,01$ KN/m

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Date	**: 23 April 2019 / Ver. 2017.12**
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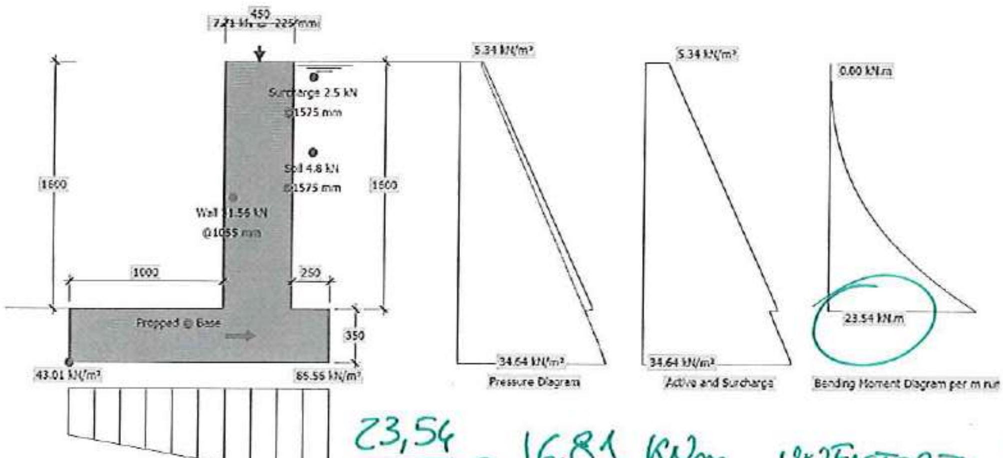
Approved	**:**

MasterKey : Retaining Wall Design to BS 8002 and BS 8110 : 1997
Load Case 1 - Permanent Case
Reinforced Concrete Retaining Wall with Reinforced Base



$\frac{23,54}{1,40} = 16,81$ KNm UNFACTORED BM

Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Dry Soil 20.00, Saturated Soil 22.00, Submerged Soil 12.00, Concrete 24.00
Concrete grade	fcu 35 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 460 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Water table level 1600 mm
Unplanned excavation depth	Front of wall 195 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Vertical Line Loads	63.57 kN/m @ X -225 mm and Y 0 mm - Load type Dead
	7.71 kN/m @ X -225 mm and Y 0 mm - Load type Live
† Dimensions	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Soil bearing pressure	Allowable pressure @ front 50.00 kN/m², @ back 50.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	28.014/132.122	0.212	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(39.743+0.395)	0.000	OK
Prop Reaction Case 2 (Service)	38.5 kN @ Base		

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Job Ref	: J001413
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Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
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Approved	:

Soil Pressure

Virtual Back (No uplift)	Max(43.013/50, 86.563/50) kN/m²	1.731	**Warning**
Wall Back (No uplift)	Max(47.653/50, 81.924/50) kN/m²	1.638	**Warning**

Structural Design

At Rest Earth Pressure

At rest earth pressures magnification	$(1+\mathrm{Sin}(\phi)) \times \sqrt{OCR} = (1+\mathrm{Sin}(17.74) \times \sqrt{1}$		1.3

Prop Reaction

Maximum Prop Reaction (Ultimate)	53.1 kN @ Base

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main B10@125 (30 mm) Dist. B10@125 (40 mm)	628 mm²	OK
Compression Steel Provided (Cover)	Main B10@300 (30 mm) Dist. B10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	415 mm, 1000 mm, 628 mm², 460 N/mm², 35.0 N/mm²	394 mm	
Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 20 mm, 0.05	108.3 kN.m	
Moment Capacity Check (M/Mr)	M 23.5 kN.m, Mr 108.3 kN.m	0.217	OK
Shear Capacity Check	F 38.5 kN, vc 0.373 N/mm², Fvr 154.9 kN	0.25	OK

Base Top Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 0.0 kN.m, Mr 64.1 kN.m	0.000	OK
Shear Capacity Check	F 0.0 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.00	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 30.4 kN.m, Mr 64.1 kN.m	0.474	OK
Shear Capacity Check	F 67.0 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.53	OK

! DESIGN AS RAFT

	Project		Job Ref	
GSE GREEN STRUCTURAL ENGINEERING	79 Guilford Street		J001413	
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		V. Ferraiuolo	M. Egan	
	Part of Structure		Date	
	Basement design		March 2019	

UNDERPINNING RETAINING WALL 5a H = 1,60 m

WALL

H = 3,00 m

330mm + pl = 7,40 kN/m² | DL = 22,20 kN/m

STUD WALL = 0,70 kN/m²
H = 10,00 m | DL = 7,00 kN/m

STAIRS

DL = 0,80 kN/m²

LL = 1,50 kN/m²

Span load = 1,00 m

N° STAIRS = 2

| DL = 1,60 kN/m
| LL = 3,00 kN/m

TOTAL LOAD DL = 30,80 kN/m
LL = 3,00 kN/m

Spread from 1,33 to 8,08

DL = (30,80 × 1,33) / 8,08 = 5,07 kN/m

LL = (3,00 × 1,33) / 8,08 = 0,50 kN/m

	Project		Job Ref	
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UNDERPINNING RETAINING WALL 6 $H = 1,60m$

REAR FAÇADE LOAD

* BASEMENT TO GROUND 660mm + pl = 12,60 KN/m²
$H = 3,00m$

TOTAL AREA = 3,00 x 6,50 = 19,50 m²
VOID = (1,45 + 0,76) x 1,95 = 4,31 m²
VOID = 22,1 %

DL = 29,45 KN/m

* GROUND TO 1ST 500mm + pl = 9,50 KN/m²
$H = 3,50$

TOTAL AREA = 3,50 x 6,50 = 22,75 m²
VOID (2,16 + 0,91) x 2,20 = 6,754 m
VOID 29,70 %

DL = 23,38 KN/m

* 1ST to 2ND 440mm + pl = 8,40 KN/m²
$H = 3,70$

TOTAL AREA = 3,70 x 6,50 = 24,05 m²
Void = (2,00 x 2,50) + [(1,38 + 0,71) x 1,00] = 7,09 m²
29,50 %

DL = 21,92 KN/m

	Project		Job Ref	
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2nd to Roof $\quad H = 6{,}70\,m$

$375\,mm = 7{,}10\,kN/m^2$

$DC = 8{,}45\,kN/m$

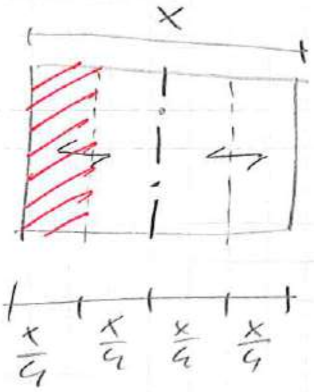
Area $= 6{,}70 \times 6{,}50 = 43{,}55\,m^2$

Void $= (1{,}30 \times 1{,}05) + (1{,}95 \times 1{,}95) + (1{,}07 \times 1{,}20 \times 2)$

$= 7{,}74\,m^2 = 17{,}77\%$

TOTAL WALL $DC = 83{,}20\,kN/m$

ROOF AND FLOORS AS PER URW 46 TAKEN 1/4 TOTAL LOAD



TOTAL LOAD = $DL = 30{,}72\,kN/m$, $LL = 49{,}92\,kN/m$ $\xrightarrow{25\%}$ $DC = 7{,}68$, $LL = 12{,}48$

TOTAL LOAD $\quad DC = 90{,}88\,kN/m$

$LL = 12{,}48\,kN/m$

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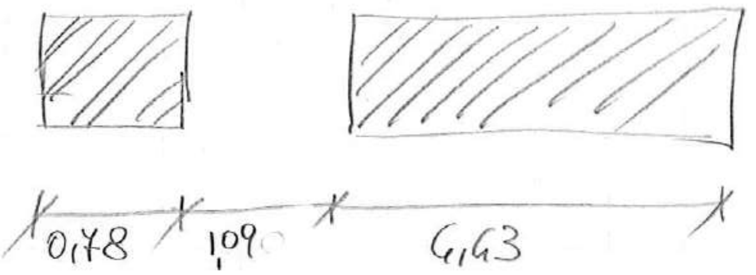
increase load considering open @ BASEMENT



PLAN

$1,09/2 = 0,545\ m$ — $DC = (90,88 \times 0,545)/6,63 = 11,18\ KN/m$

$CC = (12,68 \times 0,545)/6,63 = 1,56\ KN/m$

← $1,09/2 = 0,545\ m$

$DC = (90,88 \times 0,545)/0,78 = 63,50\ KN/m$

$LC = (12,68 \times 0,545)/0,78 = 8,72\ KN/m$

TOTAL LOAD

MASS CONCRETE

$DC = 90,88 + 63,50 = 154,38\ KN/m$

$LC = \qquad = 21,20\ KN/m$

URW 6

$DC = \qquad = 102,06\ KN/m$

$LC = \qquad = 14,02\ KN/m$

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URW 6 point load from 4,43 to 11,17 m

$$DL = \frac{107,06 \cdot 4,43}{11,17} = 40,48 \ KN/m$$

$$LL = \frac{14,02 \cdot 4,43}{11,17} = 5,56 \ KN/m$$

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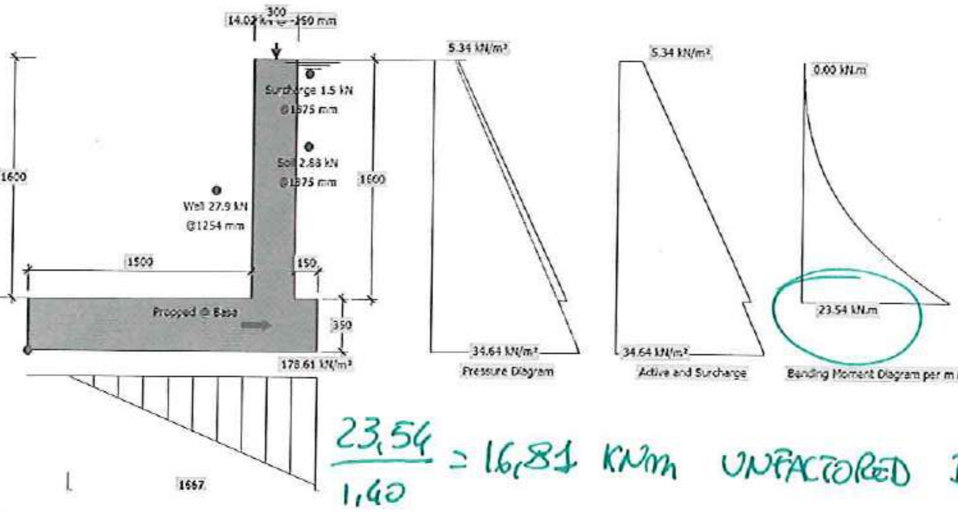
25642

Job Ref	: J001413
Sheet	: RCW6 - Permanent condi /
Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
Checked	: M. Egan
Approved	:

MasterKey : Retaining Wall Design to BS 8002 and BS 8110 : 1997
Load Case 1 - Permanent Case
Reinforced Concrete Retaining Wall with Reinforced Base



$\frac{23,54}{1,40} = 16,81$ KNm UNFACTORED BM

Summary of Design Data

Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Dry Soil 20.00, Saturated Soil 22.00, Submerged Soil 12.00, Concrete 24.00
Concrete grade	fcu 35 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Wall inner cover 30 mm, Wall outer cover 30 mm, Base cover 50 mm
Reinforcement design	fy 460 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 10.00 kN/m², Water table level 1600 mm
Unplanned excavation depth	Front of wall 195 mm

† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice

Additional Loads

Wall Propped at Base Level	Therefore no sliding check is required
Vertical Line Loads	102.6 kN/m @ X -150 mm and Y 0 mm - Load type Dead
	14.02 kN/m @ X -150 mm and Y 0 mm - Load type Live
† Dimensions	Ties, line loads and partial loads are measured from the inner top edge of the wall

Soil Properties

Soil bearing pressure	Allowable pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°
Base Friction and Cohesion	δ = Atn(0.75xTan(Atn(Tan(30)/1.2))) = 19.84°
Front Soil Friction and Cohesion	ϕ = Atn(Tan(21)/1.2) = 17.74°

Loading Cases

G_{Soil}- Soil Self Weight, G_{Wall}- Wall & Base Self Weight, Fv_{Heel}- Vertical Loads over Heel,
P_a- Active Earth Pressure, $P_{surcharge}$- Earth pressure from surcharge, P_p- Passive Earth Pressure

Case 1: Geotechnical Design	$1.00\ G_{Soil}+1.00\ G_{Wall}+1.00\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$
Case 2: Structural Ultimate Design	$1.40\ G_{Soil}+1.40\ G_{Wall}+1.60\ Fv_{Heel}+1.00\ P_a+1.00\ P_{surcharge}+1.00\ P_p$

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	28.014/235.614	0.119	OK

Wall Sliding - Virtual Back Pressure

$Fx/(Rx_{Friction}+Rx_{Passive})$	0.000/(53.730+0.395)	0.000	OK
Prop Reaction Case 2 (Service)	38.5 kN @ Base		

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Job Ref	: J001413
Sheet	: RCW6 - Permanent condi /
Made by	: V. Ferraiuolo
Date	: 23 April 2019 / Ver. 2017.12
Checked	: M. Egan
Approved	:

Soil Pressure

Virtual Back	178.609/100 kN/m², Length under pressure 1.667 m	1.786	**Warning**
Wall Back	173.913/100 kN/m², Length under pressure 1.712 m	1.739	**Warning**

Structural Design

At Rest Earth Pressure

At rest earth pressures magnification	$(1+\text{Sin}(\phi)) \times \sqrt{OCR} = (1+\text{Sin}(17.74) \times \sqrt{1}$	1.3

Prop Reaction

Maximum Prop Reaction (Ultimate)	53.1 kN @ Base

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Main B16@150 (30 mm) Dist. B10@200 (46 mm)	1340 mm²	OK
Compression Steel Provided (Cover)	Main B10@300 (30 mm) Dist. B10@300 (40 mm)	262 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	262 mm, 1000 mm, 1340 mm², 460 N/mm², 35.0 N/mm²	243 mm	
Mr=fn(above,As',d',x,x/d)	262 mm², 35 mm, 42 mm, 0.16	142.5 kN.m	
Moment Capacity Check (M/Mr)	M 23.5 kN.m, Mr 142.5 kN.m	0.165	OK
Wall Axial Design (N/Ncap)	N 182.2 kN, Ncap 4200.0 kN	0.043	OK
Wall Slendernessλ	Leff/tk =2.00x1600.0/300.0	10.7	OK
Kmin = (Nuz-N)/(Nuz-Nbal)	Min(1.0, 4666.7 - 182.2)/(4666.7 - 1672.5)	1.0	
M_{add}= N.Kmin.h.λ^2/2000	182.2x1.0x300.0x10.7²/2000	3.0kN.m	
(M+Madd)/Mr_{Axial}	M+Madd 26.6 kN, Mr_{Axail} 162.1 kN.m	0.164	OK
Shear Capacity Check	F 38.5 kN, vc 0.629 N/mm², Fvr 164.7 kN	0.23	OK

