Pell Frischmann

Longhurst Road, Blackley

Geotechnical Interpretative Report

April 2023

Report Ref: 106240-PEL-GEN-XX-RP-D-0002

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Appendix A – Proposed Development Drawings

Executive Summary

Site Name and Location	Longhurst Road, Blackley, Manchester
Development Proposals	The proposed development comprises construction of 9 No. blocks of new residential dwellings which are up to three storeys in height built into the existing slope. To provide level platforms, the associated earthworks includes up to 4.5m of cut across the central portion of the site, and nominal filling (0.5m to 1.0m) across the central southern portion
Ground Investigation	A ground investigation was undertaken in January 2023 and comprised 3 nr. windowless sampler boreholes to a maximum depth of 6.05 m bgl, 4 nr. machine excavated trial pits to a maximum depth of 4.0 m bgl, geotechnical and geoenvironmental soil laboratory testing, soakaway testing and ground gas and groundwater monitoring.
Ground Conditions	

Stratum	West, TP01, TP02, WS01		East, TP03, TP04, WS02, WS03	
	Top of Stratum m bgl (m aOD)	Thickness m	Top of Stratum m bgl (m aOD)	Thickness m
Topsoil	0.0 (90.4)	0.40		
Made Ground	-	-	0.0 (88.9)	1.0
Glacial Till	0.40 (90.0)	5.65+	1.0 (87.9)	5.05+

Groundwater Conditions	Western Portion of Site: typical/average groundwater level = 1.50m bgl (88.9mOD) Eastern Portion of Site: typical/average groundwater level = 2.00m bgl (87.0mOD)
Foundation Options	Shallow foundations are considered to be a suitable foundation type for the proposed development given the size of the proposed structures and the anticipated loadings, either as pad/strip footings, or rafts. Foundations should consider the effects of soil volume change resulting from excavation and the removal of vegetation.
Void Former	To prevent heave effects impacting the ground floor slabs, void former will be required under most of the floor slabs, where suspended over shallow strip or pad footings. A raft foundation may be used to counteract heave movements and remove the need for void formers, subject to detailed design.
Retaining Walls	Two retaining walls are required as part of the development, the Primary Retaining Wall is up to 3.70m in height, the Secondary Retaining Wall is up to 2.0m in height. For wall heights in excess of 3.0m, a piled wall is likely to be required, with steel king posts. For the smaller wall up to 2.0m, gravity retaining walls, or reinforced concrete walls would be suitable. Benching of the slope behind the Primary Wall at its highest point (3.70m) could reduce the height of the wall significantly.
Buried Concrete	A concrete mix design sulphate class of DS-1, with an aggressive chemical environment for concrete class (ACEC) of AC-1.
Unexploded Ordnance	A risk assessment in accordance with CIRIA Report C681 indicates the risk is Medium, and that a detailed risk assessment should be undertaken ahead of the site works.
Cut Slopes	Cut slopes behind the proposed building blocks should incorporate drains to prevent shallow instability and water ponding in the void. A maximum angle of 1V:2.5H should be adopted for cut slopes up to 4.50m in height.
Earthworks	The soils on site were classified as low to intermediate plasticity and therefore are likely suitable for reuse provided moisture contents are within the optimum bounds and adequate compaction is achieved. If earthworks are conducted during the summer months controls on moisture content will be more readily available for soils that had moisture contents wetter of optimum. If earthworks are proposed during the winter, stabilisation of soils with lime or cement stabilisation may be required.
Onsite Infiltration	Infiltration testing undertaken within the Glacial Till soils recorded insufficient infiltration to calculate an infiltration rate, therefore it is not considered suitable for onsite infiltration
Further Work	Detailed design should be undertaken once structural loads are confirmed to determine foundation depths and layouts, considering the possible effects of soil volume change. Retaining wall geometry should be reviewed with the goal to identify areas where cut slopes can be used to reduce retaining wall heights. Retaining wall types should be confirmed and preliminary design undertaken to inform the cost plan.

1 Introduction

1.1 Appointment and Scope

This Geotechnical Interpretative Report (GIR) for the proposed Longhurst Road development has been prepared by Pell Frischmann (PF), acting under instruction from Great Housing Places. The GIR follows on from:

i. A Phase 1 Preliminary Risk Assessment issued by Curtins in April 2021;
ii. A preliminary retaining wall optioneering report issued by PF in June 2022;
iii. An intrusive site-specific ground investigation (GI) scheduled by PF, the fieldwork for which was undertaken in December 2022.

Further to the background geotechnical and geoenvironmental information collection set out in the desk study, the objectives of this report are as follows;

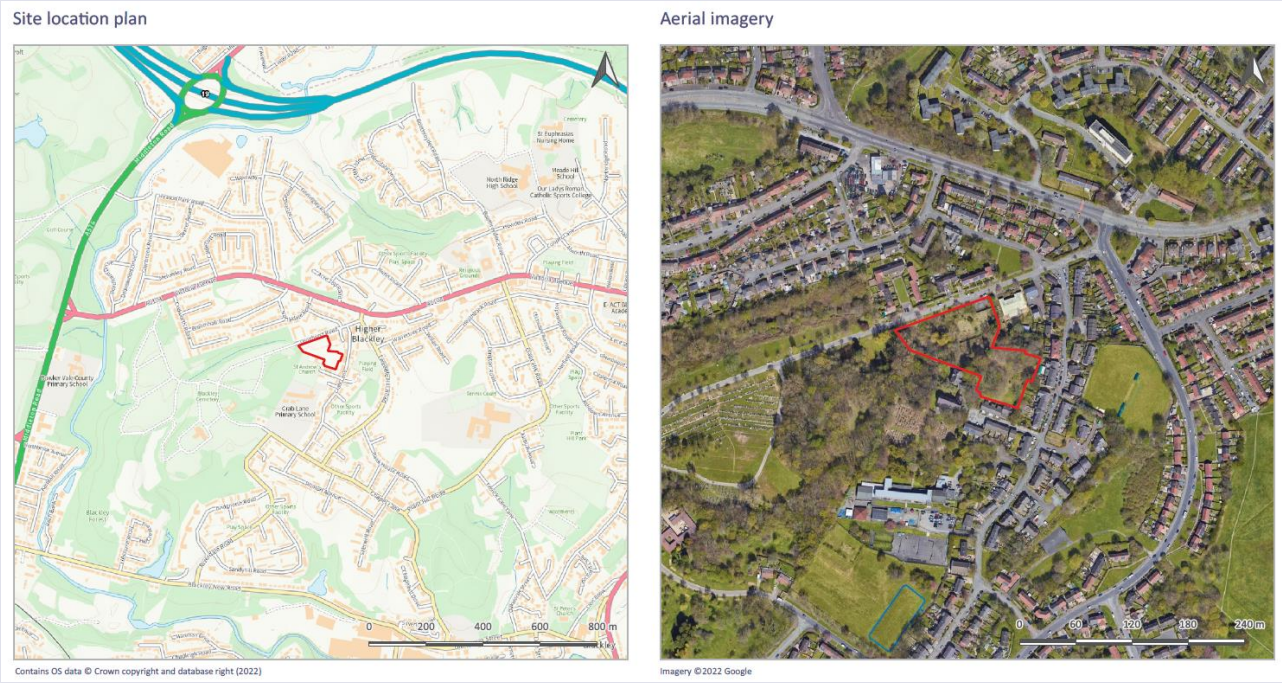
- Review of the history of development on the site and surroundings,
- Assess the local geology, hydrogeology and surface water environment, and identify potential geotechnical risk.
- Provide an interpretation of ground conditions and data from the intrusive investigation works.
- Provide preliminary recommendations with respect to geotechnical parameters, foundations and infrastructure design.

The geoenvironmental conditions are presented in the GQRA, Ref: *106240-PEF-XX-XX-RP-GG-600001*.

1.2 Site Location and Description

The site of the proposed development is located approximately 6.0km north of the Manchester City Centre along Longhurst Road, the site is currently unoccupied and located at approximate postal address M9 8NS. For this report, the bounds of the development area, hereafter referred to as 'site' are as shown in **Figure 1** below:

Figure 1: Site Location Plan



The site is irregular in shape, approximately 0.82 Ha and is located south of Longhurst Road, in Upper Blackley at grid references 384800, 403930. The site is level over the hardstanding area in the north eastern section, and then slopes from approx. 89.0mOD to 98.9mOD at the south eastern edge of the site. The north-eastern portion of the site comprises an area of level hardstanding bound by an existing concrete retaining wall (approx. 2.0m in height). The remainder of the site is currently grassed and undeveloped with medium to dense vegetation. The approximate locations of existing retaining walls on site are shown in **Figure 2**, the extents of these have been determined from survey information and aerial mapping, both walls are described as brick, the central wall (outlined in green) retains up to 1.5m in height, and the western boundary wall (outlined in blue) height was not determined as part of the topographic survey.