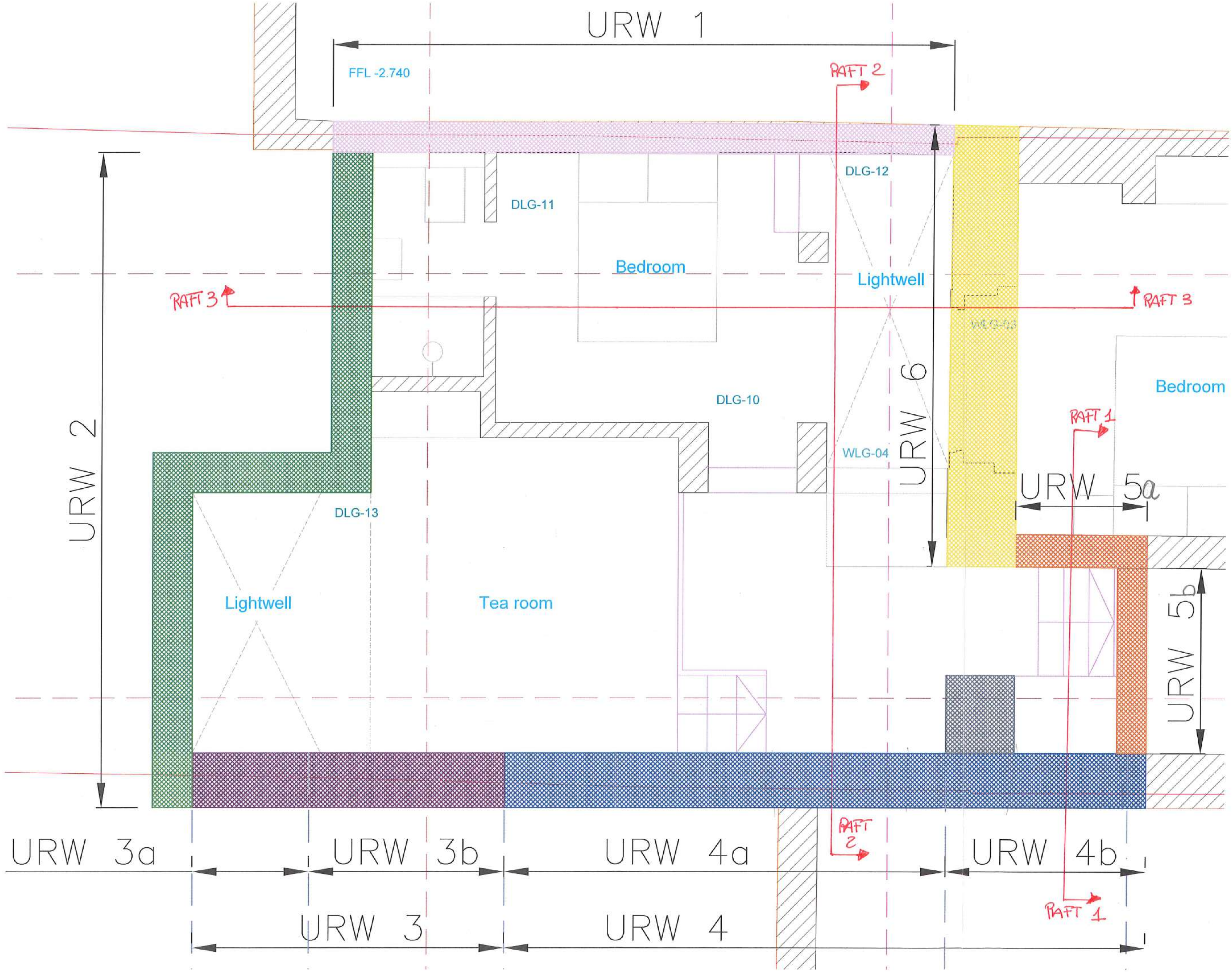
Base Top Steel Design

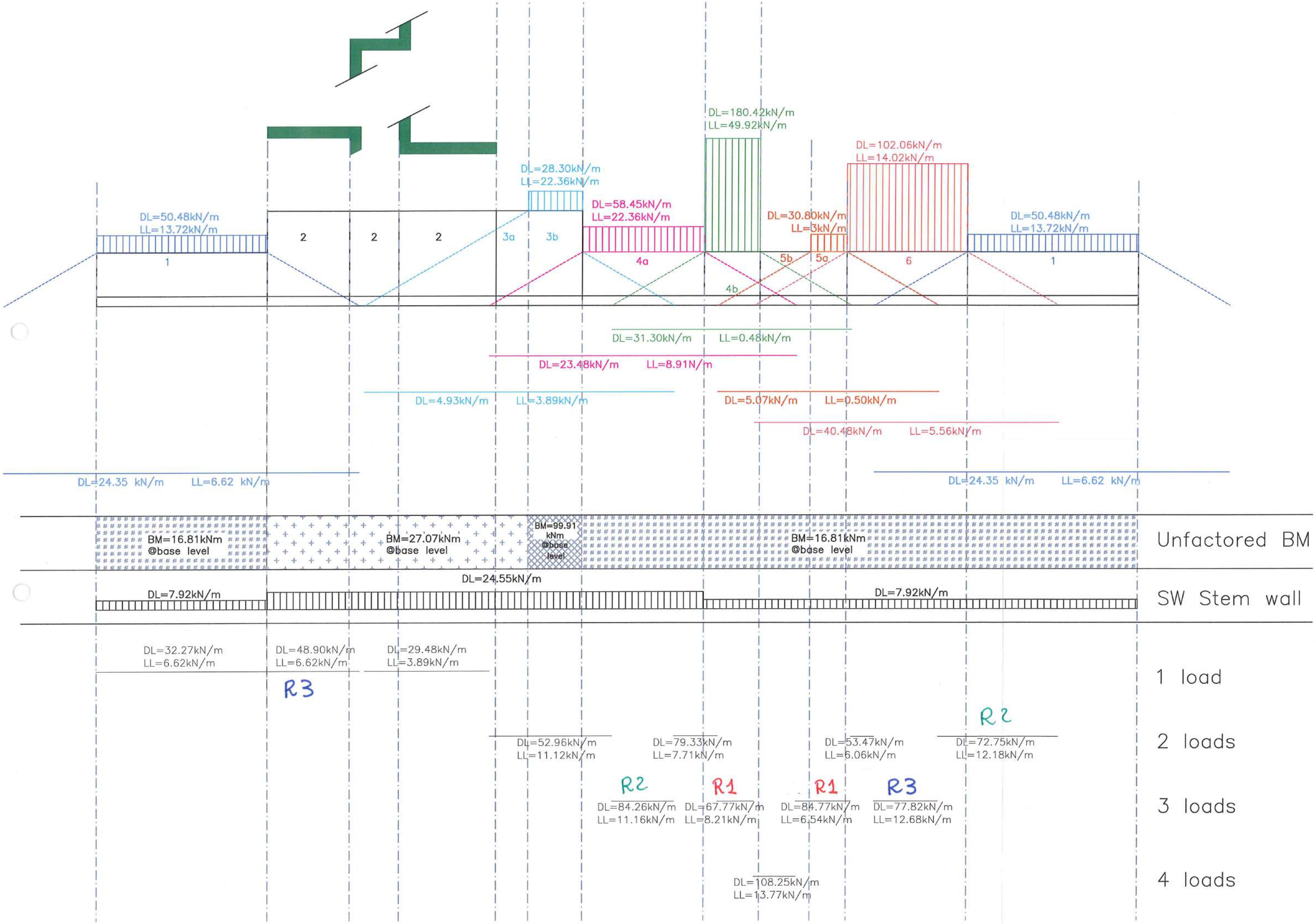
Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	OK
Compression Steel Provided (Cover)	Main B16@150 (50 mm) Dist. B10@150 (66 mm)	1340 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	295 mm, 1000 mm, 524 mm², 460 N/mm², 35 N/mm²	280 mm	
Mr=fn(above,As',d',x,x/d)	1340 mm², 58 mm, 16 mm, 0.06	64.1 kN.m	
Moment Capacity Check (M/Mr)	M 1.4 kN.m, Mr 64.1 kN.m	0.021	OK
Shear Capacity Check	F 12.2 kN, vc 0.429 N/mm², Fvr 126.5 kN	0.10	OK

Base Bottom Steel Design

Steel Provided (Cover)	Main B16@150 (50 mm) Dist. B10@150 (66 mm)	1340 mm²	OK
Compression Steel Provided (Cover)	Main B10@150 (50 mm) Dist. B10@150 (60 mm)	524 mm²	
Leverarm z=fn(d,b,As,fy,Fcu)	292 mm, 1000 mm, 1340 mm², 460 N/mm², 35 N/mm²	273 mm	
Mr=fn(above,As',d',x,x/d)	524 mm², 55 mm, 42 mm, 0.14	160.1 kN.m	
Moment Capacity Check (M/Mr)	M 32.1 kN.m, Mr 160.1 kN.m	0.200	OK
Shear Capacity Check	F 94.7 kN, vc 0.590 N/mm², Fvr 172.3 kN	0.55	OK

! DESIGN AS RAFT


URW 1
FFL -2.740
RAFT 2
DLG-11
DLG-12
Bedroom
Lightwell
RAFT 3
RAFT 3
URW 6
DLG-10
Bedroom
RAFT 1
WLG-04
URW 5a
URW 2
DLG-13
Lightwell
Tea room
URW 5b
RAFT 2
URW 3a
URW 3b
URW 4a
URW 4b
RAFT 1
URW 3
URW 4



DL=180.42kN/m
LL=49.92kN/m
DL=102.06kN/m
LL=14.02kN/m
DL=28.30kN/m
LL=22.36kN/m
DL=58.45kN/m
LL=22.36kN/m
DL=50.48kN/m
LL=13.72kN/m
DL=30.80kN/m
LL=3kN/m
DL=50.48kN/m
LL=13.72kN/m
2
2
2
3a
3b
1
4a
5b
5a
6
1
4b
DL=31.30kN/m LL=0.48kN/m
DL=23.48kN/m LL=8.91N/m
DL=4.93kN/m LL=3.89kN/m
DL=5.07kN/m LL=0.50kN/m
DL=40.48kN/m LL=5.56kN/m
DL=24.35 kN/m LL=6.62 kN/m
DL=24.35 kN/m LL=6.62 kN/m
BM=16.81kNm @base level
BM=27.07kNm @base level
BM=99.91 kNm @base level
BM=16.81kNm @base level
Unfactored BM
DL=7.92kN/m
DL=24.55kN/m
DL=7.92kN/m
SW Stem wall
DL=32.27kN/m
LL=6.62kN/m
DL=48.90kN/m
LL=6.62kN/m
DL=29.48kN/m
LL=3.89kN/m
R3
1 load
R2
DL=52.96kN/m
LL=11.12kN/m
DL=79.33kN/m
LL=7.71kN/m
DL=53.47kN/m
LL=6.06kN/m
DL=72.75kN/m
LL=12.18kN/m
2 loads
R2
R1
R1
R3
DL=84.26kN/m
LL=11.16kN/m
DL=67.77kN/m
LL=8.21kN/m
DL=84.77kN/m
LL=6.54kN/m
DL=77.82kN/m
LL=12.68kN/m
3 loads
DL=108.25kN/m
LL=13.77kN/m
4 loads

GSE GREEN STRUCTURAL ENGINEERING	Project	79 Guilford Street		Job ref.	J 001 413
	Calculations by VF	Checked by	AA	Sheet No.	of
	Element	Raft load		Date	April 2019

Underpinning Retaining wall	Dead Load [kN/m]	Live Load [kN/m]	Bending moment @base [kNm]
1	72.75	12.18	16.81
2	48.90	6.62	27.07
3a	52.96	11.12	27.07
3b			99.91
4a	84.26	11.16	16.81
4b	67.77	8.21	16.81
5a	84.77	6.54	16.81
5b	108.25	13.77	16.81
6	77.82	12.68	16.81

	Project		Job Ref	
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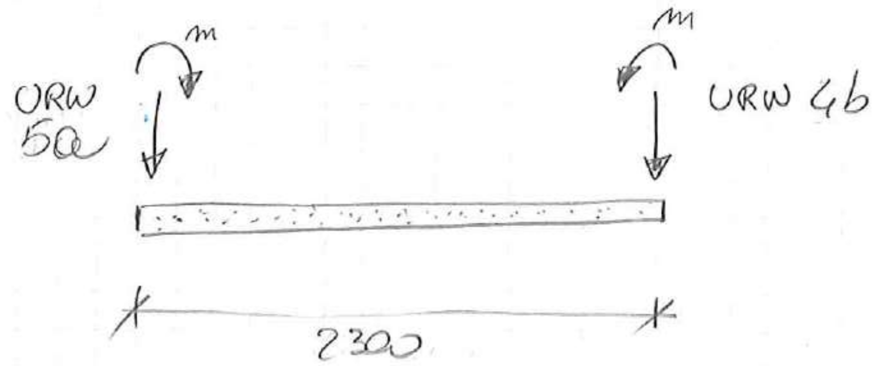
	Part of Structure Basement design		Date March 2019	

RAFT 1



M = 16,81 KNM

URW 4b
THK = 660 mm
DL = 67,78 KN/m
LL: 8,21 KN/m

URW 5
THK 330 mm
DL = 86,48 KN/m
LL = 6,56 KN/m

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	Calcs for Raft 1				Start page no./Revision 1 / P1	
	Calcs by V. Ferraiuolo	Calcs date Apr.19	Checked by V. Ferraiuolo	Checked date Apr.19	Approved by M. Egan	Approved date Apr.19

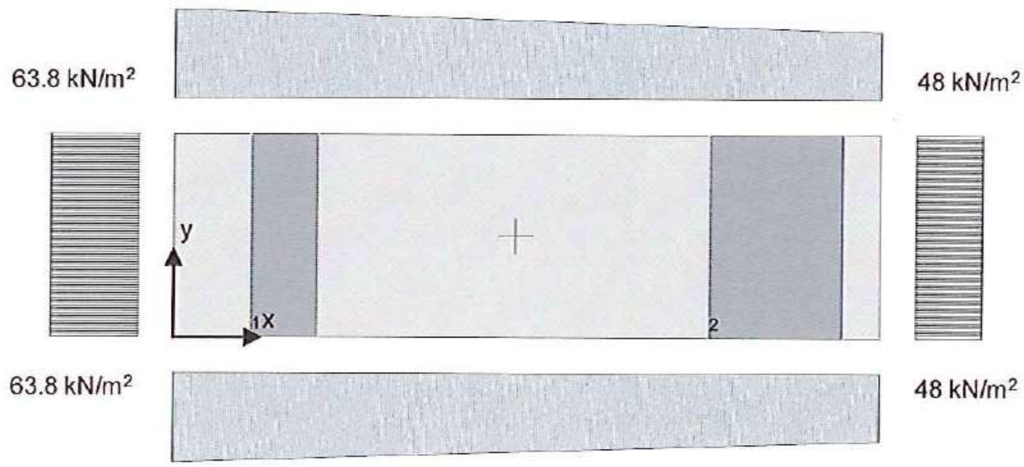
FOUNDATION ANALYSIS (EN1997-1:2004)

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

TEDDS calculation version 3.2.14

Pad foundation details

Length of foundation	L_x = **3520 mm**
Width of foundation	L_y = **1000 mm**
Foundation area	$A = L_x \times L_y$ = **3.520 m²**
Depth of foundation	h = **350 mm**
Depth of soil over foundation	h_{soil} = **0 mm**
Level of water	h_{water} = **0 mm**
Density of water	γ_{water} = **9.8 kN/m³**
Density of concrete	γ_{conc} = **24.0 kN/m³**



Column no.1 details

Length of column	l_{x1} = **325 mm**
Width of column	l_{y1} = **1000 mm**
position in x-axis	x_1 = **550 mm**
position in y-axis	y_1 = **500 mm**

Column no.2 details

Length of column	l_{x2} = **660 mm**
Width of column	l_{y2} = **1000 mm**
position in x-axis	x_2 = **3000 mm**
position in y-axis	y_2 = **500 mm**

Soil properties

Density of soil	γ_{soil} = **19.0 kN/m³**
Characteristic cohesion	c'_k = **0 kN/m²**
Characteristic effective shear resistance angle	ϕ'_k = **25 deg**
Characteristic friction angle	δ_k = **19.3 deg**

Foundation loads

Self weight	$F_{swt} = h \times \gamma_{conc}$ = **8.4 kN/m²**

Column no.1 loads

Permanent load in z — F_{Gz1} = **84.8** kN

Variable load in z — F_{Qz1} = **6.5** kN

Permanent moment in x — M_{Gx1} = **-16.8** kNm

Column no.2 loads

Permanent load in z — F_{Gz2} = **67.8** kN

Variable load in z — F_{Qz2} = **8.2** kN

Permanent moment in x — M_{Gx2} = **16.8** kNm

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-axis — $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Gz2} + F_{Qz1} + F_{Qz2}$ = **196.9** kN

Moments on foundation

Moment in x-axis — $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + M_{Gx1} + F_{Gz2} \times x_2 + M_{Gx2} + F_{Qz1} \times x_1 + F_{Qz2} \times x_2$ = **330.2** kNm

Moment in y-axis — $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Gz2} \times y_2 + F_{Qz1} \times y_1 + F_{Qz2} \times y_2$ = **98.4** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_x = M_{dx} / F_{dz} - L_x / 2$ = **-83** mm

Eccentricity of base reaction in y-axis — $e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y)$ = **63.8** kN/m²

$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y)$ = **63.8** kN/m²

$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y)$ = **48** kN/m²

$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y)$ = **48** kN/m²

Minimum base pressure — $q_{min} = \min(q_1, q_2, q_3, q_4)$ = **48** kN/m²

Maximum base pressure — $q_{max} = \max(q_1, q_2, q_3, q_4)$ = **63.8** kN/m²

Presumed bearing capacity

Presumed bearing capacity — $P_{bearing}$ = **65.0** kN/m²

PASS - Presumed bearing capacity exceeds design base pressure

	Project		Job Ref	
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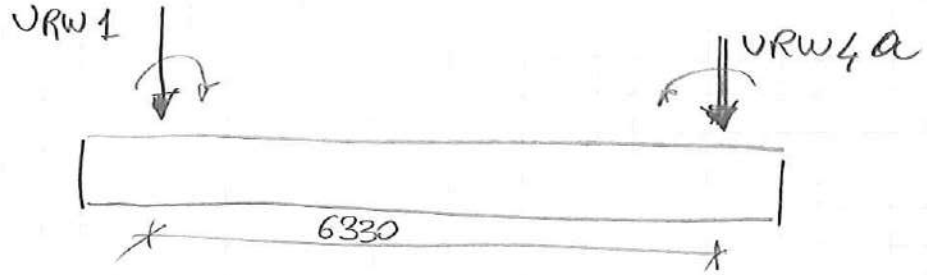
	Part of Structure		Date	
	Basement design		March 2019	

PART 2



M = 16,81 kNm (H = 1,6 m retaining soil)

URW 4a THK = 330 mm
DL = 84,26 kN/m
LL = 11,16 kN/m

URW 1 : THK = 330 mm
DL = 72,75 kN/m
LL = 12,18 kN/m

Green Structural Engineering Ltd Unit 5 Quayside Lodge William Morris Way London SW6 2UZ	Project 79 Guilford Street				Job no. J 001413	
	Calcs for Raft 2				Start page no./Revision 1 / P1	
	Calcs by V. Ferraiuolo	Calcs date Apr.19	Checked by V. Ferraiuolo	Checked date Apr.19	Approved by M. Egan	Approved date Apr.19

FOUNDATION ANALYSIS (EN1997-1:2004)

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

TEDDS calculation version 3.2.14

Pad foundation details

Length of foundation L_x = **6660** mm

Width of foundation L_y = **1000** mm

Foundation area $A = L_x \times L_y$ = **6.660** m^2

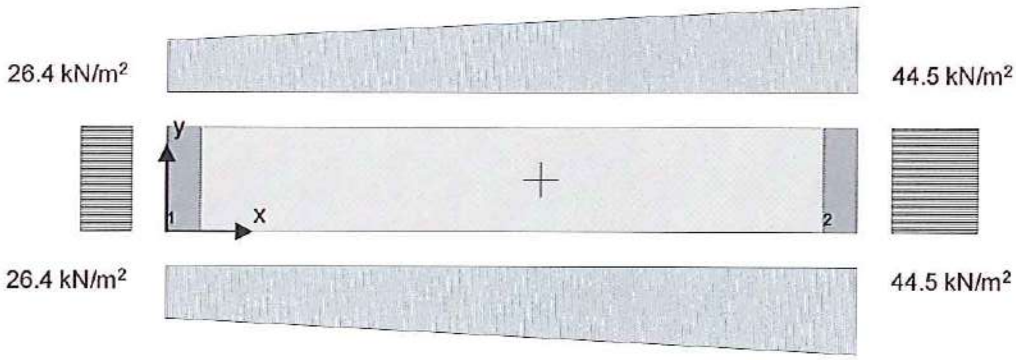
Depth of foundation h = **350** mm

Depth of soil over foundation h_{soil} = **0** mm

Level of water h_{water} = **0** mm

Density of water γ_{water} = **9.8** kN/m^3

Density of concrete γ_{conc} = **24.0** kN/m^3



Column no.1 details

Length of column l_{x1} = **330** mm

Width of column l_{y1} = **1000** mm

position in x-axis x_1 = **165** mm

position in y-axis y_1 = **500** mm

Column no.2 details

Length of column l_{x2} = **330** mm

Width of column l_{y2} = **1000** mm

position in x-axis x_2 = **6495** mm

position in y-axis y_2 = **500** mm

Soil properties

Density of soil γ_{soil} = **19.0** kN/m^3

Characteristic cohesion c'_k = **0** kN/m^2

Characteristic effective shear resistance angle ϕ'_k = **25** deg

Characteristic friction angle δ_k = **19.3** deg

Foundation loads

Self weight $F_{swt} = h \times \gamma_{conc}$ = **8.4** kN/m^2

Column no.1 loads

Permanent load in z F_{Gz1} = **72.8** kN

Variable load in z F_{Qz1} = **12.2** kN

Permanent moment in x — M_{Gx1} = **16.8** kNm

Column no.2 loads

Permanent load in z — F_{Gz2} = **84.3** kN

Variable load in z — F_{Qz2} = **11.2** kN

Permanent moment in x — M_{Gx2} = **16.8** kNm

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-axis — $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Gz2} + F_{Qz1} + F_{Qz2}$ = **236.3** kN

Moments on foundation

Moment in x-axis — $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + M_{Gx1} + F_{Gz2} \times x_2 + M_{Gx2} + F_{Qz1} \times x_1 + F_{Qz2} \times x_2$ = **853.7** kNm

Moment in y-axis — $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Gz2} \times y_2 + F_{Qz1} \times y_1 + F_{Qz2} \times y_2$ = **118.1** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_x = M_{dx} / F_{dz} - L_x / 2$ = **283** mm

Eccentricity of base reaction in y-axis — $e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y)$ = **26.4** kN/m²

$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y)$ = **26.4** kN/m²

$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y)$ = **44.5** kN/m²

$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y)$ = **44.5** kN/m²

Minimum base pressure — $q_{min} = \min(q_1, q_2, q_3, q_4)$ = **26.4** kN/m²

Maximum base pressure — $q_{max} = \max(q_1, q_2, q_3, q_4)$ = **44.5** kN/m²

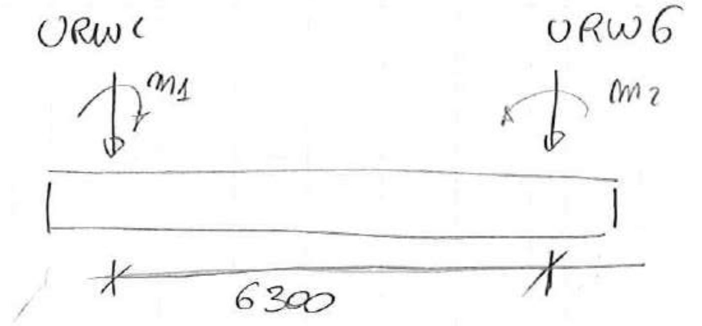
Presumed bearing capacity

Presumed bearing capacity — $P_{bearing}$ = **65.0** kN/m²

PASS - Presumed bearing capacity exceeds design base pressure

	Project		Job Ref	
GSE GREEN STRUCTURAL ENGINEERING	79 Guilford Street		J001413	
	Drawing Ref	Calculations by: V. Ferraiuolo	Checked by: M. Egan	Sheet No
	Part of Structure: Basement design		Date: March 2019	

PART 3



$m_1 = -27{,}07$ kNm (3,10m eff. height with prop top and bottom)
$m_2 : 16{,}81$ kNm (1,60m eff. height)

URW2 THK = 330 mm
DC = 48,90 kN/m
LC = 6,62 kN/m

URW 6 THK 660 mm

DC = 77,82 kN/m
LC = 12,68 kN/m

Green Structural Engineering Ltd Unit 5 Quayside Lodge William Morris Way London SW6 2UZ	Project 79 Guilford Street				Job no. J 001413	
	Calcs for Raft 3				Start page no./Revision 1 / P1	
	Calcs by V. Ferraiuolo	Calcs date Apr.19	Checked by V. Ferraiuolo	Checked date Apr.19	Approved by M. Egan	Approved date Apr.19

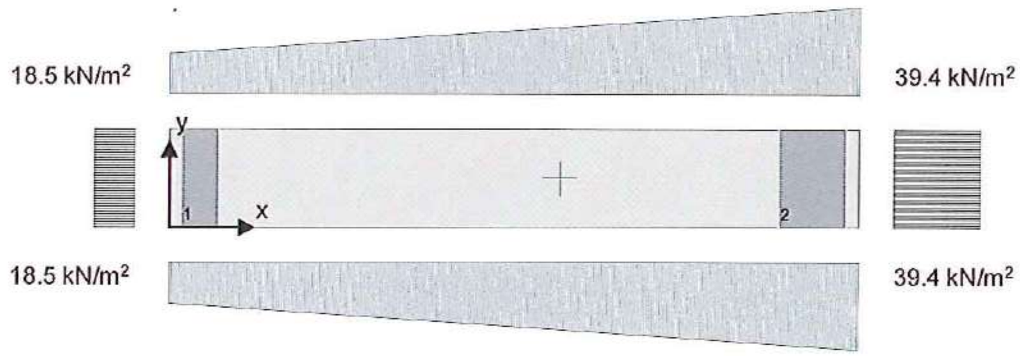
FOUNDATION ANALYSIS (EN1997-1:2004)

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

TEDDS calculation version 3.2.14

Pad foundation details

Length of foundation	L_x = **7100** mm
Width of foundation	L_y = **1000** mm
Foundation area	$A = L_x \times L_y$ = **7.100** m²
Depth of foundation	h = **350** mm
Depth of soil over foundation	h_{soil} = **0** mm
Level of water	h_{water} = **0** mm
Density of water	γ_{water} = **9.8** kN/m³
Density of concrete	γ_{conc} = **24.0** kN/m³



Column no.1 details

Length of column	l_{x1} = **350** mm
Width of column	l_{y1} = **1000** mm
position in x-axis	x_1 = **315** mm
position in y-axis	y_1 = **500** mm

Column no.2 details

Length of column	l_{x2} = **660** mm
Width of column	l_{y2} = **1000** mm
position in x-axis	x_2 = **6620** mm
position in y-axis	y_2 = **500** mm

Soil properties

Density of soil	γ_{soil} = **19.0** kN/m³
Characteristic cohesion	c'_k = **0** kN/m²
Characteristic effective shear resistance angle	ϕ'_k = **25** deg
Characteristic friction angle	δ_k = **19.3** deg

Foundation loads

Self weight	$F_{swt} = h \times \gamma_{conc}$ = **8.4** kN/m²

Column no.1 loads

Permanent load in z	F_{Gz1} = **48.9** kN
Variable load in z	F_{Qz1} = **6.6** kN

Permanent moment in x — M_{Gx1} = **-27.1** kNm

Column no.2 loads

Permanent load in z — F_{Gz2} = **77.8** kN

Variable load in z — F_{Qz2} = **12.7** kN

Permanent moment in x — M_{Gx2} = **16.8** kNm

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-axis — $F_{dz} = A \times F_{swt} + F_{Gz1} + F_{Gz2} + F_{Qz1} + F_{Qz2}$ = **205.7** kN

Moments on foundation

Moment in x-axis — $M_{dx} = A \times F_{swt} \times L_x / 2 + F_{Gz1} \times x_1 + M_{Gx1} + F_{Gz2} \times x_2 + M_{Gx2} + F_{Qz1} \times x_1 + F_{Qz2} \times x_2$ = **818.1** kNm

Moment in y-axis — $M_{dy} = A \times F_{swt} \times L_y / 2 + F_{Gz1} \times y_1 + F_{Gz2} \times y_2 + F_{Qz1} \times y_1 + F_{Qz2} \times y_2$ = **102.8** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-axis — $e_x = M_{dx} / F_{dz} - L_x / 2$ = **428** mm

Eccentricity of base reaction in y-axis — $e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

Pad base pressures

$q_1 = F_{dz} \times (1 - 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y)$ = **18.5** kN/m²

$q_2 = F_{dz} \times (1 - 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y)$ = **18.5** kN/m²

$q_3 = F_{dz} \times (1 + 6 \times e_x / L_x - 6 \times e_y / L_y) / (L_x \times L_y)$ = **39.4** kN/m²

$q_4 = F_{dz} \times (1 + 6 \times e_x / L_x + 6 \times e_y / L_y) / (L_x \times L_y)$ = **39.4** kN/m²

Minimum base pressure — $q_{min} = \min(q_1, q_2, q_3, q_4)$ = **18.5** kN/m²

Maximum base pressure — $q_{max} = \max(q_1, q_2, q_3, q_4)$ = **39.4** kN/m²

Presumed bearing capacity

Presumed bearing capacity — $P_{bearing}$ = **65.0** kN/m²

PASS - Presumed bearing capacity exceeds design base pressure

APPENDIX C

GSE UNDERPINNING SPECIFICATION

General Underpinning Specification

1. The walls to the perimeter of the new basement shall be underpinned in reinforced concrete. The underpins shall take the vertical loads from the walls and horizontal loads from the earth.
2. Underpinning bases shall be excavated in short sections not exceeding 1000mm in width.
3. The sequence of the underpinning shall be such that any given underpin will be completed, dry packed and a minimum period of 48 hours lapsed before an adjacent excavation commenced to form another underpin.
4. In the event that the existing foundations to the wall are found to be unstable, sacrificial steel jacks shall be installed underneath the foundation to prop the bottom few courses of bricks. These steel jacks shall be left in place and shall be incorporated into the concrete stem.
5. In the event that the ground is unstable, lateral propping shall be provided as required to the rear of the excavation and to the sides of the excavated working trench. The front and side faces of the excavation shall be propped using trench sheeting or plywood, timber boards and acrow props as appropriate. Sacrificial back– shutters shall be used to the rear face of the excavation (i.e. underneath the wall) if required. Cementitious grout will be poured behind the back – shutters to fill up the voids behind the back – shutters.
6. Excavation for an underpin section shall be dug in a day, and the concrete to the base shall be poured by the end of the same day.
7. The concrete to the stem of the underpin shall be poured the following day. This shall be poured up to within 50 – 75mm of the underside of the existing wall foundations.
8. On the following day, the gap between the concrete and the underside of the existing foundation shall be dry packed with C35 concrete using 5 – 10mm coarse aggregate and "Combex 100" expanding admixture by Fosroc UK Ltd in accordance with their instructions.
9. Once the dry pack has gained sufficient strength, any protrusions of the footings into our site shall be carefully trimmed back using hand tools to avoid causing any damage to the foundation. The protrusions shall be trimmed back to be flush in-line with the face of the wall above.
10. A minimum of 48 hours shall be allowed before adjacent sections are excavated to form a new underpin.
11. Adjacent underpins shall be connected using T12 dowel bars 600mm long, 300mm embedment each side, at 300mm vertical centers.
12. Concrete cover to reinforcement shall be 35mm for cast against shutter or the top surface of the basement slab, 40mm for cast against blinding and 75mm for cast against earth.
13. Grade of concrete shall be C35 with minimum cement content 300kg/m3, maximum free water to cement ratio 0.60, slump 100mm.