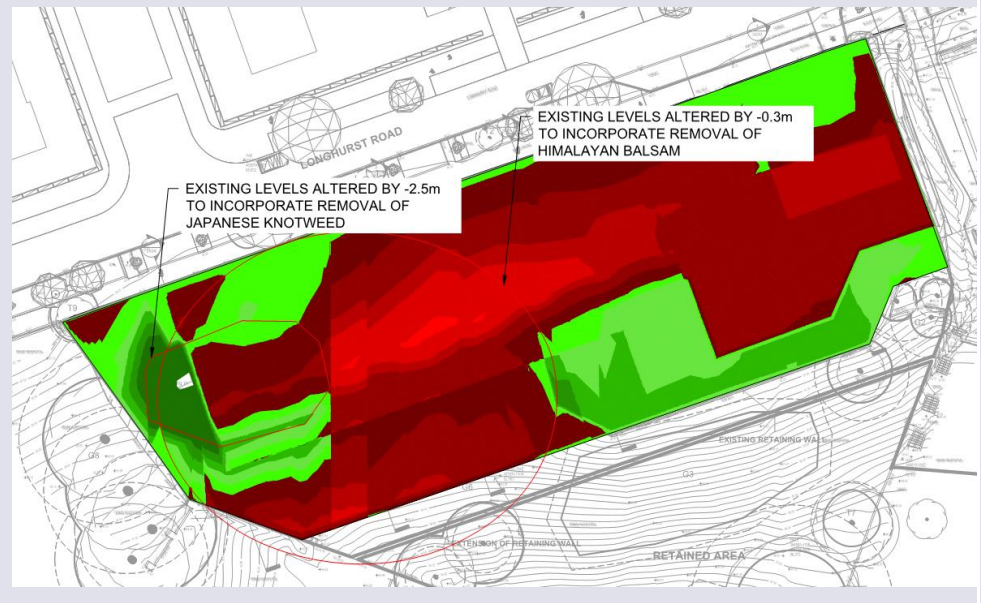
Figure 2: Approximate Extents of Existing Retaining Walls

1.3 Proposed Development

Although the site boundary extends upslope at the south eastern corner of the site, the proposed site works are confined to within the central existing retaining wall (green in **Figure 2).**

The proposed development comprises construction of 9 No. blocks of new residential dwellings which are up to three storeys in height built into the existing slope. To provide level platforms, the associated earthworks includes up to 4.5m of cut across the central portion of the site, and nominal filling (0.5m to 1.0m) across the central southern portion, the proposed cut and fill levels are shown in **Figure 3**.

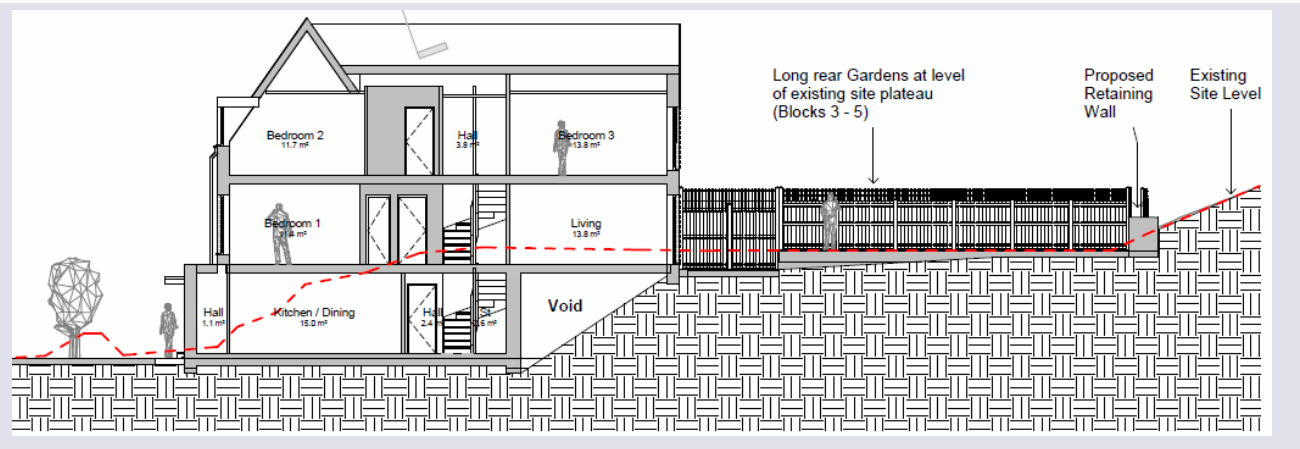
Figure 3: Proposed Cut and Fill



MINIMUM LEVEL	MAXIMUM LEVEL	COLOUR
-4.50	-4.00	
-4.00	-3.50	
-3.50	-3.00	
-3.00	-2.50	
-2.50	-2.00	
-2.00	-1.50	
-1.50	-1.00	
-1.00	-0.50	
-0.50	0.00	
0.00	0.50	
0.50	1.00	
1.00	1.50	
1.50	2.00	
2.00	2.50	
2.50	3.00	

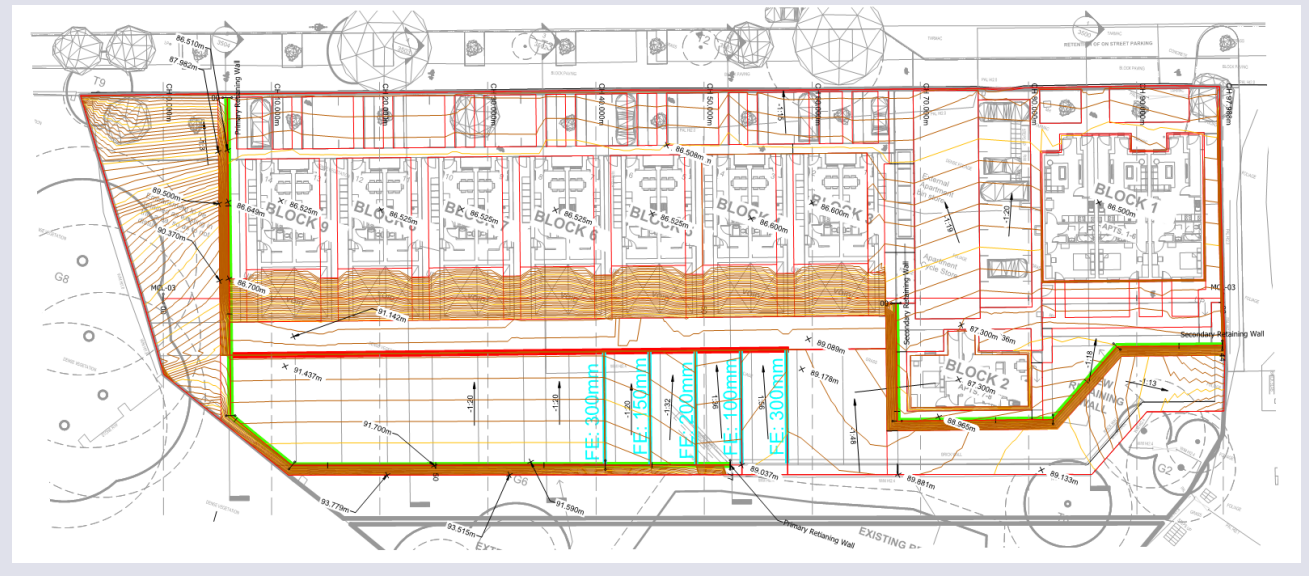
Architectural images of the proposed development are shown below in **Figure 4**, which shows how the structures are proposed to be constructed into the existing slope, no basement wall is shown instead a void with a permanent slope is proposed.

Figure 4: Architectural Render of Proposed Development (Bowker Sadler Architecture)



The finished floor level at Longhurst Road varies between 86.5mOD and 86.6mOD, with the 7 No. western blocks separated by a paved parking area in the north western portion of the site. Block 2 is situated on the upslope terrace at a finished floor elevation of 87.30mOD. The existing central retaining wall will be extended by a new retaining wall running further west to the site boundary (shown as *Primary Retaining Wall* in light green on **Figure 5**) which up to 2.10m of soil when running east to west, then returns back to the level of Longhurst Road, retaining up to 3.70m along this section where the existing soil is retained to avoid disturbing the Root Protection Area of adjacent trees. A secondary retaining wall runs from the eastern corner of the site to behind Block 2 (shown as *Secondary Retaining Wall* in light green) retaining up to 2.0m.

Figure 5: Proposed Block Layouts



1.4 Sources of Information

The following available information in **Table 1** was reviewed and used within the preparation of this report.

Table 1: Previous Information

Report Title and Reference	Date	Author
Phase 1 Preliminary Risk Assessment, 077873-CUR-00-XX-RP-GE-005	April 2021	Curtins
Geotechnical Design Options, 106240-PEL-GEN-XX-RP-D-0001	June 2022	Pell Frischmann
Tree Survey	June 2022	Mulberry Tree Management
Ground Investigation Factual Report	April 2023	Soiltechnics

1.5 Limitations

This report shall be read in conjunction with the relevant drawings, reports and specifications produced by PF and the rest of the Design Team. The information and recommendations contained within this report shall not be used by or passed on, in any form, to unauthorised persons.

In addition, recommendations and design parameters established in this report apply to this proposed development. Changing the proposed scheme may require reviewing the assessment methodology and conclusions provided within this report. PF has endeavoured to assess all information provided to them but makes no guarantees or warranties as to the accuracy or completeness of this information.

2 Existing Information

Key details from the above reports are summarised in the following sections.

2.1 Curtins - Phase 1 Preliminary Risk Assessment (April 2021)

Curtins carried out a Phase 1 Risk Assessment for the Site in April 2021, this assessment intended to provide an overview of the geo-environmental and geotechnical setting of the site, and develop a working conceptual and Ground Model and present an initial assessment of any risks that could be presented to the development, including its intended end users and the wider environment.

Key sections of that report are repeated in the following sections.