Unit 5, Quayside, Sir William Morris Way, Fulham SW6 2UZ
Registered in England Number 06533911

APPENDIX D

SITE INVESTIGATION REPORT (extract)

Report

Basement Impact Assessment

79 Guilford Street, London
WC1N 1DF

for

Andrew and Romain Pingannaud

Ref: GGC19750/R1.1 April 2019

Gabriel GeoConsulting Limited

Highfield House, Rolvenden Road, Benenden, Kent TN17 4EH

Company No. 6455714, registered in England and Wales. Registered office as above.

Tel: 01580 241044

e: info@gabrielgeo.co.uk www.gabrielgeo.co.uk

Basement Impact Assessment

Site: **79 Guilford Street**
London,
WC1N 1DF

Client: **Andrew and Romain Pingannaud**

Report Status: **FINAL**		
Role	**By**	**Signature**
Desk study, factual reporting, ground model, PDISP analyses and damage category assessment by:	Heather Baker MSci FGS	
PDISP analyses checked by:	Alexander Goodsell BSc ACSM FGS	
Slope/ground stability aspects approved by: Surface flow and flooding aspects approved by:	Mike Summersgill MSc CEng MICE C.WEM FCIWEM	
Factual report checked by: Impact Assessments by: & Subterranean (Groundwater) flow aspects approved by:	Keith Gabriel MSc DIC CGeol FGS UK Registered Ground Engineering Adviser	

Foreword

This report has been prepared in accordance with the scope and terms agreed with the Client, and the resources available, using all reasonable professional skill and care. The report is for the exclusive use of the Client and shall not be relied upon by any third party without explicit written agreement from Gabriel GeoConsulting Ltd.

This report is specific to the proposed site use or development, as appropriate, and as described in the report; Gabriel GeoConsulting Ltd accept no liability for any use of the report or its contents for any purpose other than the development or proposed site use described herein.

This assessment has involved consideration, using normal professional skill and care, of the findings of ground investigation data obtained from the Client and other sources. Ground investigations involve sampling a very small proportion of the ground of interest as a result of which it is inevitable that variations in ground conditions, including groundwater, will remain unrecorded around and between the exploratory hole locations; groundwater levels/pressures will also vary seasonally and with other man-induced influences; no liability can be accepted for any adverse consequences of such variations.

This report must be read in its entirety in order to obtain a full understanding of our recommendations and conclusions.

Contents Page

1. INTRODUCTION

1.1 This Basement Impact Assessment (BIA) has been prepared in support of a planning application to be submitted to the London Borough of Camden (LBC) for the extension of an existing single-storey basement beneath No.79 Guilford Street, WC1N 1DF. Further details of the proposed works are given in Section 3. This assessment is in accordance with the requirements of the London Borough of Camden (LBC) Local Plan 2017, Policy A5 in relation to basement construction, and follows the requirements set out in LBC's guidance document 'CPG Basements' (March 2018).

1.2 This assessment has been supervised/approved by Keith Gabriel, a Chartered Geologist with an MSc degree in Engineering Geology (who has specialised in slope stability and hydrogeology), and reviewed by Mike Summersgill, a Chartered Civil Engineer and Chartered Water and Environmental Manager with an MSc degree in Soil Mechanics (geotechnical and hydrology specialist). Both authors have previously undertaken assessments of basements in several London Boroughs.

1.3 **Desk Study**: A site inspection (walk-over survey) of the property and its surroundings was undertaken on 19th February 2019. Photos from that visit are presented in Appendix A. Desk study data have been collected from various sources including geological data, environmental data and historic maps from Groundsure which are presented in Appendices C, D and E. Relevant information from the desk study and site inspection is presented in Sections 2–6.

1.4 **Ground Investigations:** Sitework for the ground investigation (borehole and trial pits) was undertaken on 19th February 2019, the findings from which are presented in Section 9 and Appendix F.

1.5 The Screening, Scoping and basement impact assessments in accordance with CPG Basements, Stages 1-4, are presented in Sections 7, 8 & 10 respectively.

1.6 The following site-specific documents in relation to the proposed extension and planning application have been considered:

BÜF Architecture (Existing):

- Drg No. A010 20-P010 — As Existing Basement Plan
- Drg No. A010 20-P011 — As Existing Ground Floor Plan
- Drg No. A010 20-P012 — As Existing First Floor Plan
- Drg No. A010 20-P013 — As Existing Second Floor Plan
- Drg No. A010 20-P014 — As Existing Third Floor Plan
- Drg No. A010 20-P015 — As Existing Roof Plan
- Drg No. A010 20-P020 — As Existing Elevation AA (Front)
- Drg No. A010 20-P021 — As Existing Elevation BB (Rear)
- Drg No. A010 20-P030 — As Existing Section AA (Longitudinal, through rear projection)
- Drg No. A010 20-P031 — As Existing Section BB (Longitudinal, through rear garden area)

BÜF Architecture (Proposed):

- Drg No. A010 20-P110 — As Proposed Basement
- Drg No. A010 20-P111 — As Proposed Ground Floor Plan
- Drg No. A010 20-P112 — As Proposed First Floor Plan
- Drg No. A010 20-P113 — As Proposed Second Floor Plan
- Drg No. A010 20-P114 — As Proposed Third Floor Plan
- Drg No. A010 20-P115 — As Proposed Roof Plan
- Drg No. A010 20-P120 — As Proposed Elevation AA (Front)
- Drg No. A010 20-P121 — As Proposed Elevation BB (Rear)
- Drg No. A010 20-P130 — As Proposed Section AA (Longitudinal, through rear projection)
- Drg No. A010 20-P131 — As Proposed Section BB (Longitudinal, through rear garden area)
- Drg No. A010 20-P132 — As Proposed Section CC (Transverse, through rear extension).

Green Structural Engineering (GSE):

- Drg No.J001413-SSK001 — 'Typical section through Section'(CC) & Proposed Sequence
- Drg No. J001413-SK-2 — As Proposed Basement – Wall Mark-up/Line Loads
- Drg No. (?) — Mark Up of U/P Retaining Walls (for basement extension) showing Raft Section lines 1-3.
- Drg No. (?) — Load distribution around raft
- Load Takedown calculations & Retaining Wall moment calculations.
- Tabulated 'Raft Loads', including retaining wall loads (Walls 1-6)
- Email dated 27/03/19 — Responses to GGC queries, including underpin dimensions & 4No. sections.
- Email**s** dated 01/04/19 — Line loads and self-weight of raft slab
- Email**s** dated 02/04/19 — Loads for front vaults, garden boundary walls and No.5 Colonnade

This report should be read in conjunction with all the documents and drawings listed above.

1.7 Instructions to prepare this Basement Impact Assessment were confirmed by email on 14th February 2019.

2. THE PROPERTY AND TOPOGRAPHIC SETTING

2.1 No.79 Guilford Street is a five-storey (including basement) terraced house with a single-storey rear projection and a single-storey rear extension at ground floor level, situated within the Bloomsbury Conservation Area, in the London Borough of Camden. As shown in Figure 1, No.79 is located on the north side of Guilford Street, between the adjoining No.78 to the west and adjoining No.80 to the east (see also Cover Photo and Photo 1 in Appendix A). To the north, the site is bounded by No.5 Colonnade.

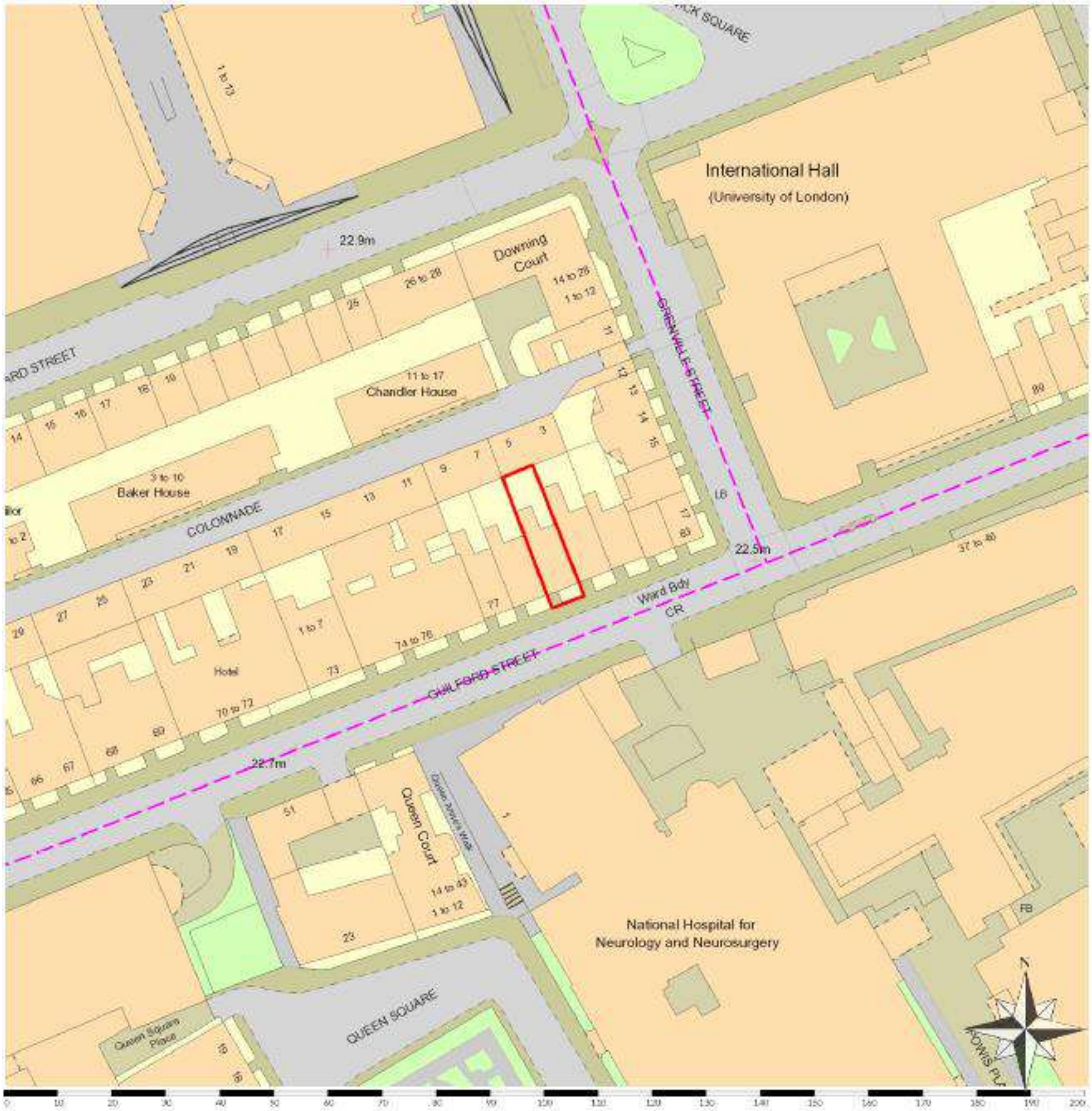


Figure 1: Extract from 1:1,250 OS map (not to scale) with the site outlined in red.

2.2 Externally, at the front of the property, there is a lightwell which can be accessed from the Guilford Street public footway via a concrete staircase lined with metal railings (Photo 2). Beneath the footway at the front of the property there is a single vault which can be accessed via the front lightwell. There is a second vault to the south-west of this (so straddling the No.79/78 boundary) that was inaccessible at the time of the ground investigation, possibly infilled. The main front entrance to the property, at ground floor level, is two steps (approximately 0.37m) up from the Guilford Street footway.

2.3 To the rear of the property is a garden, partly covered in pea gravel, with an area of wooden decking alongside the original rear projection. The garden is bounded by brick boundary walls shared with No.78 to the south-west and No.80 to the north-east, and by the rear wall of No.5 Colonnade to the north-west (Photos 3, 4 & 5). Immediately behind the decking, 'L-shaped' raised planters are present on both sides of the garden, extending to the rear end of No.80's rear projection and to the rear end of No.79's extension (excluding the aviary). Further planting beds are located adjacent to the rear wall of No.5 Colonnade (Photo 5). The rear basement lightwell, adjacent to the main rear wall of No.79, is surrounded by metal railings with no upstand (Photo 3).

2.4 At the front of the property there was some evidence of minor crack damage and past repairs, particularly to the window lintels at ground floor level and to the front door frame (Photo 6). The front wall of the terrace also shows distortion out of plane, which is particularly noticeable at the junction of No's 80 & 81 where No.80's wall is bowed out relative to No.81's re-built(?) wall. There was also evidence of distortion and repairs to No.79's rear wall and either extensive re-pointing, or re-building, of the brickwork to the upper storey (Photo 3). There were no significant trees observed at No.79, though there was a semi-mature Plane tree in the footway outside No.78. Successful applications were found on LBC's planning database for the removal of sycamore trees at No's.77 & 78 in 1994 (see Section 2.12 below). The height of the trees was not provided.

2.5 Reference to the earliest available historic Ordnance Survey (OS) maps (see Appendix E) shows that No.79 and the surrounding properties on Guilford Street had all been constructed prior to 1875. Two maps at the same scale were published around this time, 1871-1875 and 1874-1875, both of which show properties to the rear of the site, along 'Colonnade Mews'. Both maps show No's 5 & 3 Colonnade with longer site footprints than now, which projected into the rear gardens of No's 79 and 80, whereas the remainder of the Mews terrace to the west were as the current layout. Both these maps show No's 5 & 3 with small irregular courtyards and separate buildings at the south end of their sites, adjoining the rear projections to No's 79 & 80. Two hospitals had already been constructed to the south of No.79 ('National Hospital for Paralyzed & Epileptic' and the 'Hospital for Sick Children' on Great Ormond Street); these were separated from Guilford Street by properties on the south side of Guilford Street and by Grenville Mews.

2.6 By the publication of 1896 (1:1,056 scale), No’s 1, 3 & 5 Colonnade Mews had been demolished, including the buildings adjoining the rear projections of No’s 79 & 80. This map is the first to show rear lightwells adjacent to the main rear walls of No’s 75-80. No.79’s site had been extended to its current size and the site of the former No.1 Colonnade had become the rear gardens to houses on Grenville Street.

2.7 Between 1896 and 1951 there are very few changes to No.79 and the surrounding properties, with the exception that by 1920 (1:10,560 scale map) No.5 Colonnade to the rear and the adjacent No.3 had been re-built. The rear lightwells of No’s 77-80 are all shown as extending between the north-east boundaries and rear projections of the respective properties, though whether this was a correction because the presence of bridges over these lightwells had not previously been appreciated is unknown. The rear projection of the adjoining No.78 (and No.77) are both shown as having been reduced in length. Along the terrace to the south-west, No’s 70 to 76 are shown as one large building (a number of these properties were shown as combined on the 1916 1:2,500 map). On the south side of Guilford Street, significant redevelopment of the hospitals and surrounding buildings is recorded, including demolition of the terrace properties directly opposite No.79, leaving only a single small building opposite No.79.

2.8 Between publication of the 1951 map and the 1995 map (the most recent historic OS map available at 1:1,250 scale), No.79 and the adjoining No’s 78 and 80 show no significant changes, and the building footprints remain as they are shown in 2019 (Figure 1). The National Hospital on the south side of Guilford Street has changed during this time, with a large extension towards Guilford Street shown between 1974 (when the site was vacant) and 1987, and the demolition and reconstruction of significant sections of the main building between 1987 and 1995.

2.9 Other notable changes to the wider area around the site include the demolition of the rear projections of the properties fronting onto Bernard Street and the mews buildings on the north side of Colonnade, and a subsequent redevelopment into Chandler House and Baker House. The small-scale maps (1:10,000 scale) indicate this occurred between 2002 and 2010. Other changes include the construction of Russell Square Tube Station, roughly 110 - 120m north-west of No.79, first shown on 1916 (1:2,500 scale), and also the demolition of the Foundling Hospital by 1938 (1:10,560 scale) roughly 250m north-east of No.79.

2.10 The London County Council Bomb Damage Map for this area (London Topographical Society, 2005) indicates that No.79 suffered “General blast damage - not structural”. This classification was given to all properties in this terrace block on the north side of Guilford Street, with the exception of No.74 which is recorded as “Damaged beyond repair” and No.81, which is recorded as “Seriously damaged; doubtful if repairable”. The Colonnade properties to the rear were not affected, and the properties on the south side of Guilford Street (opposite No.79) are recorded as “Clearance areas”. The closest V1 flying bomb is recorded as landing at the centre of Russell Square, some 250m south-west of No.79.

Topography:

2.11 Guilford Street is located on a broadly east/south-east facing slope which leads down to the base of a very shallow valley which was formed by the River Fleet, one of the 'lost' rivers of London. The location of this valley is defined by the 15m and 20m contours to the east of No.79, shown in Figure 2.

2.12 The contours on Figure 2 indicate an overall slope across the site of approximately 0.4° towards the east, calculated between the 25m contour to the west and 20m contour to the east. Using the spot heights from Figure 1, the Guilford Street carriageway outside No.79 falls north-eastwards with a slope angle of 0.13°.

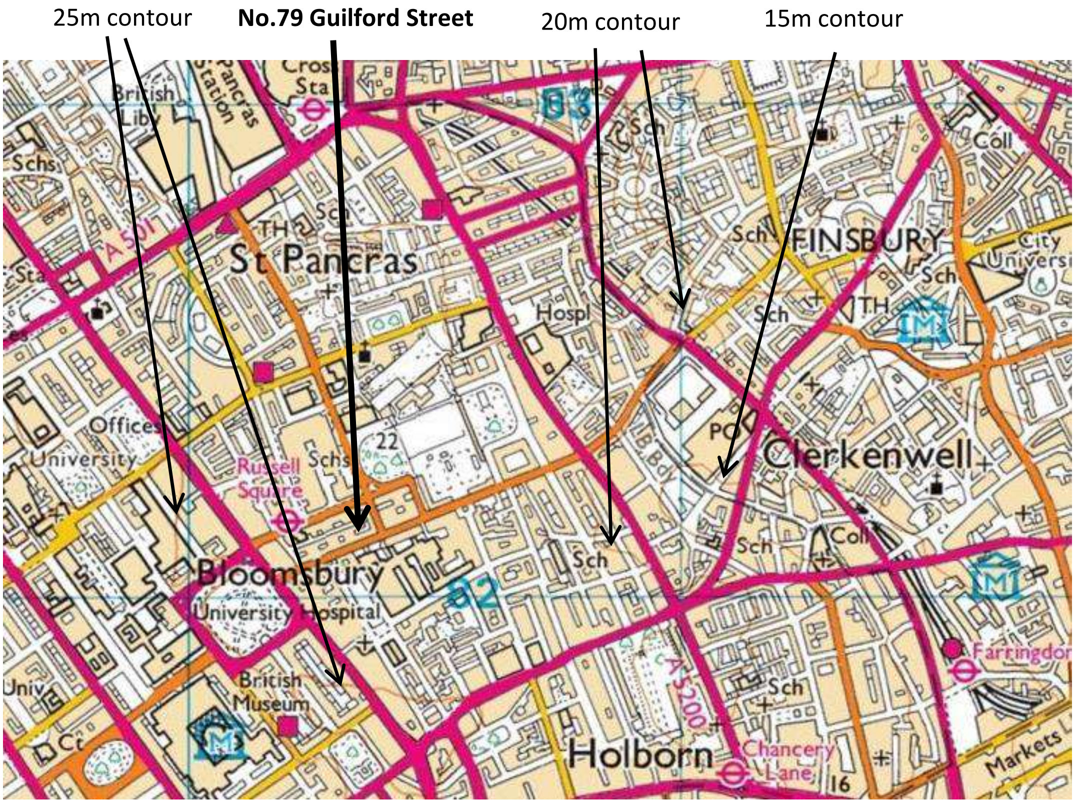


Figure 2: Extract from 1:25,000 scale Ordnance Survey map showing site location.

Planning Searches:

2.13 A search was made of planning applications on the Camden Council's website on 12th February 2019, in order to obtain details of the planning history for No.79 and details of any other basements which have been constructed or are planned in the vicinity of the property, the results of which are listed below:

- **No.79 Guilford Street**: Applications (2012/6198/L & 2012/6170/P) relating to "*Change of use from nurses hostel (Sui Generis) to single-family dwellinghouse (Class C3)*" were both granted on 2nd May 2013.
- **(Adjoining) No.78 Guilford Street**: Various applications:
  - Applications (2018/0303/P & 2018/1104/L) relating to "*Change of use from nurses' hostel (Sui Generis) to residential (C3) to provide 5 self-contained flats (…) and associated alterations including reinstatement of front entrance, creation of rear lightwell and window, various external alterations including landscaping works and provision of refuse and cycle storage*" were both registered on 28th February 2018.
  - Application (2012/6203/L) for the "*Removal of basement front door, installation of ground floor entrance door and internal alterations in connection with change of use of existing nurses' hostel (Sui Generis) to residential flats (Class C3)…*" was granted on 22nd May 2013.
  - Application (9492345) "*Seeking permission to remove a Sycamore tree at the above address*" was given the status "*Agree to tree removal without replacement*" on 7th December 1994. There were no documents available with the application.
- **(Adjoining) No.80 Guilford Street**: Extensive applications and amendments relating to application (2012/6167/P) for the "*Change of use from Nurses' hostel (Sui Generis) to Residential (Use Class C3) (…) and associated alterations, including removal of sub division walls*" which was granted (subject to Section 106 Legal Agreement) on 22nd May 2013. A later application (2013/8203/P) includes reference to "*associated basement terrace*", this was granted (subject to a Section 106 legal agreement) on 25th September 2014.
- **(Adjoining site) No.5 Colonnade**: Database searched, no relevant applications found.
- **No.77**: Application (9492344) "*Seeking permission to remove a Sycamore tree at the above address*" was given the status "*Agree to tree removal without replacement*" on 7th December 1994. There were no documents available with the application.

2.14 Many of the properties in this terrace have applications to convert nurses' hostels to residential properties, with plans attached showing existing single-storey basements. Other large buildings in the vicinity have substantial basements/lower ground floors, including the National Hospital of Neurology and Neurosurgery on the south side of Guilford Street, Downing Court between Colonnade and Grenville Street; and International Hall, between Guilford Street and Brunswick Square Gardens.

3. PROPOSED BASEMENT

3.1 Planning permission will be sought for the proposed works at No.79 Guilford Street, as shown in BÜF's drawings (see paragraph 1.6), which include:

- Excavation and construction of a single-storey rear basement extension.
- Retaining and lowering the existing rear lightwell.
- Construction of new basement lightwell to the rear of the proposed extension.
- Lowering the floor levels of front vaults.
- Alterations to internal basement walls.
- Demolition of ground floor rear projection and construction of rear extension, to create a 'Lower Gallery' and 'Upper Gallery'. Two courtyards are proposed at ground floor level adjacent to the two lightwells. The 'Lower Gallery' and rear courtyard will extend to the rear wall of No.5 Colonnade.
- Redesigning the ground floor interior to create a 'Georgian Gallery' and a 'Library', including construction of a new wall adjacent to the front door.
- At first floor level, a 'lightweight metalwork balcony' is proposed on the main rear wall, above the rear lightwell. Comments on BÜF's 'As Proposed First Floor Plan' (Drg No. 20-P112) indicate this balcony is to provide access to the roof terrace above the upper gallery.
- To the front of the first and second floors, reinstated original metalwork is indicated around the front windows.
- Other internal alterations to upper floors.

3.2 BÜF's Sections AA, BB and CC (Drg No's A010 20-P130, 20-P131, 20-P132 respectively) show variable finished floor levels (FFL) throughout the proposed basement and ground floor extension; these are summarised in Table 1 below. The FFLs are relative to ground floor FFL, which is taken as **23.60m AOD** from Edward Gardner Surveys' 'Existing Upper Ground Floor' (Combined Survey, Drg No. 18-007-2).

3.3 The structural design proposed by Green Structural Engineering (GSE) consists of reinforced concrete (RC) underpins with 300mm thick stems and 350mm thick base slab to be designed as a raft, as shown in their drawings listed in paragraph 1.6 and discussed in subsequent correspondence. A mass concrete underpin is proposed beneath part of an internal basement wall, below the main rear wall of the upper floors of the property and adjacent to No.79/78 party wall, and a column pad footing is proposed beneath a central internal wall in the basement extension. Despite the varying FFLs (paragraph 3.2 and Table 1), a consistent formation (founding) level for the rear extension of **4.72m** below ground floor FFL, or **18.88m AOD,** is proposed. This includes an allowance of 200mm for insulation, cavity drainage and floor finishes in addition to the underpin base/slab beneath the 'tea room' and rear lightwell, so the floor levels in other sections of the basement extension will be built up above this. The front vaults will be founded at **19.55m AOD** and the rear ground floor extension at **21.93m AOD**.

3.4 Based on a search of the LBC's planning applications (paragraph 2.12), the adjoining No's 80 and 78 both have existing single-storey basements. No.78 has a very similar footprint to that of No.79, meaning underpinning will be required along the No.79/78 party wall, as No.78's rear basement projection is along the No.78/77 party wall. No.80's basement extends beyond the rear wall of No.79's proposed basement extension; however, the lower level of No.79's proposed basement relative to No.80's means the No.79/80 party wall will require underpinning. The lowered rear extension relative to No.79's existing basement means the main rear wall of the building will also require underpinning. Although the proposed basement does not extend to the rear boundary of the site, the lowered level of the rear ground floor extension means the No.79/No.5 Colonnade rear wall and 79/78 and 79/80 garden boundary walls will also all need to be underpinned.

Table 1: Existing and proposed levels, and depths of excavation

Location (Proposed)	Existing FFL		Proposed FFL		Proposed Formation Level		Excavation depth (m)
	m ASD	m AOD	m ASD	m AOD	m ASD	m AOD	
Front Vaults	-3.05	20.55	-3.50	20.10	-4.05	19.55	1.00
Existing rear lightwell	-2.84	20.76	-4.02	19.58			1.88
Rear basement Bedroom and Ensuite	-0.51	23.09	-4.02	19.58			4.21
Front end of Tea Room	-2.74	20.86	-4.17	19.43	-4.72	18.88	1.98
Rear end of Tea Room	-0.51	23.09	-4.17	19.43			4.21
Tea Room lightwell	-0.58	23.02	-4.17	19.43			4.14
'Ante Space'	-2.74	20.86	-3.57	20.03			1.98
Ground Floor 'Upper Gallery' and courtyard adjacent to 79/80 party wall	-0.51	23.09	-0.67	22.93	N/A	N/A	0.36 (assuming floor finishes of 0.2m)
Ground Floor 'Lower Gallery' and rear courtyard	-0.55	23.05	-1.27	22.33	-1.67	21.93	1.12

4. GEOLOGICAL SETTING

4.1 Mapping by the British Geological Survey (BGS) indicates that the site is underlain by the Lynch Hill Gravel Member over London Clay Formation. Figure 3 shows an extract from Figure 16 of the Camden GHHS (Camden Geological, Hydrogeological and Hydrological Study by Arup, November 2010) which illustrates the site geology of the Bloomsbury area.

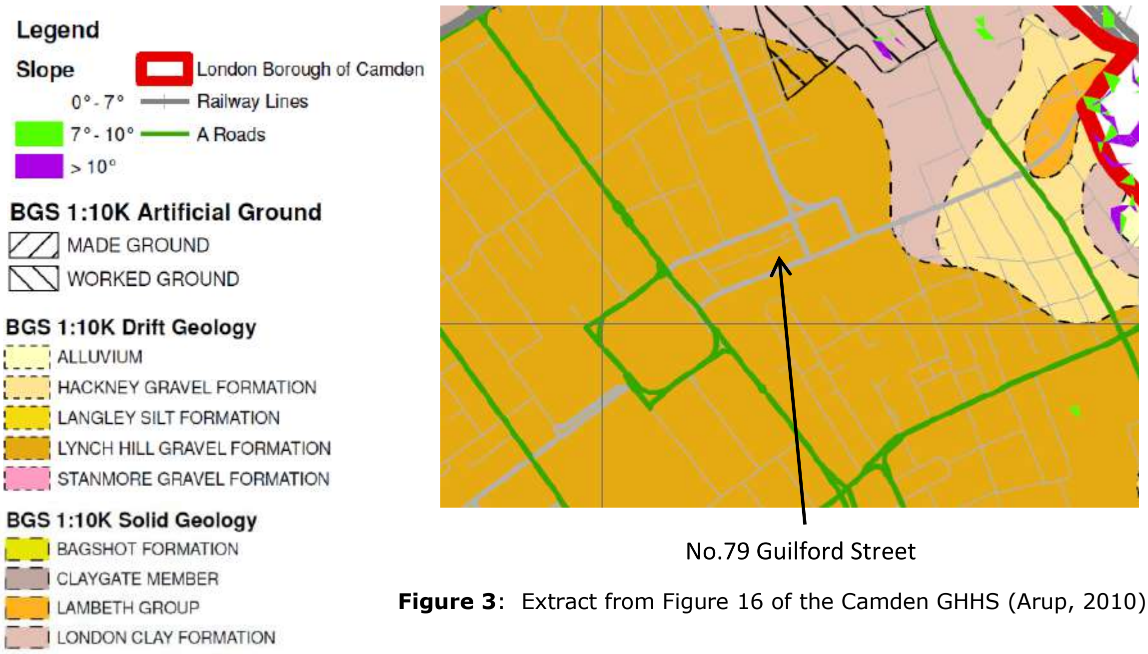


Figure 3: Extract from Figure 16 of the Camden GHHS (Arup, 2010)

4.2 In urban parts of London, the River Terrace Deposits are typically overlain by Made Ground.

4.3 The Lynch Hill Gravel Member is a River Terrace Deposit (RTD) associated with the River Thames and its tributaries. This was formerly classified as a Formation, and is described collectively with the other RTDs by the BGS memoir (Ellison et al., 2004) as "*variable proportions of sands and gravels*" along with local "*impersistent beds … of clayey and silty sand*". Subordinate clay horizons also occur at all levels within the River Terrace Deposits, while peats are rarely present (usually at the base of the sequence). The thickness of the Lynch Hill Gravel Member varies across London, and is known to reach 7.5m thick in the Paddington area. The thickness can also vary significantly locally over short distances, owing to the presence of deep drift-filled hollows (sometimes called scour features).

4.4 The London Clay is well documented as being a firm to very stiff over-consolidated clay which is typically of high or very high plasticity and high volume change potential. As a result it undergoes considerable volume changes in response to variations in its natural moisture content (the clay shrinks on drying and swells on subsequent rehydration). These changes can occur seasonally, in response to normal climatic variations, to depths of up to 1.50m and to much greater depths in the presence of the trees whose roots abstract moisture from the clay. The clay will also swell when unloaded by excavations such as those required for the construction of basements.

4.5 The Groundsure GeoInsight report (Appendix C) records:

- Three records of historical surface ground working features within 250m of the site, all of which refer to a 'Disused Cemetery', 239-241m north of the site (App.C, Section 4.1).
- Multiple records of historical underground working features within 1000m of the site. These all consist of recorded tunnels, the closest of which is 726m north-east of No.79 though this database clearly does not include London Underground Ltd's (LUL) Piccadilly Line tunnels (App.C, Section 4.2 and see below).
- No records of historical 'mining' features within 1000m of the site (App.C, Section 5.1).
- LUL's Piccadilly Line passes 112m west of the site; this is the only tunnel or underground railway line recorded within 250m of the site (App.C, Section 9.1). A single record of historical railway and tunnel features is recorded within 250m of the site; this a tunnel 242m north-west of the site (App.C, Section 9.2).
- The site is within 500m of the route of the Crossrail 1 rail project (App.C, Section 9.5).

It should be noted that these databases are based on mapping evidence, so inevitably will provide an incomplete record of underground workings.