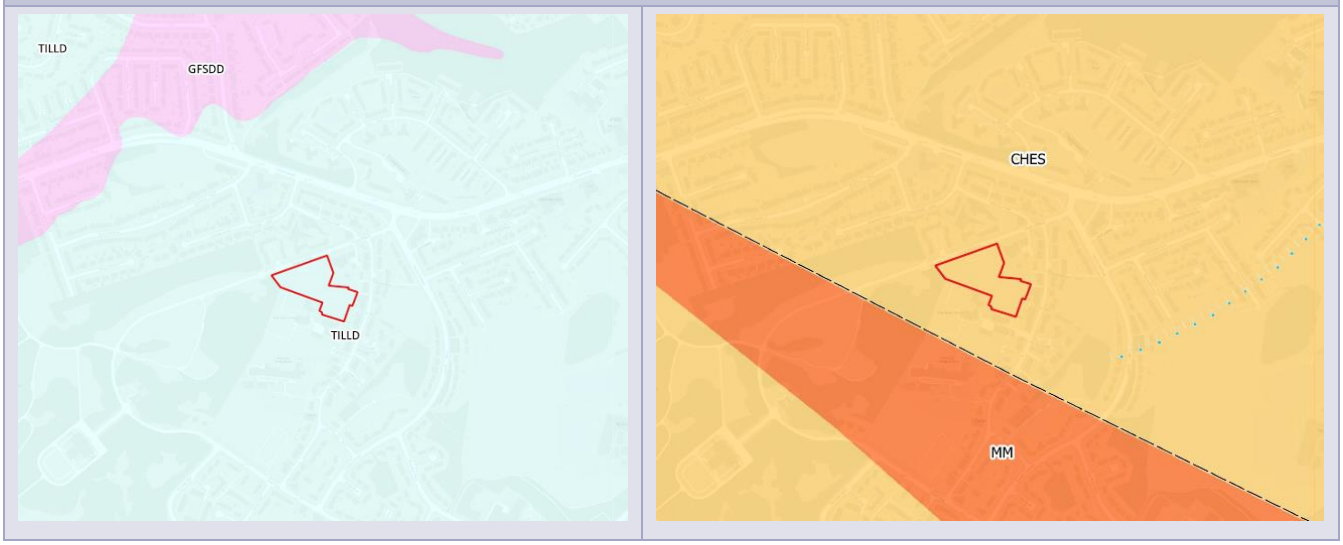
2.1.1 Site History

From earliest available mapping (1848) the site comprises two parcels (majority of the site and north-eastern parcel) of undeveloped land with woodland present. A footpath is present north to south across the site. In later 1909 mapping, the site has been cleared of woodland. In addition, a small structure is located within the north-eastern parcel, however has been removed by the 1930s and later replaced with a glass structure. In 1980s mapping the northern area comprises a Day Nursey, with parking and retaining structure located within north-eastern corner. The remainder of site is split into two parcels of undeveloped land. Between 2006 and 2020, the day nursey has been demolished on-site and site is within current configuration with hardstanding area and retaining wall remaining in the north-eastern corner. Throughout the site's history, its local surroundings are generally occupied by residential properties with limited contaminative potential.

2.1.2 Geological and Hydrogeological Mapping

BGS mapping indicates the site is underlain by Glacial Till Deposits (TILLD) superficial deposits, and Chester Formation (CHES) bedrock as shown in **Figure 6**. Outside of the site extents mapped are Glaciofluvial Sheet Deposits (GFSD) and Manchester Marls Formation (MM).

Figure 6: BGS Mapping, Superficial Deposits (Left) and Bedrock (Right)



The strata and aquifer are described in Table 2 below.

Table 2: Geological Description and Hydrogeological Classification		
Strata	Description	Aquifer Classification
Glacial Till	A heterogenous mixture of clay, sand, gravel, and boulders varying widely in size and shape.	Secondary Aquifer
Chester Formation	Sandstone bedrock formed between 250 and 247.1 million years ago during the Triassic period.	Principal Aquifer

The site is not situated within a Source Protection Zone (SPZ). There are no groundwater abstraction points within 500m of the site.

2.1.3 Hydrology

The nearest surface water feature is a brook located 47m west. There are no surface water abstractions within 1km. There are no discharge consents located within 500m of the site.

2.1.4 Mining Risk

The site is located within a Coal Mining Reporting Area, however it is not located within a Development High-Risk Area. There are no recorded mine entries within 1km of the site and there are no recorded coal workings and no probable coal workings within 1km of the site. Therefore the risk is considered to be Low.

2.1.5 Unexploded Ordnance (UXO)

The risk of encountering potential UXO as part of future intrusive works is considered Moderate.

2.2 Pell Frischmann - Geotechnical Design Options (June 2022)

A preliminary options assessment was undertaken to determine possible types of retaining walls for the proposed development.

The assessment concluded that King Post walls would likely be most suitable for the proposed retaining wall at the western portion of the site, and either a King Post wall or Mechanically Stabilised Earth would be suitable for the secondary retaining wall at the eastern portion of the site, refer **Figure 5.**

2.3 Soiltechnics – Ground Investigation (April 2023)

Soiltechnics were employed by Great Places Housing Group to carry out a PF-scheduled project-specific GI. The scheduled fieldwork took place from the 13th to 14th December 2022 and included the following:

- 3 nr. windowless sampler boreholes, to a maximum depth of 6.05 m bgl with regular SPTs.
- 4 nr. Machine excavated trial pits, to a maximum depth of 4.0 m bgl with pocket penetrometer tests;
- Geotechnical soil laboratory testing comprising:
    - 12 nr. moisture content tests;
    - 12 nr. Atterberg limit tests;
    - 6 nr. wet sieve PSD tests;
    - 7 nr. BRE SD1 suite D (brownfield – pyrite present).
- Geo-environmental soil laboratory analysis this testing is described and interpreted within the associated PF Phase 2 land contamination assessment report, Ref: *106240-PEF-XX-XX-RP-GG-600001*.
- Soakaway testing.
- Ground gas and groundwater monitoring installations with 5 nr. return monitoring visits.
- Full time UXO specialist supervision.

The locations of the completed exploratory holes are shown in **Figure 7** below.

Figure 7: Soiltechnics Exploratory Location Plan

The elevations of the investigations were not available from the factual report, therefore for this assessment have been estimated based on the topographic survey of the site, see **Table 3**.

Table 3: Investigation Details and Made Ground Thickness

Investigation	Top of Hole (mOD)[1]	Termination Depth (m bgl)	Made Ground Thickness	Installation	In-situ Testing
WS01	90.5	6.05	-	Groundwater and ground gas	SPT, PP
WS02	88.9	6.05	1.00	-	PP
WS03	89.0	5.45[4]	0.90	Groundwater and ground gas	PP
TP01	90.3	0.90[2]	0.90+[2]	-	Soakaway, PP
TP01A	90.3	2.50	-	-	PP
TP02	89.2	4.00	-	-	PP
TP03	89.9	2.50	1.20	-	Soakaway, PP
TP04	88.9	4.00	2.40[3]		PP

1: Based on nearby topographic features
2: Terminated on suspected buried service, Made Ground likely associated with service backfill
3: Thickener Made Ground likely associated with retaining wall construction
4: Terminated due to refusal on dense Sand

3 Ground Conditions

The ground sequence encountered is described below, and each of the strata properties presented in the following sections.

3.1 Summary

The boreholes generally encountered a thin layer of Topsoil described as a dark grey to brown gravelly clayey sand. Made Ground was generally approximately 1.0m in thickness but was not encountered in WS01, TP01A or TP02, at the central-western portion of the site. Thicker Made Ground (2.40m) was encountered at TP04, likely associated with construction of the concrete retaining wall present at the north eastern corner of the site. The Made Ground was described either as a GRAVEL of brick, concrete and ash, or as a gravelly, sandy CLAY with gravel of limestone and brick.

Glacial Till was encountered in all investigation locations and the base was not proven as part of the investigation. It was encountered as a mixture of gravel, sand, silt and clay, generally as a sandy, silty CLAY, a very gravelly CLAY with gravels of mudstone, or as a gravelly clayey SAND. It was generally described as firm where cohesive, and medium dense where predominately granular.

The following table summarises the anticipated depth below ground (m bgl) of each stratum across the site. The different strata are discussed in the following sections. Two geological cross sections across the development area are presented in **Appendix B**.

Table 4: Distribution of Strata

Stratum	West, TP01, TP02, WS01		East, TP03, TP04, WS02, WS03	
	Top of Stratum m bgl (m aOD)	Thickness m	Top of Stratum m bgl (m aOD)	Thickness m
Topsoil	0.0 (90.4)	0.40		
Made Ground	-	-	0.0 (88.9)	1.0
Glacial Till	0.40 (90.0)	5.65+	1.0 (87.9)	5.05+
+ indicates that it extended to the maximum depth of the investigations in this area				

There were no historic investigations near the site that would indicate the depth to the Sandstone bedrock.

3.2 Made Ground

The Made Ground was encountered below concrete or paved areas in the eastern area of the site and was of mixed composition and description, though typically described as GRAVEL of brick, concrete and ash, or locally as in WS02 and WS03 as gravelly, sandy CLAY with gravel of limestone and brick. Due to the isolated instances of cohesive Made Ground, for the purpose of this assessment the Made Ground was considered as a granular material.

3.2.1 Plasticity and Particle Size

No particle size distribution was undertaken in the Made Ground. 2 nr. Atterberg Limit tests were undertaken on the cohesive fraction of the Made Ground and returned Plasticity Index results of 12% and 13% indicating it is a has a low plasticity. As discussed above the Made Ground was considered as a granular material.

3.2.2 Standard Penetration Testing

No SPTs were recorded within the Made Ground.

3.2.3 Density

Based on correlations presented in BS 8002 (2015) and a density term of medium dense, a unit weight of 17 kN/m^3 and 18 kN/m^3 is considered appropriate for Made Ground above and below the water table respectively.

3.2.4 Drained Shear Strength Parameters

As no in-situ testing was available for the Made Ground, shear strength parameters were derived from previous experience with granular Made Ground, the following value is recommended for design:

Granular Made Ground: $\emptyset' = 28$ °

3.2.5 Design Sulphate Class

A total of two samples of Made Ground were analysed for sulphate content and pH in accordance with BRE Special Digest 1*: Concrete in Aggressive Ground*. These returned pH results of 7.2 and 8.5. The water extractable sulphate content of the samples were 14 and 60 mg/l. As the data set is less than 10, the mean of the two highest values have been taken as the characteristic value (CV). the Design Sulphate Class for the Made Ground may be taken as DS-1. The associated ACEC Class is AC-1.

3.3 Glacial Till

Glacial Till was encountered in all investigation locations beneath Made Ground and was encountered as a mixture of cohesive and granular material. The cohesive material was generally described as a sandy CLAY with fine to coarse sand. It was locally gravelly, with the gravel comprising angular to subrounded mudstone.

3.3.1 Plasticity and Particle Size

Atterberg limits tests were carried out on 12 nr. samples of Glacial Till. The values of plasticity index (PI) ranged from 9 to 13 %, with an average of 11%. The Plastic Limit ranged from 12% to 21% with an average of 15%. The Glacial Till samples fall above the A-line which indicates the material is classified as a clay with a low to intermediate plasticity. 3 nr. of the samples were classified as Non-Plastic. Moisture contents were undertaken on 12 nr. samples of Glacial Till and derived an average moisture content of 18%.