4.6 The results of the BGS natural ground subsidence hazard classifications are provided in GroundSure GeoInsight report (Appendix C, Section 6). All indicated 'Negligible' or 'Very Low' hazard ratings within 250m of the site, with the exception of 'Shrink – Swell Clay' for which a 'Moderate' hazard rating was given, reflecting the presence of the London Clay Formation close to surface.

4.7 Three natural cavities are recorded within 1000m of the site; these are an 'unknown' cavity 358m north-east, a 'sinkhole' 446m east and a 'Scour Hollow' 606m north-east of the site (App. C, Section 5.6).

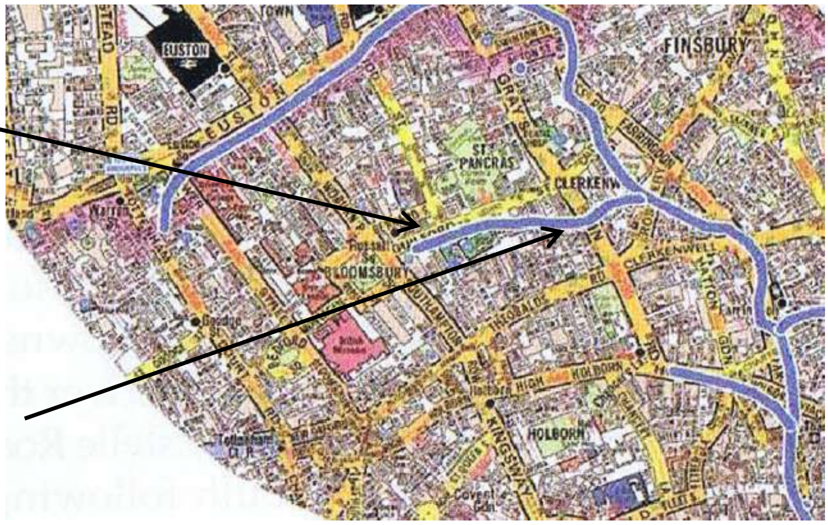
4.8 A search of the BGS boreholes database was undertaken for information on previous ground investigations or wells in the vicinity of the site, the locations of which are presented on the location plan in Appendix B. The strata depths in a selection of the closest boreholes are summarised in Table 1. For full strata descriptions, reference should be made to the logs in Appendix B.

- The location plan indicates a number of closer BGS borehole logs to No.79, however these have not been presented as they are either incomplete records or, in the case of the five records at Great Ormond Street Hospital, are relatively shallow trial pit records.
- The unit names given on BGS log TQ38SW/2850 are outdated: the Woolwich and Reading Beds refer to the Lambeth Group. The current BGS terminology is used throughout this report.

Table 2: Summary of Strata in BGS Boreholes and Local Ground Investigations							
Strata (abbreviated descriptions)	**Depths (m) and levels (m AOD) to base of strata**						
	TQ38SW/123		TQ38SW/531	TQ38SW/1021		TQ38SW/2850	
Ground level (m AOD):	Depth	**Level 22.25**	Depth	Depth	**Level 25.28**	Depth	**Level 19.20**
Date drilled	March 1950		July 1954	March 1970		January 1927	
Made Ground and/or Topsoil	1.77	**20.48**	1.07	1.45	**23.83**	-	**-**
Stiff, fissured, brownish grey, silty CLAY (River Terrace Deposit?)	-	**-**	-	3.30	**21.98**	-	**-**
'Dense' clayey f-m SAND and f,m,c GRAVEL (River Terrace Deposit)	4.27	**17.98**	3.05	5.00	**20.28**	4.57	**14.63**
Soft, light brown/red CLAY (Weathered London Clay Formation)	4.42	**17.83**	3.51	-	**-**	4.88	**14.32**
Firm to stiff, fissured, variably grey, blue and brown mottled, silty CLAY with occasional pockets of fine sand and rare claystone fragments (London Clay Formation)	32.92	**-10.67**	>6.55	>12.00	**13.28**	16.46	**2.74**
Stiff, dark grey, grey, red & brown mottled sandy CLAY with some gravel (Lambeth Group)	>37.49	**-15.24**				36.27	**-17.07**
Thanet Sand						39.62	**-20.42**
Upper Chalk						>167.64	**-148.44**
Groundwater Seepage/Strike	13.72	**8.53**	2.74	4.00	**21.28**	81.69	**-62.49**
Groundwater Standing	-	**-**	2.44	4.50	**20.78**	79.25	**-60.05**

5. HYDROLOGICAL SETTING (SURFACE WATER)

No.79 Guilford Street



Former course of River Fleet

Figure 4: Extract from Map 9 of Barton & Myers' Lost Rivers of London (2016) – 'The course of the Fleet from Hampstead and Highgate to the Thames at Blackfriars'.

5.1 Barton and Myers' map of the 'lost' rivers of London (Figure 4) indicates that a former tributary of the Fleet (which is now carried in dedicated culverts or the sewer system) runs along Guilford Street, from the junction between Guilford Street and Russell Square to the main channel of the Fleet at Clerkenwell. There is some discrepancy between the 2016 edition (presented in Figure 4) and the 1992 edition, which records this tributary forking into three smaller tributaries, with one running north along Guilford Place into Coram Fields, one running west along Great Ormond Street and one running south along Lamb's Conduit Street.

5.2 The historic OS maps (presented in Appendix E) do not show any surface water features in the vicinity of No.79. This is compatible with the local tributary (or tributaries) of the River Fleet having been culverted prior to the publication of the earliest map, dated 1871-1875.

5.3 To the front of the property, the lightwell is partially protected from surface water run-off from the Guilford Street footway by a low up-stand which forms the base of the metal railings (Photos 1 & 2). This up-stand is not present where the access steps meet the footway. There are two steps up from the footway to the front door of No.79, which is also raised above the level of neighbouring No.78's lightwell (see Photo 2). Although the adjacent level of No.80 is above that of No.79, there is another low up-stand topped with metal railings along the boundary. These upstands reduce the potential for surface water run-off into the lightwell from the adjoining properties.

5.4 The rear garden is partly surfaced by pea gravel above a plastic membrane, with wooden decking covering the area adjacent to the rear main wall and rear lightwell (Photos 3 & 5). The raised planting beds along sections of the boundary wall and to the rear of the garden (Photo 4) will provide some surface water infiltration and temporary retention.

5.5 To the rear of No.79, the garden is bounded on all sides by high brickwork walls (No.80's rear extension, garden boundary walls, party with No's 78 and 80, and the rear wall of No.5), so there is no potential for surface water run-off from the adjoining properties.

5.6 The following hydrological data for the site has been obtained from the Groundsure Enviro Insight report (see Appendix D), including:

- The 'Ordnance Survey MasterMap Water Network' does not hold records for any entries of rivers or other water courses within 500m of the site (App.D, Section 6.10).
- There are no surface water features recorded within 250m of the site (App.D, Section 6.11).
- There are four surface water abstraction licences within 2000m of the site. Two are active and are along Regent's Canal; at Maiden Lane Bridge, 1378m north of the site and at City Road Basin, 1948m north-east of the site (App.D, Section 6.4).
- There are no flood defences, no areas benefitting from flood defences and no flood storage areas within 250m of the site (App.D, Sections 7.4, 7.5 & 7.6).

5.7 Figure 15 of the Camden Geological, Hydrogeological and Hydrological Study (Arup, 2010) shows that all the flooding in the 1975 and 2002 flooding events occurred to the north and north-west of the borough, so are all over 2km from No.79. The "areas with the potential to be at risk of surface water flooding" are highlighted in the same locations and along the far eastern boundary of the borough, the closest of which is approximately 850m east of No.79.

5.8 Maps provided by the Environment Agency on the GOV.UK website show that the site lies within Flood Zone 1, which is defined as areas where flooding from rivers and the sea is very unlikely, with less than a 0.1% (1 in 1000) chance of flooding occurring each year. The EA's website also shows that this area does not fall within an area at risk of flooding from reservoirs.

5.9 The Environment Agency (EA) published a new map of 'Flood Risk from Surface Water' in January 2014, and a more detailed version has since become available on the Government's 'Long Term Flood Risk Information' website, an extract of which is presented in Figure 5 below. This map identifies four levels of risk (high, medium, low and very low), and appears to be based primarily on topographic levels, flood depths and flow paths. The EA's definitions of these risk categories are:

'Very low' risk:	Each year, these areas have a chance of flooding of less than 1 in 1000 (0.1%).
'Low' risk:	Each year, these areas have a chance of flooding of between 1 in 1000 (0.1%) and 1 in 100 (1%)
'Medium' risk:	Each year, these areas have a chance of flooding of between 1 in 100 (1%) and 1 in 30 (3.3%).
'High' risk:	Each year, these areas have a chance of flooding of greater than 1 in 30 (3.3%).

5.10 The EA's modelling shows a 'Very Low' risk of surface water flooding for the entire site of No.79, the adjoining properties and along the Guilford Street carriageway outside the site. A 'Low' risk classification is given to a linear section of the Guilford Street footway outside No's 73 to 68, to the rear garden of No.73 and to various isolated pockets between hospital buildings on the south side of Guilford Street, with a small associated area of 'Medium' risk 35m south-east of No.79. The closest areas at 'High' risk of flooding are along Colonnade carriageway to the rear of the site, approximately 40m north-west of No.79. An extract of the EA's most recent model is presented in Figure 5.

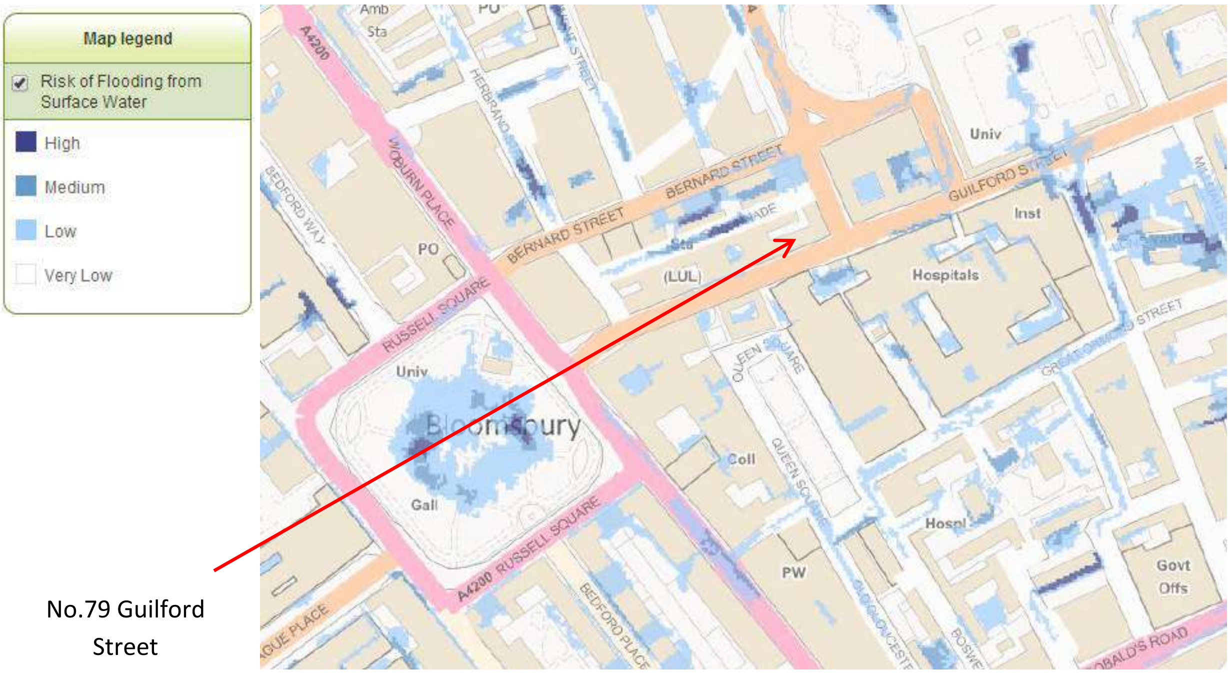


Figure 5: Extract from the Environment Agency's map of 'Flood Risk from Surface Water' map.

5.11 Surface water modelling has been undertaken by URS as part of a Strategic Flood Risk Assessment for the London Borough of Camden, and was published in July 2014; an extract from their model is presented in Figure 6. As per the Environment Agency modelling, this map identifies the same four levels of risk (high, medium, low and very low), and also shows a 'Very Low' risk of flooding for the site of No.79 and the surrounding area. The areas of 'Low', 'Medium' and 'High' risk are also similar to those identified by the EA, as described in 5.10.

5.12 Figure 6 also shows that Guilford Street falls within Critical Drainage Area Group3_003, but does not fall within a Local Flood Risk Zone (LFRZ).

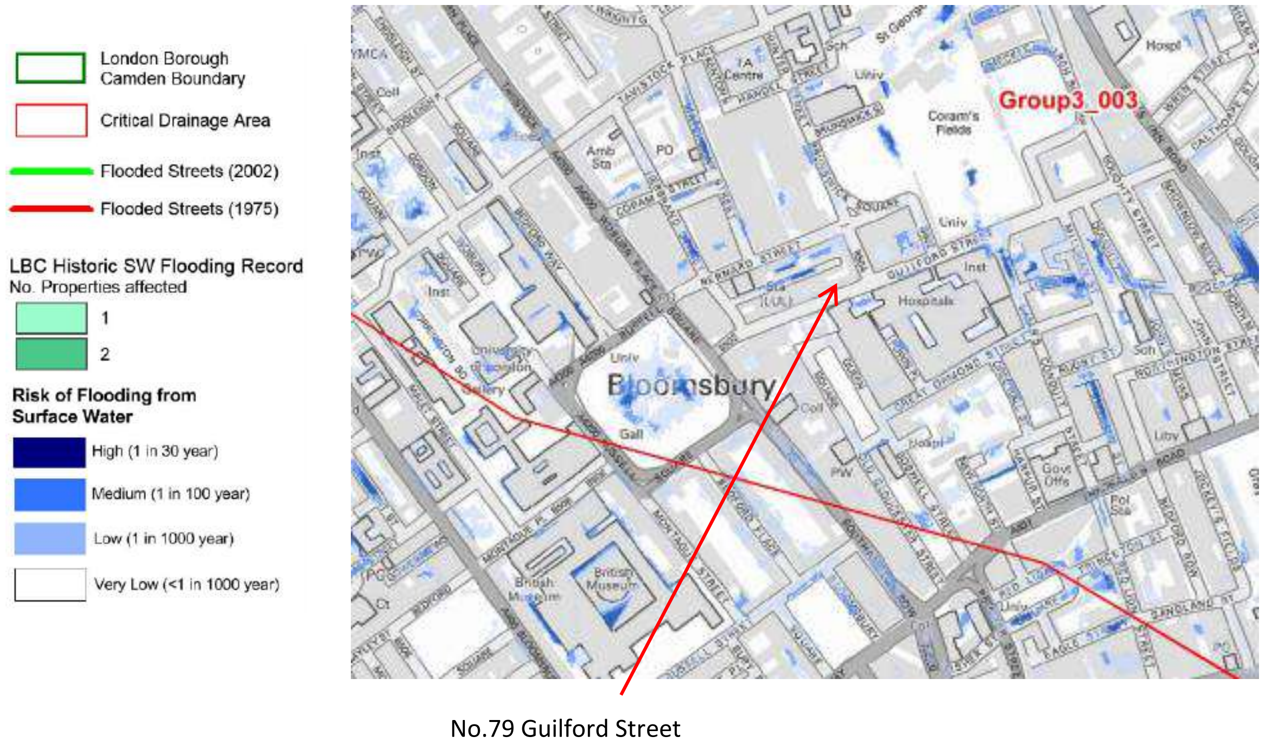


Figure 6: Extract from Figure 3i of the Camden Strategic Flood Risk Assessment (SFRA) (URS, July 2014) showing risk of flooding from surface water.