Particle size distribution analysis was undertaken on a total of 7 nr. samples logged as cohesive Glacial Till. The results of the particle size distribution analysis are summarised in **Table 5.** These results indicate that the Glacial Till particle distribution is consistent with the log descriptions, comprising a mixture of sands and fines (silt and clay). The clay fraction of the Glacial Till was on average 46% of the particles and therefore this is considered to govern the behaviour of the strata as a cohesive material.

Table 5: Glacial Till - Particle Size Distribution Summary

Hole ID	Depth (m bgl)	Description	Distribution (%)			
			Very Coarse	Gravel	Sand	Fines (<0.063mm)
WS02	3.0 – 4.0	Sandy CLAY	0	1	52	47
WS02	4.0 – 5.0	Slightly gravelly clayey SAND	0	1	71	28
WS02	5.0 – 6.0	Slightly gravelly sandy CLAY	0	3	49	48
WS03	2.0 – 3.0	Sandy CLAY	0	0	41	59
WS03	3.0 – 4.0	Sandy CLAY	0	0	38	62
WS03	4.0 – 5.0	Clayey SAND	0	4	57	39

3.3.2 Pocket Penetrometer Testing

Pocket Penetrometer (PP) tests were undertaken at regular intervals in the trial pits and windowless samples. The inferred undrained shear strength is plotted on **Figure 8.**

3.3.3 Standard Penetration Testing

17 nr. SPT tests were carried out within the Glacial Till material. They resulted in 'N' values ranging from 5 to 54, with an average value of 19. **Figure 8** shows a plot of SPT 'N' values with elevation. A characteristic design line has been added correlating to the following design SPT relationship with depth:

*SPT N = 5 + 3z blows/300 mm, w*here z is depth below 89.5 m aOD

3.3.4 Density

Based on correlations presented in BS 8002 (2015) and a shear strength term of medium, a unit weight of 18 kN/m^3 and 19 kN/m^3 is considered appropriate for cohesive Glacial Till above and below the water table respectively.

3.3.5 Undrained Shear Strength Parameters

The results of the Pocket Penetrometer (PP) are plotted on **Figure 8**, along with inferred undrained shear strength from SPT testing adopting a f_2 factor of 6.0 based on an average plasticity index of 11% in accordance with CIRIA Report 143.

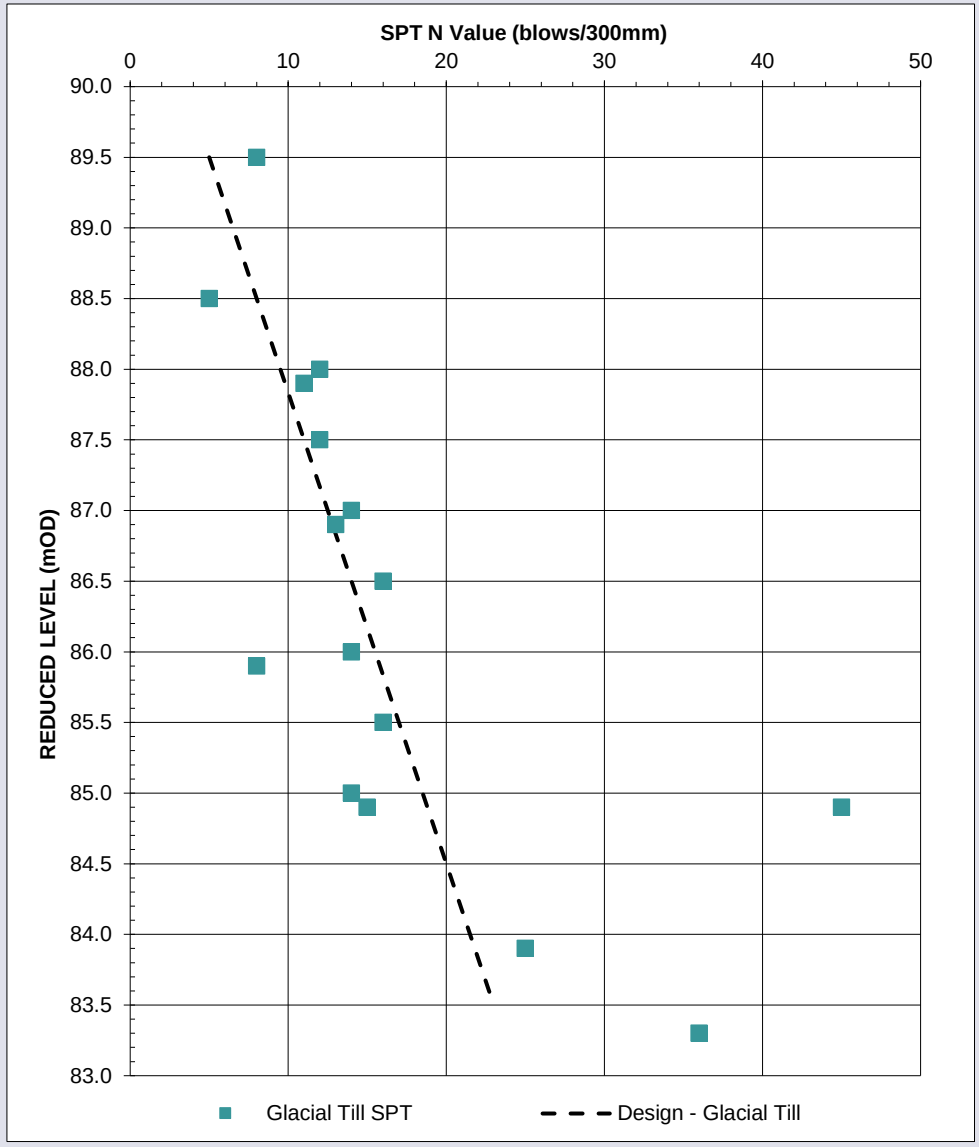
The design lines presented correlate to the following undrained shear strength profiles:

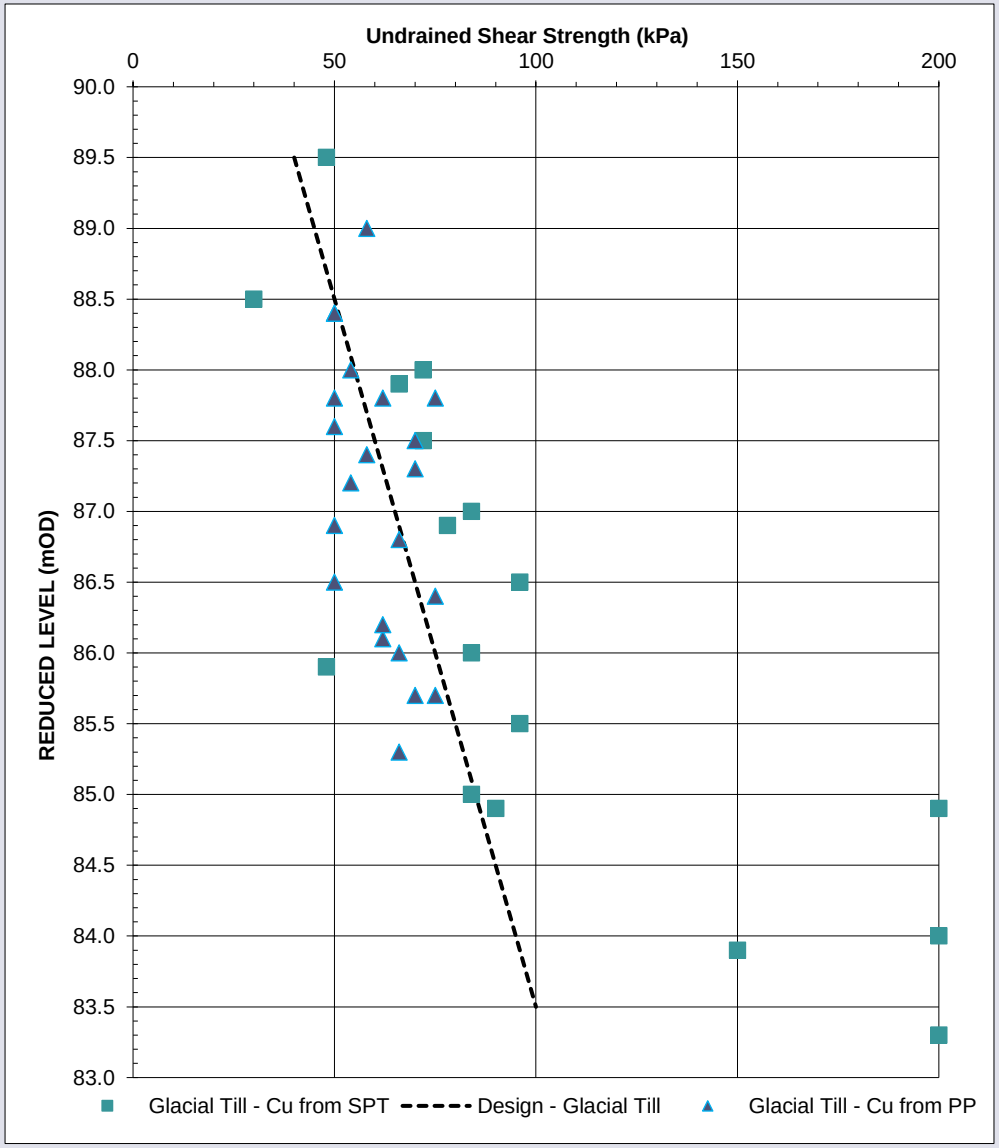
Glacial Till: $c_u = 40 + 10z$ kN/m^2

Where z is the depth below starting/datum elevation 89.5 m aOD

For low plasticity glacial tills, the undrained shear strength is closely related to the moisture content of the material (CIRIA 504). Strength testing of on remoulded specimens of Glacial Till at varying water contents predicted undrained shear strengths of approximately 100 kN/m^2 for a moisture content of 18 %, which is generally consistent with the undrained shear strength profiles derived from the in-situ testing.

Figure 8: Glacial Till - SPT Test Results (Left) and Undrained Shear Strength (Right)





3.3.6 Drained Shear Strength Parameters

The peak friction angle for cohesive soil can be estimated from the plasticity index of the soil, assuming an apparent cohesion of zero (BS 8002:2015). The peak friction angle is derived from the following relationship:

$$\emptyset'_{pk} = 42° - 12.5 \log_{10} I_p + \emptyset'_{dil}$$

Adopting an average plasticity index of 11 %, a critical stage friction angles of 29° and a peak angle of 31° is derived for cohesive Glacial Till.

The effective friction angle of cohesive Glacial Tills can also be estimated from the plasticity index as per Engineering in Glacial Tills (CIRIA Report 504) Figure 5.1. Considering a PI of 14 % a ϕ'_{peak} of 33° is derived.

Considering the above derived friction angles the following values are recommended as characteristic values for design:

Glacial Till: $\phi'_{cs} = 30°$

Glacial Till: $\phi'_{pk} = 32°$

In accordance with CIRIA 504 (Engineering in Glacial Tills) a critical state cohesion c'_{cs} value of 0 kN/m² is recommended for design for the cohesive Glacial Till. For non-fissured (intact) tills the peak cohesion can range from 0 kN/m^2 to 25 kN/m^2, therefore a peak cohesion c'_{pk} of 2 kN/m^2 is recommended for design. These values should be confirmed by the designer on a case-by-case basis when considering the design element and the anticipated soil stress level.

3.3.7 Stiffness

The undrained stiffness of the cohesive Glacial Till has been derived using the following relationship to the undrained shear strength as based on relationships presented in Tomlinson (2001), adopting an overconsolidation ratio (OCR) greater than 2 and an average PI less than 30 %.

$$E_u = 650\ c_u$$

Thus, for the design undrained shear strength value the following stiffness is recommended:

Cohesive Glacial Till: E_u = 26,000 + 6,500z kN/m^2, *elevation range 13.0 m aOD to 7.0 m aOD*

Where z is the depth below starting/datum elevation 13.0 m aOD

The drained stiffness of the cohesive Glacial Till is derived from the following relationship:

$$E' = 0.6\ E_u$$

3.3.8 Sulphate Class

For the 5 nr. Glacial Till samples the pH results were between 6.2 and 7.9. The water extractable sulphate contents of the samples were between 8 and 49 mg/l. As the data set is less than 10, the mean of the two highest values has been taken as the characteristic value (CV). Based on the CV, and in accordance with BRE Special Digest 1 (2005), the Design Sulphate Class may be taken as DS-1. The associated ACEC Class is AC-1.

3.4 Summary of Design Parameters

The following is a summary of recommended general characteristic design parameters for use at the site, based on the findings of the ground investigation.

Table 6: Summary of Recommended Parameters

Strata	γ_{DRY} (kN/m^3)	γ_{SAT} (kN/m^3)	ϕ'_{peak} (°)	ϕ'_{crit} (°)	c'_{peak} kPa	c'_{crit} kPa	c_u kPa	E_u MPa	E' MPa	BRE Class
Made Ground	17	18	28	28	0	0	-	-	-	DS-1 AC-1
Glacial Till	16	17	32	30	2	0	40 + 10z	26.0 + 6.5z	15.6 + 3.9z	DS-1 AC-1
z = Depth below 89.5 m aOD										

Note: geotechnical parameter selection is dependent on the context of the particular design activity; in this respect it is recommended that a geotechnical engineer reviews all final parameter selection within any detailed design stage calculations being carried out by other discipline engineers. Where available, a geotechnical design report/statement for the particular design application should also be referenced in preference to this general table of parameters.

3.5 Groundwater Conditions

A summary of the groundwater observations during the geotechnical investigations is provided in **Table 7**. The results of the groundwater monitoring are summarised in **Table 8.**

Table 7: Groundwater Strikes During 2023 Investigation

Hole ID	Hole Elevation (m OD)	Strike Depth (m bgl)	Strike Depth (m OD)
WS01	90.5	3.0	87.5
WS02	88.9	3.0	85.9
WS03	89	3.0	86.0
TP01	90.3	NE	-
TP01A	90.3	NE	-
TP02	89.2	NE	-
TP03	89.9	NE	-
TP04	88.9	Minor seepage at 3.7	Minor seepage at 85.2

NE = Not encountered

Table 8: Groundwater Observations During Monitoring

Hole ID	Hole Elevation (m aOD)[1]	Screen Depth (m bgl)	Date	Water Depth (m bgl)	Water Level (m aOD)	Stratum
WS01	90.5	1.0 – 5.5	23/01/2023	1.53	88.97	Glacial Till
			10/02/2023	1.60	88.9	
			20/02/2023	1.50	89	
			27/02/2023	1.70	88.8	
			13/03/2023	1.70	88.8	
			24/03/2023	1.73	88.77	
WS03	89.0	1.0 – 5.0	23/01/2023	1.70	87.3	Glacial Till
			10/02/2023	2.00	87.0	
			20/02/2023	2.00	87.0	
			27/02/2023	2.00	87.0	
			13/03/2023	2.00	87.0	
			24/03/2023	2.02	86.98	

WS01 is located at the western potions of the site, and recorded groundwater between 88.8m and 89.0m OD over the period of monitoring. WS03 at the eastern end of the site recorded groundwater at between 87.3m and 87.0m OD. The measured groundwater levels from the monitoring period were quite stable, recorded over the wettest portion of the year.

Considering the site monitoring data, the following groundwater level is considered appropriate for design.

Western Portion: *typical/average groundwater level = 1.50m bgl (88.9mOD)*

Eastern Portion: *typical/average groundwater level = 2.00m bgl (87.0mOD)*

3.6 Infiltration Testing

Infiltration testing was undertaken in accordance with BRE Digest 365 in TP01 and TP03 at depths of 1.0m and 1.5m below ground level. In both tests, insufficient infiltration took place after 90 minutes of monitoring therefore an infiltration rate was unable to be calculated, indicating infiltration is not likely to be feasible.

4 Geotechnical Recommendations

The key geotechnical design elements for the proposed scheme are listed below:

- Shallow foundation design,
- Pavement subgrade requirements,
- Soil volume change potential,
- Cutting slope stability,
- Cut and fill earthworks to form the scheme finished levels,
- Recommendations for buried concrete aggressivity,
- Unexploded Ordnance risk assessment,
- Proposed retaining wall design, and
- On site infiltration.

Key geotechnical considerations for these elements are described in the sections below, and where further design is required, it will be referred to the Geotechnical Design Report.

4.1 Design of Shallow Foundations

Shallow foundations are considered to be a suitable foundation type for the proposed development given the size of the proposed structures and the anticipated loadings, either as pad/strip footings, or rafts. A preliminary design of typical shallow foundations is presented in the following sections.

4.1.1 Ultimate Bearing Capacity

The ultimate bearing capacity of shallow foundations is calculated using total and effective stress methods dependent on the type of strata and loading conditions present. The equations to calculate bearing capacity are as per EN 1997-1:2004.

4.1.2 Allowable Bearing Capacity

The allowable bearing capacity of shallow foundations has been calculated using the following combinations for Eurocode design approach 1 (EN 1997-1:2004).

Combination 1: A1 “+” M1 “+” R1

Combination 2: A2 “+” M2 “+” R1

The partial factors to be applied for each of the above combinations can be found in **Table 9**, **Table 10** and **Table 11**:

Table 9: Shallow Foundation Partial Factors on Actions or the Effects of Actions (EN 1997-1:2004)

Action		Symbol	Set	
			A1	A2
Permanent	Unfavourable	γ_G	1.35	1.0
	Favourable		1.0	1.0
Variable	Unfavourable	γ_Q	1.5	1.3
	Favourable		1.0	1.0

Table 10: Shallow Foundation Partial Factors for Soil Parameters (EN 1997-1:2004)			
Soil Parameter	Symbol	Set	
		M1	M2
Shear Strength*	$\gamma_{\varnothing}'$	1.0	1.25
Effective Cohesion	γ_c'	1.0	1.25
Unit Weight	γ_γ	1.0	1.0
* This factor is applied to tan Ø'			

Table 11: Shallow Foundation Partial Resistance Factors for Spread Foundations		
Resistance	Symbol	Set
		R1
Bearing	$\gamma_{R;v}$	1.0

4.1.3 Preliminary Shallow Foundation Design

Made Ground is generally variable and given it was encountered as a relatively thin layer, its recommended that shallow foundations are taken through the Made Ground and founded in the natural Glacial Till soils. The bearing capacity calculation for load combination 2 in **Table 12** has assumed 0.6m wide strip footings on level ground with a nominal 0.6m embedment. Combination 2 output only is presented as this is the critical case for soil bearing capacity, and should be combined with a load combination of 1.0 Dead Load + 1.3 Live Load.

Table 12: Preliminary Shallow Foundation Bearing Capacity		
Founding Material	Combination 2 Drained Bearing Capacity (kN/m^2)	Combination 2 Undrained Bearing Capacity (kN/m^2)
Cohesive Glacial Till	150	190[1]
1: Design has assumed an undrained shear strength of 50 kN/m^2, therefore an elevation of 88.5mOD		

These assumptions and preliminary design values should be reviewed by the Geotechnical Engineer once loads and foundation types are confirmed.

4.1.4 Settlement of Shallow Foundations

An assessment of the estimated settlement of shallow foundations should be undertaken once loads and foundation configurations are known.

4.2 Pavement Subgrade

The California Bearing Ratio (CBR) of the pavement subgrade can be estimated using the following relationship as per LR1132 and soil stiffness:

$$E = 17.6(CBR)^{0.64}\ MN/m^2$$

For Glacial Tills, CIRIA 504 provides guidance on deriving CBR values based on the undrained shear strength and moisture content of the soil. Considering the above derived values, and previous experience with similar materials the following preliminary CBR values are recommended for design, these values should be tested at formation level using either Dynamic Cone Penetrometer or Plate Bearing testing of the exposed subgrade. Pavements should incorporate appropriate surface and subsurface drainage with the aim of maintaining a relatively stable moisture condition within the subgrade soils.