5.13 The implications from these flood models are discussed in Section 10.8.

5.14 Figures 5a & 5b of the Camden Strategic Flood Risk Assessment present historic records of internal and external sewer flooding respectively, based on Thames Water's DG5 Flood Register. These figures do not record any properties affected by internal or external sewer flooding within the 'WC1N 1' postcode (as of July 2014, when the SFRA was written), therefore a 'Sewer Flooding History Enquiry' has not been obtained from Thames Water Utilities Ltd (TWU). A report can be obtained on request if required.

6. HYDROGEOLOGICAL SETTING (GROUNDWATER)

6.1 The Lynch Hill Gravel Member is classified by the Environment Agency as a Secondary 'A' Aquifer and the underlying London Clay Formation is classified as an 'Unproductive Stratum', as indicated by Figure 7. Under the old groundwater classification scheme, which now applies only to superficial soils, the site is in an area which is classed as 'Minor Aquifer High' groundwater vulnerability.

Figure 7: Extract from Figure 8 of the Camden GHHS (Arup, 2010) showing aquifer designations.

6.2 Perched groundwater would typically be expected in any Made Ground, where underlain by strata of lower permeability such as clays within the River Terrace Deposits (RTDs), in at least the winter and early spring seasons. Variations in groundwater levels and pressures will occur seasonally and with other man-induced influences such as groundwater abstraction from wells or boreholes.

6.3 The Secondary Aquifers in the superficial River Terrace Deposits are collectively known as the 'Upper Aquifer'. The Upper Aquifer generally occurs in the lower part of the River Terrace Deposits (in this case the Lynch Hill Gravel Member), and it is possible that multiple areas of perched groundwater may be present above the main groundwater table in the Upper Aquifer.

6.4 While the London Clay Formation is classified as an 'Unproductive Stratum' it can still be water-bearing. The water pressures within the clay in the depths of current interest are likely to be hydrostatic, which means they increase linearly with depth, except where they are modified by tree root activity or the influence of man-made changes such as utility trenches (which can act either as land drains or as sources of water and high groundwater pressures). Any silt or sand partings, laminations or thicker beds are likely to contain free groundwater and, where these are laterally continuous, they can give rise to moderate water entries into excavations. In most cases there will be only very limited or no natural flow in these silt/sand horizons.

6.5 The Chalk Principal Aquifer, which occurs at depth beneath the London Clay, is not considered relevant to the proposed basement, so is not considered further.

6.6 The groundwater catchment areas upslope of No.79 are likely to differ for each of the main stratigraphic units:

- Made Ground: The catchment for any perched groundwater in the Made Ground is probably limited to the immediately adjoining areas of Made Ground, as well as infiltration within No.79's own garden where not hard surfaced or overlain by the plastic sheeting found in the external trial pits, except where the trenches for drains and other services provide conduits for water from a wider area.
- Lynch Hill Gravel Member: The catchment for the Upper Aquifer within the Lynch Hill Gravel Member will comprise recharge from both the overlying soils in the vicinity of the site and a wider subterranean area due to the expected lateral permeability.
- London Clay Formation: The catchment for the underlying London Clay will comprise predominantly recharge from the overlying aquifer in the vicinity of the site plus potentially a wider area determined by the lateral extent of any interconnected silt/sand horizons, though the contribution from the surrounding area is likely to be minimal given the general low permeability of the London Clay.

6.7 Other hydrogeological data obtained from the Groundsure EnviroInsight report (Appendix D) include:

- There are no Source Protection Zones (SPZ) within 500m of the site (Figure 8 above and App.D, Section 6.6 and 6.7)
- The nearest groundwater abstraction licence is 556m south-west of the site, this was an active licence for the London School of Hygiene and Tropical Medicine, on Keppel Street (borehole 1). There is another active licence at this site (557m south-west, for borehole 2) and two further historical licences; both the active licences were for heat pumps, so the water would have been circulated back to the aquifer; both expired on 31st March 2019, but should be assumed to have been renewed. There are 67 licences in total within 2000m of the site, 19 of which are still active licences (App.D, Section 6.3). All of these are probably irrelevant to the proposed basement extension.
- There are 33 abstraction licences for potable water within 2000m of the site, seven of which are active licences. The closest active licence is 1091m north-east of the No.79 (App.D, Section 6.5). These licences are also irrelevant for the proposed basement.
- The BGS has classified the area within 50m of the site as being susceptible to flooding at surface from superficial deposits. A moderate confidence rating has been provided by the BGS for the accuracy of this classification (App.D, Sections 7.7 & 7.8). The implications of this classification for the proposed basement are discussed in 10.3 below.

6.8 Groundwater records from local BGS boreholes are presented in Table 1. Boreholes TQ38SW/531 and ~/1021 both record groundwater strike/seepage within the sands and gravels of the River Terrace Deposits, with groundwater standing levels between 2.4m and 4.5m below ground level (bgl). Borehole TQ38SW/123 records groundwater strike within the London Clay at 13.7m bgl, but with the remark of "seepage from sandy pocket in the grey-blue clay".

6.9 Reference to the historic OS maps (presented in Appendix E) record the presence of two pumps at the southern end of Queen Square, roughly 180-190m south of No.79 on 1878 1:2,500 map. The 1896 1:2,500 map records these two and a further pump 200m south-west on Russell Square. These are absent for subsequent maps up to 1987, which records a single pump to the south of Queen Square Gardens. This same pump is recorded up to 1995, the most recent OS map published at this scale. These pumps probably extracted relatively small quantities of groundwater from the Upper Aquifer.

6.10 Details of the groundwater regime found by the site-specific ground investigation in February 2019 are presented in Section 9.

7. STAGE 1 - SCREENING

7.1 The screening has been undertaken in accordance with the three screening flowcharts presented in LBC's CPG Basements guidance document. Information to assist with answering these screening questions has been obtained from various sources including the site-specific ground investigation, the Camden geological, hydrogeological and hydrological study (Arup, 2010), historic maps and data obtained from Groundsure (see Appendices C, D & E) and other sources as referenced.

7.2 Subterranean (groundwater) flow screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1a	Is the site located directly above an aquifer?	Yes	Carried forward to Scoping: 8.2, Sections 10.2 & 10.3
1b	Will the proposed basement extend beneath the water table surface?	Yes, probably, given that groundwater was encountered at 0.5m below the floor of the front vault and at 19.16m AOD in BH1.	Carried forward to Scoping: 8.2, 9.9, 9.10, Sections 10.2 & 10.3
2	Is the site within 100m of a watercourse?	No - There are no surface water features within 250m of the site. The former minor tributary of the Fleet (Figure 4) has been culverted since 1800s.	5.1 & 5.6, and Figure 4
3	Is the site within the catchment of the pond chains on Hampstead Heath?	No – Site is in Bloomsbury	
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	Yes – the ground floor rear extension and proposed courtyards will cause a small increase in hard surfacing.	Carried forward to Scoping: 8.2, Section 10.8
5	As part of the site drainage, will more surface water (eg: rainfall and run-off) than at present be discharged to the ground (eg: via soakaways and/or SUDS)?	No – Soakaways would be inappropriate in London Clay (on which the basement extension is likely to be founded); mitigation to offset the possible small increase in hard surfacing will include SuDS, but volume of water discharged to ground will not increase.	
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 in any local pond (not just the pond chains on Hampstead Heath) or spring line?	Yes - There are no surface water features within 250m of the site, however Barton & Myers' map (Figure 4) indicates the possible spring for the former tributary of the Fleet is 150m south-west of No.79.	Carried forward to Scoping: 8.2

7.3 Slope/ground stability screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1	Does the existing site include slopes, natural or man-made, greater than 7°? (approximately 1 in 8)	No –Area is nearly level and fully developed	2.11, 2.12
2	Will the proposed re-profiling of landscaping at site change slopes at the property boundary to more than 7°?	No – No significant re-profiling is proposed.	
3	Does the development neighbour land, including railway cuttings and the like, with a slope greater than 7°?	No – Adjoining sites are also broadly level.	
4	Is the site in a wider hillside setting in which the general slope is greater than 7°?	No – Eastwards fall of Guilford Street is less than 1°	2.11, 2.12
5	Is the London Clay the shallowest strata at the site?	No - River Terrace Deposits are mapped at surface by the BGS.	4.1
6	Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree root protection zones where trees are to be retained?	Unknown - There is a semi-mature plane tree in the Guilford Street footway in front of No.78. This will not be felled but the development of the front vaults may be within the root protection zone.	Carried forward to Scoping: 2.4, 8.3, Section 10.4
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	Yes, minor cracking, past repairs and distorted walls were noted to No.79 and the adjoining properties.	Carried forward to Scoping: 2.4, 8.3, Section 10.4
8	Is the site within 100m of a watercourse or potential spring line?	Yes - see Q6 in subterranean flow screening above.	Carried forward to Scoping: 8.3
9	Is the site within an area of previously worked ground?	No – See BGS map extract (Figure 3 herein) and maps on pages 8 & 15 of the GeoInsight report (in App. C).	4.1
10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	Yes and Yes	Carried forward to Scoping: 8.3, Sections 10.2 & 10.3.
11	Is the site within 50m of the Hampstead Heath ponds?	No – Site is in Bloomsbury	
12	Is the site within 5m of a highway or a pedestrian right of way?	Yes	Carried forward to Scoping: 8.3, Section 10.4
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	Yes – The basement extension will increase the differential depths with the neighbouring rear gardens	Carried forward to Scoping: 8.3, Section 10.4
14	Is the site over or within the exclusion zone of any tunnels, eg railway lines.	No - No tunnels were identified by the services search below or close to the site.	

7.4 Surface flow and flooding screening flowchart:

Question		Response, with justification of 'No' answers	Clauses where considered further
1	Is the site within the catchment of the pond chains on Hampstead Heath?	No – Site is in Bloomsbury	
2	As part of the proposed site drainage, will surface water flows (eg volume of rainfall and peak run-off) be materially changed from the existing route?	No – Flow routes at surface should be unchanged.	
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	Yes – the rear extension at ground floor level and construction of rear garden courtyards will cause a small increase in hard surfacing	Carried forward to Scoping: 8.4 & Section 10.8
4	Will the proposed basement result in changes to the profile of the inflows (instantaneous and long-term) of surface water being received by the adjacent properties or downstream watercourses?	No – There is no run-off from the front and rear gardens to adjacent properties. The historic natural watercourse downslope of the property (Fleet tributary) is understood to be culverted.	5.1 to 5.8
5	Will the proposed basement result in changes to the quality of surface water being received by adjacent properties or downstream watercourses?	No – There should be no significant change in the surfaces generating run off. None of the surface run-off from this property goes directly to a watercourse.	5.1 to 5.5
6	Is the site in an area known to be at risk from surface water flooding, such as South Hampstead, West Hampstead, 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 lower part of the borough did not flood in 1975 or 2002; the site is in flood risk Zone 1 and surface water modelling by the Environment Agency and in the Camden SFRA does not indicate any increase in flood risk for the site above national background.	Section 5

7.5 Non-technical Summary – Stage 1:

The screening exercise in accordance with LBC's CPG has identified eleven issues which need to be taken forward to Scoping (Stage 2); four are related to groundwater, six are related to ground stability and one is related to flooding potential. The presence of perched groundwater in the clays of the Made Ground must also be allowed for in the design of the basement and the associated temporary works; these matters are considered in Sections 10.2 and 10.3.

8. STAGE 2 – SCOPING

8.1 The scoping stage is required to identify the potential impacts from the aspects of the proposed basement which have been shown by the screening process to need further investigation. A conceptual ground model is usually compiled at the scoping stage however, because the ground investigation has already been undertaken for this project, the conceptual ground model including the findings of the ground investigation is described under Stage 4 (see Section 10.1).

8.2 Subterranean (groundwater) flow scoping:

Issue (= Screening Question)		**Potential impact and actions**
1a	Is the site located directly above an aquifer?	**Potential impact:** Increased hard surfacing would decrease infiltration of surface water into the aquifer. See also 1b below. **Action:** Design appropriate landscaping in the courtyards
1b	Will the proposed basement extend beneath the water table surface?	**Potential impact:** If the basement extends below the groundwater table it might affect groundwater levels and flows; will require increased waterproofing measures and would create an uplift force on the basement. **Action:** Ground investigation required; then impact assessment and appropriate design for both permanent basement structure and temporary groundwater control measures.
4	Will the proposed basement development result in a change in the proportion of hard surfaced/ paved areas?	**Potential impact:** Increased hard surfacing would decrease infiltration of surface water into the ground. **Action:** Review potential impacts of proposed changes, including appropriate types of SuDS for use as site-specific mitigation where relevant (ie: where reduced infiltration would be a problem).
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 in any local pond (not just the pond chains on Hampstead Heath) or spring line?	**Potential impact:** Risk of inundation of the basement and/or excavation. Temporary dewatering and/or the permanent works might lower the 'pond' water level (although unlikely in this case, as the tributary of the River Fleet is understood to be culverted), or the basement's cavity drainage system could experience excessive flows. **Action:** Review hydrogeology of the site, undertake a ground investigation, and recommend appropriate site-specific mitigation where relevant.

8.3 Slope/ground stability scoping:

Issue (= Screening Question)		**Potential impact and actions**
6	Will any tree/s be felled as part of the proposed development and/or are any works proposed within any tree root protection zones where trees are to be retained?	**Potential impact:** Heave from removal of trees; slope(s) become less stable; damage to trees. **Actions:** Arboricultural report required, potential implications to be assessed once root protection zone for pavement tree (in front of No.78) is identified.
7	Is there a history of seasonal shrink/swell subsidence in the local area, and/or evidence of such effects at the site?	**Potential impact:** Weakened structures from past movement would be more susceptible to damage during works. Future differential movement between No.79 and adjoining No's 78 & 80 once the proposed basement has been constructed. **Action:** Review potential impact of seasonal water content changes in the clays, and any planned vegetation removal or vegetation growth. Designer and contractor to take account of any weakening of the structure caused by past movements.
8	Is the site within 100m of a watercourse or potential spring line?	**Potential impact:** No impact on slope stability is expected as there are no significant slopes in the vicinity. Dewatering during construction might cause settlement. **Action:** Review hydrogeology of the site, undertake a ground investigation, and recommend appropriate site-specific mitigation where relevant.
10	Is the site within an aquifer? If so, will the proposed basement extend beneath the water table such that dewatering may be required during construction?	**Potential impact:** Dewatering increases the effective stress in the ground and may remove fines, both of which can cause settlement of the affected area. **Action:** Ground investigation required, then appropriate design of groundwater control.
12	Is the site within 5m of a highway or a pedestrian right of way?	**Potential impact:** Construction of basement causes loss of support to footway/highway and damage to the services beneath them. **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods.
13	Will the proposed basement substantially increase the differential depth of foundations relative to neighbouring properties?	**Potential impact:** Loss of support to the ground beneath the foundations of the adjoining No.78 & No.80 (and No.5 Colonnade) if basement/rear extension excavations are inadequately supported. Possible long-term differential movement. **Action:** Ensure adequate temporary and permanent support by use of best practice underpinning methods. Consider the need for transition underpinning.

8.4 Surface flow and flooding scoping:

Issue (= Screening Question)		**Potential impact and actions**
3	Will the proposed basement development result in a change in the proportion of hard surfaced / paved external areas?	**Potential impact:** May increase flow rates to sewer, and thus increase the risk of flooding (locally and elsewhere). May change infiltration. **Action:** Assess net change in hard surfaced/ paved areas and review appropriate types of SuDS for use as site-specific mitigation.

8.5 Non-technical Summary – Stage 2:

The scoping exercise has reviewed the potential impacts for each of the items carried forward from Stage 1 screening, and has identified the following actions to be undertaken:

- A ground investigation is required (which has already been undertaken, the results of which are presented in Section 9).
- An arboricultural report is required to identify and assess the potential implications of the pavement tree in front of No.78 Guilford Street.
- Appropriate types of Sustainable Drainage System (SuDS) should be reviewed in order to offset (mitigate) the possible small increase in the area of hard surfacing.
- Designer and contractor to take account of any weakening of the structures caused by past movements.
- Ensure adequate temporary and permanent support by use of best practice underpinning methods.
- Consider the need for transition underpinning to mitigate differential foundation depths between No.79 and No's 78 & 80.
- Review flood risk and include appropriate flood resistance and mitigation measures in the scheme's design.