Glacial Till: CBR = 2.0%

4.3 Soil Volume Change

The plasticity testing undertaken within the Made Ground gave an average Plasticity Index of 11%. The fraction of particles smaller than 425µm was on average 94%, therefore the modified Plasticity Index was taken as 10%. Therefore the Glacial Till is considered to have a very low to negligible volume change potential.

The proposed site development comprises construction of a single level basement into the existing slope (refer **Figure 3**). This will require excavation of up to 4.5m in isolated areas, but generally 2.5m to 3.5m over the site. This excavation unloading will result in immediate heave as a result of elastic response to the unloading and will occur immediately during excavation and therefore will not affect the proposed structures. Long term heave will also occur in cohesive soil associated with the long term response of the soil to the change in effective stress. The shallow foundation design will need to consider the long term unloading pressure in the design of shallow footings to ensure ground movement (heave or settlement) does not affect the proposed structure significantly. Where the floor slabs are suspended over strip or pad foundations, void formers should be incorporated to prevent the effects of unloading heave on the floor slabs.

The existing vegetation will be removed as part of the site clearance, and therefore there is a risk of volume change occurring as a result of moisture change within the soil zones affected by the tree roots. As excavation is occurring across the site where the proposed floor slabs are located it is considered likely that the soil zone affected by the moisture change from the vegetation will be excavated and therefore will not affect the foundations. A tree survey was undertaken by Mulberry Tree Management (MTM), and the plan is shown on Drawing BS5837 and a snapshot shown below in **Figure 9** with the excavation and filling plan from **Figure 3** overlain.

This shows that within the footprint of the proposed structures, tree zones G1, G6, G7 and T5 will be present.

Figure 9: Tree and Excavation Plan

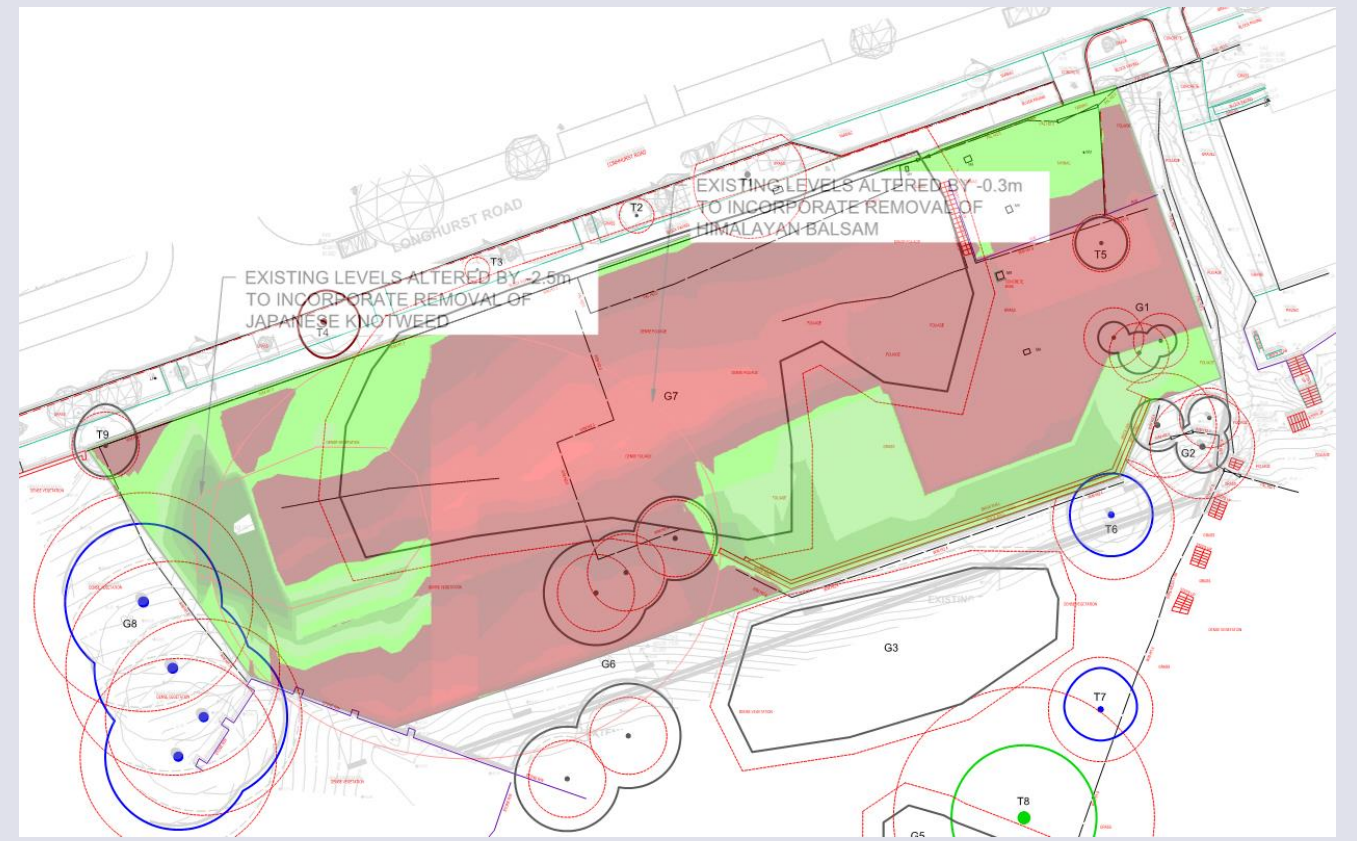


Table 13: Tree Information				
Tree Zone	Types	Max. Water Demand	Max. Mature Tree Height[1]	Min. Foundation Depth[1,2]
G1	Alder	Moderate	18.0m	1.55m
G6	Oak, Ash and Willow	High	24.0m	2.45m
G7	Alder, Birch and Willow	Moderate	24.0m	1.55m
T5	Robinia (Locust)	Low	14.0m	1.15m
1: As per NHBC (2023) *Building near trees* 2: Assumes a worst case distance of 1.0m from the trees				

The upper level tiered area is proposed to be garden planting areas, and therefore is not anticipated to comprise anything that structurally will be affected by soil volume change. But as shown on **Figure** 4, the foundation for the upper section of blocks spanning over the void cut slope will be founded at the upper level, and therefore the foundation design will need to consider the minimum foundation depths as per **Table 13**, subject to detailed design of the foundations.

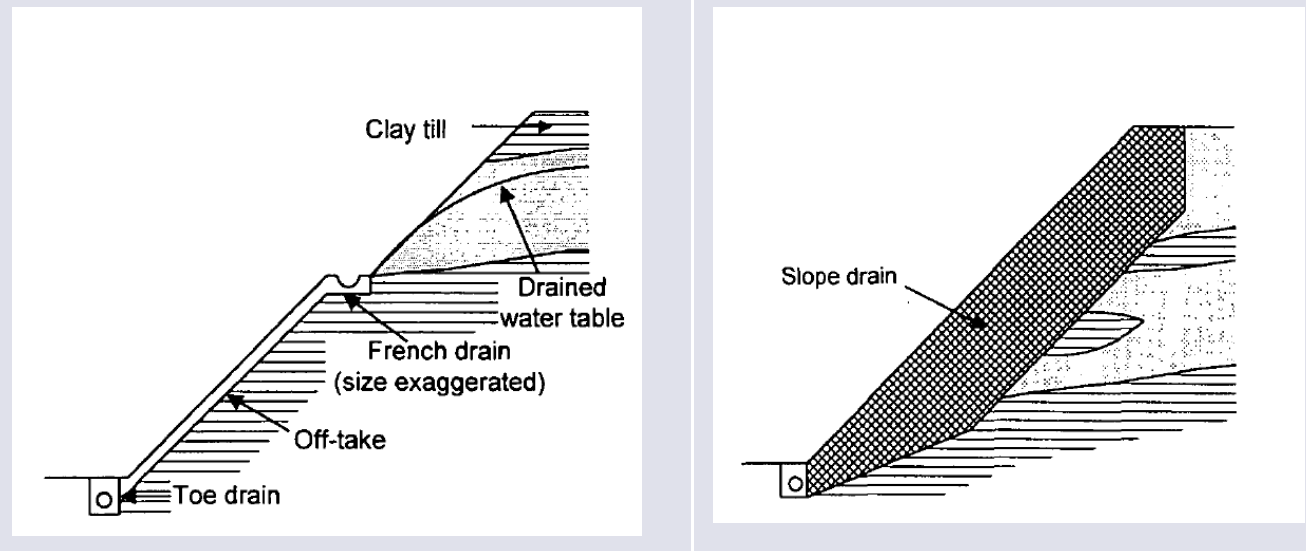
For the lower level of structural foundations (Longhurst Road level), the excavation levels shown in **Figure 9** indicates that the excavation is greater than the required minimum foundation depth as per **Table 13**, and therefore the affected soil zone will be excavated as part of the site works.

4.4 Cutting Slope Stability

Cut slopes are proposed at the back of the housing units as shown in **Figure 4**, based on the lower floor levels of 86.5mOD and upper terrace levels of approximately 91.0mOD, the cut slope will be approximately 4.5m in height. Current proposals show this cut face at an angle of 1V:1H, which is unlikely to be stable long term, based on descriptions of soils strengths a cut slope angle of 1V:2.5H is recommended for long term with appropriate slope drainage as described below.

Glacial Tills can be a mixture of predominately cohesive material that is relatively impermeable, with layers of granular material (sands and gravels) that can be water bearing. Therefore, a key consideration of cut slopes is incorporating drainage into cut slopes where granular material is encountered to prevent shallow failures from occurring. The sketches below in **Figure 10** show possible drain layouts where granular till is encountered. Toe drainage should be provided at the toe of the cut slope in the void behind the houses to prevent water ponding.

Figure 10: Cut Slope Drainage in Glacial Till as per Ciria 504



4.5 Earthworks

The Glacial Till soils on site were classified as low to intermediate plasticity and therefore may be suitable for reuse on site provided moisture contents are within the optimum bounds and adequate compaction is achieved. Generally the moisture content of the till to be reused should be less than 1.2 times the Plastic Limit (PL) at the time of placement, considering the average PL of 15%, the moisture content should be less than 18% when placed.

If earthworks are conducted during the summer months controls on moisture content will be more readily available for soils that had moisture contents wetter of optimum. If earthworks are proposed during the winter when it is wetter, stabilisation of soils with lime or cement stabilisation may be required.

4.6 Buried Concrete Summary

Based on an assessment in accordance with the BRE (2005) document presented in the relevant sections of Section 3. The results indicate that concrete in contact with Made Ground or Glacial Till requires a concrete mix design sulphate class of DS-1, with an aggressive chemical environment for concrete class (ACEC) of AC-1.

4.7 Unexploded Ordnance (UXO)

The United Kingdom was targeted in terms of bombing during World War II. The presence of unexploded bombs presents a risk for construction workers and residents of the surrounding areas.

A qualitative assessment of the UXO risk based on the CIRIA report C681, using the historical land use in conjunction with the bombing hazard records for the site is presented here. The variables for consideration are listed in **Table 14**. Scores (which depend on the potential risk for aerial delivered UXO stated in the CIRIA report C681) have been assigned to the assessable variable in accordance with C681.

Table 14: UXO Risk Score Variables

Variables	Score	Evidence
Bombing Density	4	Very High
	3	High
	2	Medium
	1	Low
Site Location	4	Cities
	3	Brownfield sites / large towns
	2	Small towns
	1	Rural
Historical Land Use at Bombing	4	Site of military use, railway marshland, power station / industrial centre
	3	Site adjacent to wartime military use, railway marshland, power station/ industrial centre.
	2	Site near to wartime military use, railway marshland, power station/ industrial centre.
	1	Greenfield site or agricultural land only
Evidence of post-war development on site	4	No evidence
	3	Minimal excavation
	2	Moderate excavation
	1	Excavation of entire site
Area of ground disturbance of the proposed works	4	Deep excavation over extended areas / driven piles
	3	Deep excavation over limited areas/ shallow excavation over extended areas/ sheet and auger piles
	2	Excavation for services
	1	Boring drilling

The level of the site-specific risk should be used to categorise any proposed work on site as severe, high, medium, low or negligible risk. The extent and type of mitigation should be determined based on the risk level as presented in **Table 15**.

Table 15: Risk Levels and Recommended Mitigation		
Score	Risk	Recommended Mitigation
> 16	Severe	Commission a full desk study and risk assessment by specialist bomb disposal company.
14 – 16	High	Carry out a UXO site search using surface and down hole magnetometers. Change the design works if possible to avoid suspected UXO's or intrusive groundworks.
11 - 13	Medium	Commission a full desk study and risk assessment by specialist bomb disposal company.
9 - 10	Low	Carry out simple site precautions: • Contactor/Operator "awareness" briefing. • Minimise the number of constructions works close to the excavation during digging. • Minimise the width and depth of excavation where possible. • Careful excavation methodology where feasible (i.e. hand excavation). Stop excavation immediately on encountering anything unexpected and alert the MoD/Pell Frischmann. • CAT scanning prior to excavation (Note that CAT scanning cannot conclusively detect UXO's)
< 9	Negligible	None (Though contractor should be made aware that the risk still exists). Standard risk mitigation measures apply.

The possible presence of UXO is considered with respect to the risk of encountering UXO during the proposed works. The UXO risk assessment is shown in **Table 16**.

Table 16: UXO Risk Score		
Variables	Score	Evidence
Bombing Density	2	Moderate from UXO risk map
Site Location	3	Urban areas
Historical Land Use	1	No nearby military installations
Post-War Development	3	Minimal excavation
Proposed Works	3	Shallow excavation over extended areas
Calculated UXO Risk Score	**12**	**Medium**

The calculated UXO risk is 12. This UXO risk score is considered Medium, therefore a more detailed desk study is recommended for this site. The workers should be briefed as to the potential risks as the site would have been a potential target during the second world war. The width and depth of excavations should be kept minimal and excavation locations should be CAT scanned prior to commencing works.

4.8 Retaining Wall Design

The retaining wall requirements are outlined in **Section 1.3** and the drawings in **Appendix A**. The primary wall has a height of up to 3.70m at its highest, and the secondary wall retains up to 2.0m. Where the primary wall is highest it retains level ground to the boundary, to reduce the wall height and associated costs this could be graded to reduce the wall height by approx. 2.0m.

These walls are in areas of cutting and therefore a king post wall or gravity wall is considered to be the most suitable for these wall heights. A preliminary retaining wall optioneering exercise was undertaken for the proposed development as outlined in 106240-PEL-GEN-XX-RP-D-0001 and gave preliminary dimensions for the possible wall solutions. These options are discussed below:

- King Post Wall: A king post wall consists of an array of steel or timber posts between which concrete or timber boards are placed to retain the fill. For heights greater than 2.50m, timber may not be suitable as a post material and steel king posts may be required. King post wall installation will require a piling

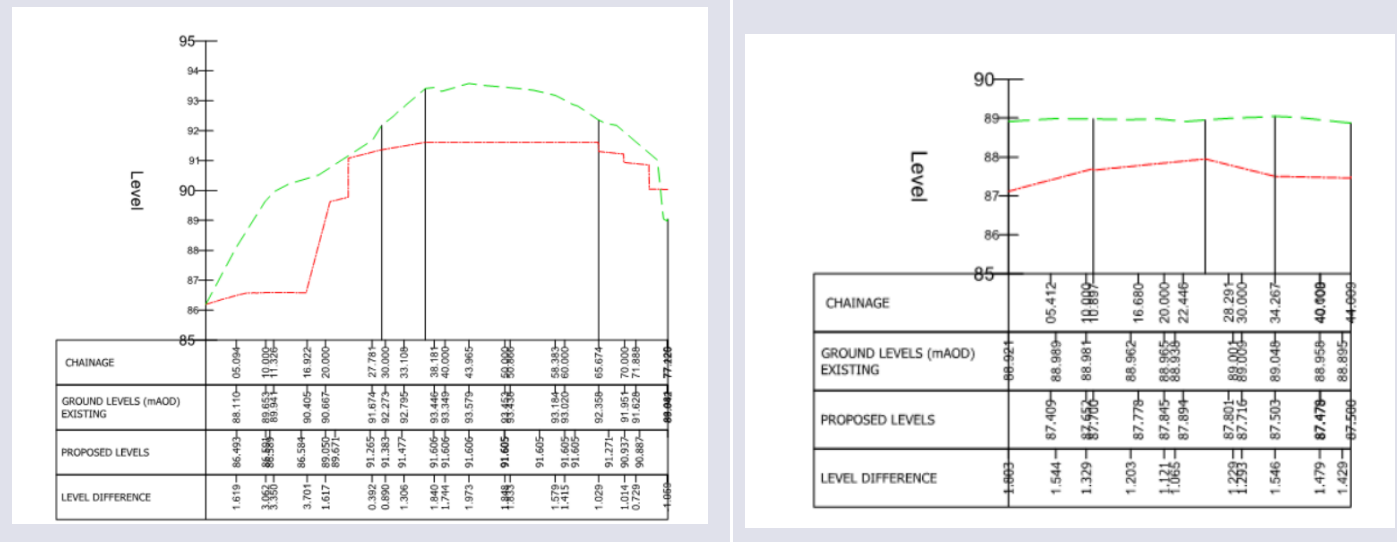
rig to form the bored piles, with king posts placed into the bore and concreted into place. For preliminary sizing the following embedment depths may be adopted, subject to detailed design.

- Maximum retained height of 3.70m, a total required pile length of 10.5m. As the highest retained height is over a relatively short section of wall, waler beams could be used to share the load to the shorter sections of wall and reduce the required post size and length.
- Maximum retained height of 2.50m, a total required pile length of 7.5m.

- Sheet Pile Wall: A sheet piled wall uses driven steel sheets to create a cantilevered retaining wall. The required embedment depth would be similar to that of a King Post Wall, but would be installed with pile driving equipment rather than a bored pile.

- Gravity Wall: A gravity wall constructed of reinforced concrete could be installed for heights up to 2.5m, however where the retained height exceeds 3.0m, a gravity wall base width required may make the wall uneconomical to construct.

Figure 11: Proposed Retaining Wall Elevations, Primary (Left) and Secondary (Right)



4.9 Onsite Infiltration

Infiltration testing undertaken within the Glacial Till soils recorded insufficient infiltration to calculate an infiltration rate, therefore it is not considered suitable for onsite infiltration.

5 Geotechnical Risk Register

A geotechnical risk register (**Table 18**) has been produced for the scheme in order to identify potential hazards, the probability of the hazard occurring, impact and risk rating. In addition, an estimate of cost implications if the risk occurred prior to the implementation of risk control measures is provided (**Table 17**).