All these actions are covered in Stage 4, or Stage 3 for the ground investigation.

9. STAGE 3 – GROUND INVESTIGATION

9.1 A site-specific ground investigation was undertaken on 19th February 2019, and comprised one 'windowless' sampler borehole (BH1) drilled to a depth of 6.0m below ground level (bgl) within the front lightwell and six hand dug trial pits (TPs 1-6). Logging of the recovered continuous 'core' samples from the 'windowless' sampler and the trial pit excavations were undertaken on site by Gabriel GeoConsulting Ltd (Alexander Goodsell and Heather Baker). The factual findings from the investigation are presented in Appendix F, including an exploratory hole location plan (Figure GI-01), borehole log (GI-02), trial pit logs (GI-03 to GI-10) and laboratory test results.

9.2 Trial pits TP1 to TP6 were dug in order to investigate the foundations to No.79, and the soils beneath the footings, at their respective locations. The findings from these trial pits may be summarised as follows:

TP1:	<u>Section A</u> Location: Rear lightwell, adjacent to No.79/80 party wall. Pit Depth: 1.00m below basement level (bbl) Materials: Brickwork, over brick rubble concrete (0.65m thick) Footing depth = 0.95m bbl; projection = 0.20m Geology: 0.02m wooden decking over 0.05m void over 0.03m paving slabs; over 0.6m MADE GROUND (*gravelly, clayey SAND*) over LYNCH HILL GRAVEL MEMBER (*gravelly SAND*) to base of pit. <u>Section B</u> Location: Rear lightwell, rear wall of No.79 basement lightwell. Pit Depth: 1.00m below lower ground floor level (blgl) Materials: Brickwork Footing depth = 0.35m bgl; no projection Geology: As Section A
TP2:	Location: Basement 'steam room', adjacent to No.79/78 party wall. Pit Depth: 0.60m below basement level (bbl) Materials: Brickwork (?), over two layers of concrete (total 0.3m thick) Footing depth = 0.35m bbl; projection = 0.25m Geology: 0.02m wooden decking over 0.02 - 0.03m tiles (variably with 0.01m void), over two layers of concrete, each 0.1m thick, over LYNCH HILL GRAVEL MEMBER (*gravelly SAND)* to base of pit.
TP3:	<u>Section A</u> Location: Rear garden, adjacent to No.79/80 party wall. Pit Depth: 1.25m below ground level (bgl) Materials: Brickwork Footing depth = 1.20m bgl; no projection Geology: Pea gravel & plastic membrane over 0.70m MADE GROUND (*firm, slightly gravelly, variably sandy CLAY*) over brick rubble in CLAY matrix to base of pit.

	<u>Section B</u> Location: Rear garden, adjacent to No.79/80 boundary wall. Pit Depth: 0.75m below ground level (bgl) - the shallower footings beneath the boundary wall meant this section of the pit was not excavated to the same depth as Section A. Materials: Brickwork (with one corbel) over concrete (0.37m thick) Footing depth = 0.70m bgl; projection = 0.15m Geology: As Section A.
TP4:	Location: Adjacent to No.79/78 party wall at ground floor level. Pit Depth: 1.10m below ground floor level (bgfl) Materials: Brickwork, over brick rubble concrete (0.20m thick) Footing depth = 1.05m bgfl, projection = 0.07m Geology: Pea gravel and plastic membrane, over 0.25m MADE GROUND (*loose, friable, sandy, silty, clayey LOAM*), over 0.20m 'transitional zone', over MADE GROUND (*firm, variably sandy, gravelly to very gravelly CLAY*) to base of pit.
TP5	Location: Rear garden, adjacent to No.79/5 Colonnade boundary wall. Pit Depth: 1.17m below ground level (bgl) Materials: Brickwork (with one corbel), over brick rubble concrete (0.62m thick) Footing depth = 1.12m bgl, projection = 0.30m Geology: Pea gravel and plastic membrane, over 0.30-0.40m topsoil (*friable sandy, silty, clayey LOAM*) over 0.52-0.62m MADE GROUND (*sandy, silty, clayey LOAM*), over MADE GROUND (*firm, silty CLAY*) to base of pit.
TP6	Location: Flank wall of front vault. Pit Depth: 0.50m below basement level (bbl) Materials: Brickwork (with two corbels) Footing depth = 0.45m bgl, projection = 0.12m Geology: 0.06m brick paving over 0.01m 'sharp' sand, over 0.05m brick paving (possibly an old floor?) over 0.01m bed of ash, clinker and sand; over firm, gravelly CLAY (LYNCH HILL GRAVEL MEMBER/possible HEAD DEPOSITS(?) () to base of pit. Half of pit adjacent to footing filled with MADE GROUND (loose CLAY with brick fragments), from base of upper brick paving to 0.45m.

9.3 All six of the trial pits excavated revealed Made Ground directly beneath the surfacing/floor structure, with the exception of TPs 2 and 6, which respectively found the Lynch Hill Gravel Member (LHGM) directly beneath 0.2m concrete, and natural clays/possible Head Deposits beneath the current and former floor structures. The Made Ground was found to vary across the site, generally consisting of either gravelly, clayey SAND (TP1); variably gravelly, sandy CLAY (TP3 & deeper TP4); sandy, silty, clayey LOAM (TP4 & 5) or firm silty CLAY (deeper TP5). All contained a range of artificial materials including fragments of brick (and half bricks), concrete, flint, mortar, slate, sandstone and glass.

9.4 The three trial pits undertaken at ground floor level terminated within the Made Ground, whereas the three trial pits undertaken at basement level (TPs 1, 2 & 6) all encountered natural ground. TPs 1 & 2 record gravelly, coarse (locally medium) SAND of the LHGM, and TP6 found firm, gravelly CLAY. This could be possible Head Deposits, or a clay horizon within the LHGM.

9.5 The site's geology as found in BH1 may be summarised as below. BH1 was drilled at approximately 20.63m above ordnance datum (AOD, taken from Edward Gardner Surveys' 'Existing Lower Ground Floor', Drg No. 18-007-2) in the front lightwell.

- Made Ground: Recorded beneath 0.05m of (pea gravel over) concrete to 1.30m bgl (19.33m AOD), the Made Ground was found to comprise "*soft to firm, slightly moist, light to mid-brown with rare dark grey mottling, sandy to very sandy, gravelly to very gravelly CLAY*". The gravel was found to be fine to coarse with occasional cobbles, and primarily of concrete, brick, flint and chalk and other assorted artificial fragments. Below 1.00m bgl, the Made Ground changed in nature and was found to comprise "*medium dense, moist to very moist, grey to dark grey and brown, very clayey, very gravelly SAND/very sandy GRAVEL*", with gravel predominately fine to medium and of "*sub-rounded to rounded flint pebbles and angular brick fragments*".
- Lynch Hill Gravel Member (?): Recorded from the base of the Made Ground (1.30m) to 1.60m bgl (19.03m AOD), this River Terrace Deposit comprised "*medium dense, very moist, multi-coloured, slightly clayey to locally clayey, very gravelly SAND*". The gravel was found to be "*fine to coarse, sub-angular to rounded flint*". This is taken to be the Lynch Hill Gravel Member (LHGM), as mapped by the BGS on site.
- Disturbed (?) London Clay Formation: Possibly disturbed London Clay was found underlying the LHGM to a depth of 4.34m bgl (16.29m AOD), which comprised "*stiff, closely fissured, brownish grey to grey, slightly silty CLAY*". Occasional claystone nodules and fractured claystone horizons were found between 2.40-2.55m, 3.40-3.44m, 3.49-3.55m and 3.67-3.75m bgl. The slightly open fissure surfaces and residual clasts encountered below 2.70m gave a 'blocky' texture to the soil. Below 3.75m, well-developed curved/dished clay mineral alignment with "*occasional open fissures, fissure surfaces rarely slightly polished*" was found.
- London Clay Formation, possibly disturbed: Recorded from the base of the disturbed LCF at 4.34m bgl to the base of BH1 at 6.00m bgl (14.63m AOD), the London Clay was recorded as "*very stiff, closely to very closely fissured, brownish grey CLAY*" with "*well-developed, curved/dished mineral alignment with frequent open, rarely slightly polished, fissures*". Rare selenite crystals and occasional fine sand partings were also found.

9.6 Standard Penetration Tests (SPTs) were carried out in BH1 at one metre intervals between 1.00m and 6.00m. The resulting 'N' values (blows to drive the 300mm test length, after 150mm of 'seating' driving) are recorded on the Standard Penetration Test Results sheet and at the relevant depths on the borehole log (Figure GI-02 in Appendix F), and have also been plotted as a profile against depth in Figure 8 below. The SPT values were found to be very low within the Disturbed (?) London Clay from 2-4m bgl; however, within the deeper London Clay at 5-6m bgl, they show an overall trend of increasing shear strength with depth, as is typically found in this stratum.

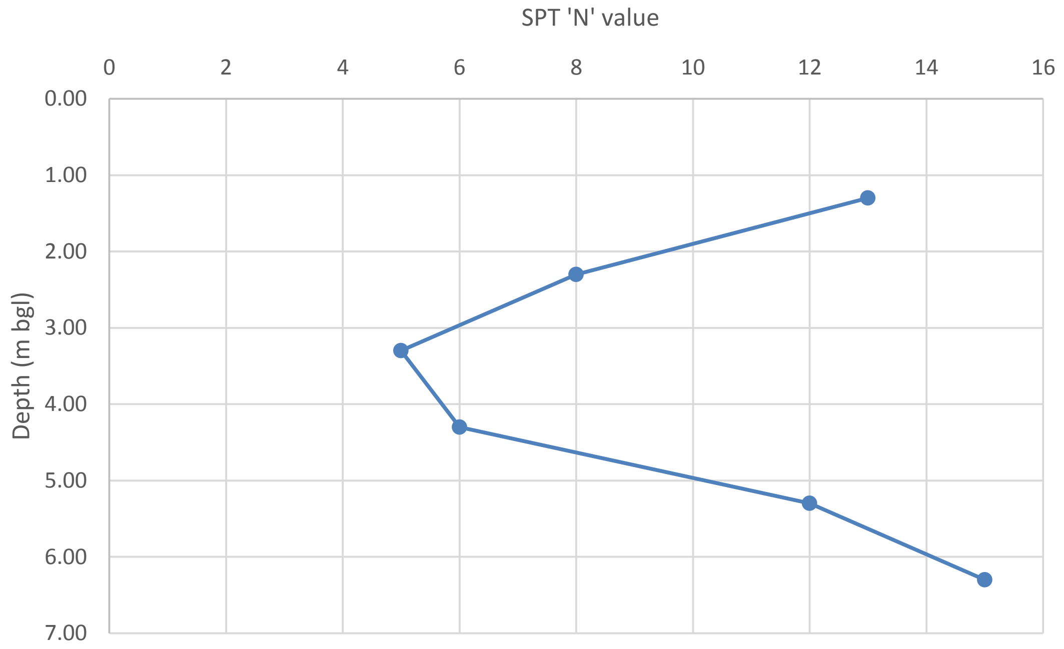


Figure 8: SPT 'N' values with depth

9.7 Measurements of shear strength were taken in all the clays within the recovered 'cores' throughout BH1 using a dial gauge type pocket penetrometer. These values have been recorded at the relevant depths on the borehole log (Figure GI-02 in Appendix F), and the averages of these values have been plotted as a profile against depth in Figure 9 below. As with the SPT results, the results show lower shear strength values within the Disturbed (?) London Clay and, with some variation, below 3.5m bgl indicate an overall increase in shear strength with depth. It should be noted that these tests do not allow for the influence of any fissures, so tend to overestimate the strength of the clays.

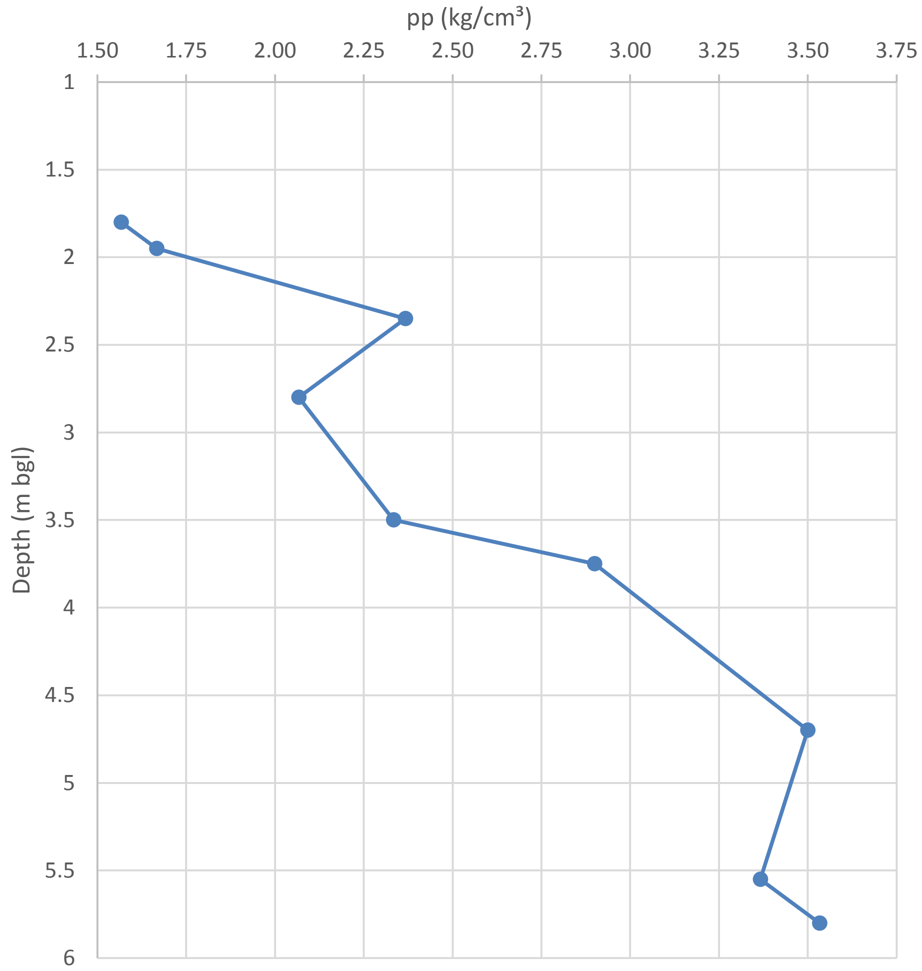


Figure 9: Average penetrometer (pp) readings with depth.

9.8 Roots were observed in TPs 3 & 5 (both located within the rear garden), and a single dead root with a diameter of 1.5mm was found at 5.60m bgl in BH1. Roots in TP3 were described as "*frequent live roots (<3mm) and occasional roots (<4mm) to base of pit*" at 1.25m bgl, and in TP5 were described as "*very frequent roots (25mm - nearly tree trunk, rest are <4mm)"* to a depth of 0.40m bgl. No roots were recorded in TPs 1, 2, 4 or 6.

9.9 A groundwater strike was recorded at 2.50m bgl (18.13m AOD) within a claystone horizon and, on completion of drilling of BH1, the groundwater level in the borehole was standing at 1.52m bgl (19.11m AOD). On completion of TP6 (front vault), standing groundwater was recorded in the base of the pit. No groundwater entries were recorded in the remaining trial pits, although the Made Ground in TP3 was recorded as "*slightly moist*" and the Lynch Hill Gravel Member in TP1 (below 0.70m bgl) was recorded as "*moist, almost wet*" and in TP2 (below 0.25m bgl) was recorded as "*moist*".