It is a simple qualitative risk assessment and should not be viewed as definitive. This Risk Assessment reflects the current level of understanding of the geotechnical and geo-environmental aspects of the scheme and will be subject to revision. The risk rating is defined by the following relationship:

Table 17: Risk Assessment Criteria and Rating

Probability (P)		Impact (I)		Time	Cost
Very likely	5	Very likely	5	>10 weeks on completion	Very High
Probable	4	Probable	4	>1 week on completion	High
Possible	3	Possible	3	>4 weeks: <1 week on completion	Moderate
Unlikely	2	Unlikely	2	1 to 4 weeks: none on completion	Low
Negligible	1	Negligible	1	<1 week to activity: none on completion	Minimal

Table 18: Geotechnical Risk Register

Hazard / Risk	Cause	Consequence(s)	Pre-Control			Mitigation	Post-Control		
			P	I	R		P	I	R
Unforeseen ground conditions / General ground risk	-Unusual ground conditions.	-Increased geotechnical risk. -Conservative design approach. -Instability of excavations	4	3	12	-Appropriate ground investigation coverage with contamination risk assessment. -Appropriate design parameters and design methods.	2	3	6
Presence of variable made ground	-Unusual and unexpected ground conditions.	-Obstructions in Made Ground encountered during construction delaying piling for retaining walls and excavation works.	2	2	4	-Appropriate ground investigation coverage. - Contractor should outline in their excavation and piling method statements how the risk of obstructions within Made Ground will be mitigated and the procedure for dealing with obstructions.	1	2	2
Unexploded Ordnance (UXO)	-Breaking ground in areas at risk of containing UXO	-Explosion resulting in anything from no injury or minor injury to multiple fatalities of construction workers and general public. -Delay in construction. -Significant cost of remediation. -Damage to existing infrastructure.	3	5	15	Request Detailed Unexploded Ordnance (UXO) Threat & Risk Assessment. Including: - UXO Safety & Awareness Briefings -Operational UXO Emergency Response Plan; -Where trial pits, excavation and trenches are undertaken Non-intrusive UXO Survey and/or EOD Banksman Support into previously undisturbed ground; Where 'blind' intrusive works into previously undisturbed ground are proposed, an intrusive UXO survey (employing down-hole magnetometer or MagCone techniques) is recommended.	1	5	5

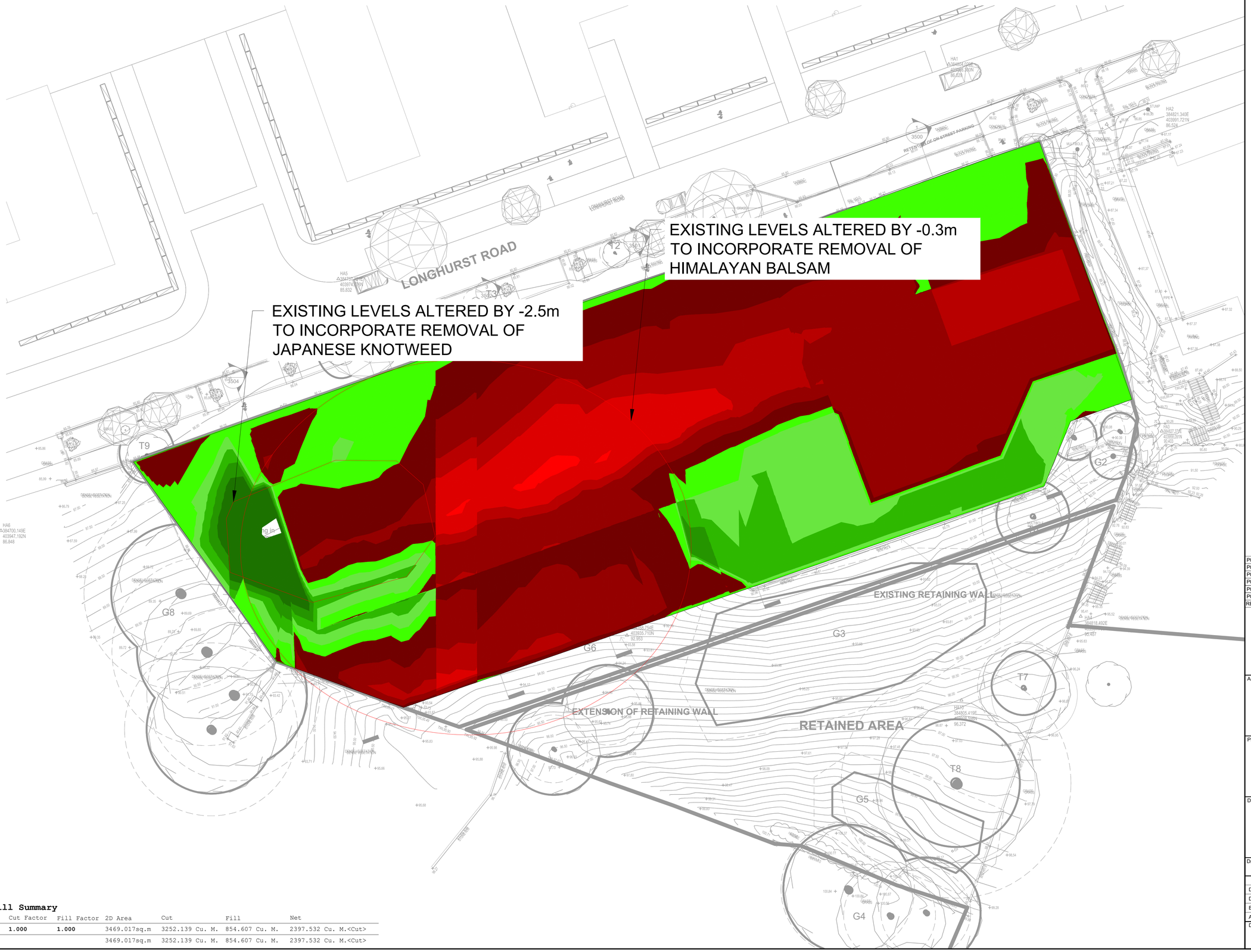
Table 18: Geotechnical Risk Register

Hazard / Risk	Cause	Consequence(s)	Pre-Control			Mitigation	Post-Control		
			P	I	R		P	I	R
Pollution of Environment	-Unsuitable handling and storage of potentially contaminated sub-surface material(s) during construction -Flooding of construction works with sediment loaded water.	-Potential cost of additional disposal -Health risks -Disturbing materials could mobilise contaminants in dust blow, leachate, etc. -Migration to adjacent land -Impact on adjacent ecology -Legal liability for nuisance, etc.	2	2	4	-Removal of unsuitable material(s) and replacement with acceptable materials -Suitable handling and storage methodologies including appropriate PPE -Removal and disposal to off-site licensed landfill -Ensure imported materials are suitable for use and have appropriate/adequate certification -Use of appropriate construction method to minimise/reduce risk of developing pathway for contaminants	1	2	4
Damage to buried services	-Inaccurate / no service plans. -Service location not undertaken prior to breaking ground during either construction or site investigation.	-Damage to utilities -Health and safety risk to site personnel and general public -Utilities temporarily unavailable -Environmental impact -Litigation	3	5	15	-Confirm accuracy of service plans on site by means of hand excavation and on-site detection to determine location and depth of services. -Services plans to be sourced / produced prior to construction works commencing. -Protect or divert services prior to construction works beginning on site. -All work locations to be scanned for services prior to work commencing.	2	5	10
Shrinkable soils	-High volume change soils -Influence of trees	- Damage to new structures.	3	4	12	- NHBC recommended use of proprietary materials and voids to accommodate heave. - Adequate design and assessment of effects on structures due to differential shrinkage.	3	1	3

Table 18: Geotechnical Risk Register

Hazard / Risk	Cause	Consequence(s)	Pre-Control			Mitigation	Post-Control		
			P	I	R		P	I	R
Slope Stability	- New proposed slope - Localised steep slope	- Construction delays. - Damage to new or existing structures.	3	4	12	- Detailed slope stability assessment undertaken as part of design. - Incorporate slope drainage.	3	1	3

Appendix A – Proposed Development Drawings



Cut/Fill Summary

Name	Cut Factor	Fill Factor	2D Area	Cut	Fill	Net
Surface6	**1.000**	**1.000**	3469.017sq.m	3252.139 Cu. M.	854.607 Cu. M.	2397.532 Cu. M.<Cut>
Totals			3469.017sq.m	3252.139 Cu. M.	854.607 Cu. M.	2397.532 Cu. M.<Cut>

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G2. ANY DIMENSIONAL DISCREPANCIES SHOULD BE NOTIFIED TO THE ENGINEER IMMEDIATELY.

G3. ALL DIMENSIONS ARE IN MILLIMETRES - (mm)
ALL LEVELS ARE IN METRES - (m) AND ARE ABOVE ORDNANCE DATUM AT NEWLYN, CORNWALL UNLESS NOTED OTHERWISE.

G4. NORTH SHOWN INDICATIVE ONLY

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CUT AND FILL NOTES

1. EXISTING GROUND MODEL BASED ON TOPOGRAPHICAL SURVEY 58510_02_01 DATED 11/04/2022
2. COMPARISON MODEL IS BETWEEN FINISHED SURFACE AND EXISTING GROUND -150mm TO ALLOW FOR TOPSOIL REMOVAL
3. THE CUT AND FILL DOES NOT TAKE INTO ACCOUNT ANY EXISTING OR PROPOSED UNDERGROUND SERVICES WITHIN THE EXTENT OF THE WORKS OR WORKS TO THE BOUNDARY WALL.

SURFACE LEVEL DATA

NUMBER	MINIMUM LEVEL	MAXIMUM LEVEL	COLOUR
1	-4.50	-4.00	
2	-4.00	-3.50	
3	-3.50	-3.00	
4	-3.00	-2.50	
5	-2.50	-2.00	
6	-2.00	-1.50	
7	-1.50	-1.00	
8	-1.00	-0.50	
9	-0.50	0.00	
10	0.00	0.50	
11	0.50	1.00	
12	1.00	1.50	
13	1.50	2.00	
14	2.00	2.50	
15	2.50	3.00	

REV	DESCRIPTION	DRN	CHK	APP	DATE
P06	INVASIVE SPECIES ADJUSTMENTS	PC	AH	MD	03.04.23
P05	REDUCED RETIANING WALL	PC	AH	MD	24.03.23
P04	NEW SITE LAYOUT	PC	AH	MD	01.11.22
P03	NEW SITE LAYOUT	PC	AH	MD	26.06.22
P02	CUT AND FILL QUANTITIES REISSUE	PC	AH	MD	26.06.22
P01	FIRST ISSUE	PC	AH	MD	20.05.22

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Architect/Client/Contractor

greatplaces HOUSING GROUP

Project

LONGHURST ROAD

Drawing Title

CUT AND FILL

Drawing Status

PRELIMINARY

	Name	Date
Drawn	P. CLARK	20.05.22
Designed	P. CLARK	20.05.22
Eng Chk	A. HALL	20.05.22
Approved	M. DAVIES	20.05.22

Status Code: S3

Scale: 1:200

Revision: P05

Drawing No.

106240 - PEL - HML - 01 - DR - D - 0001

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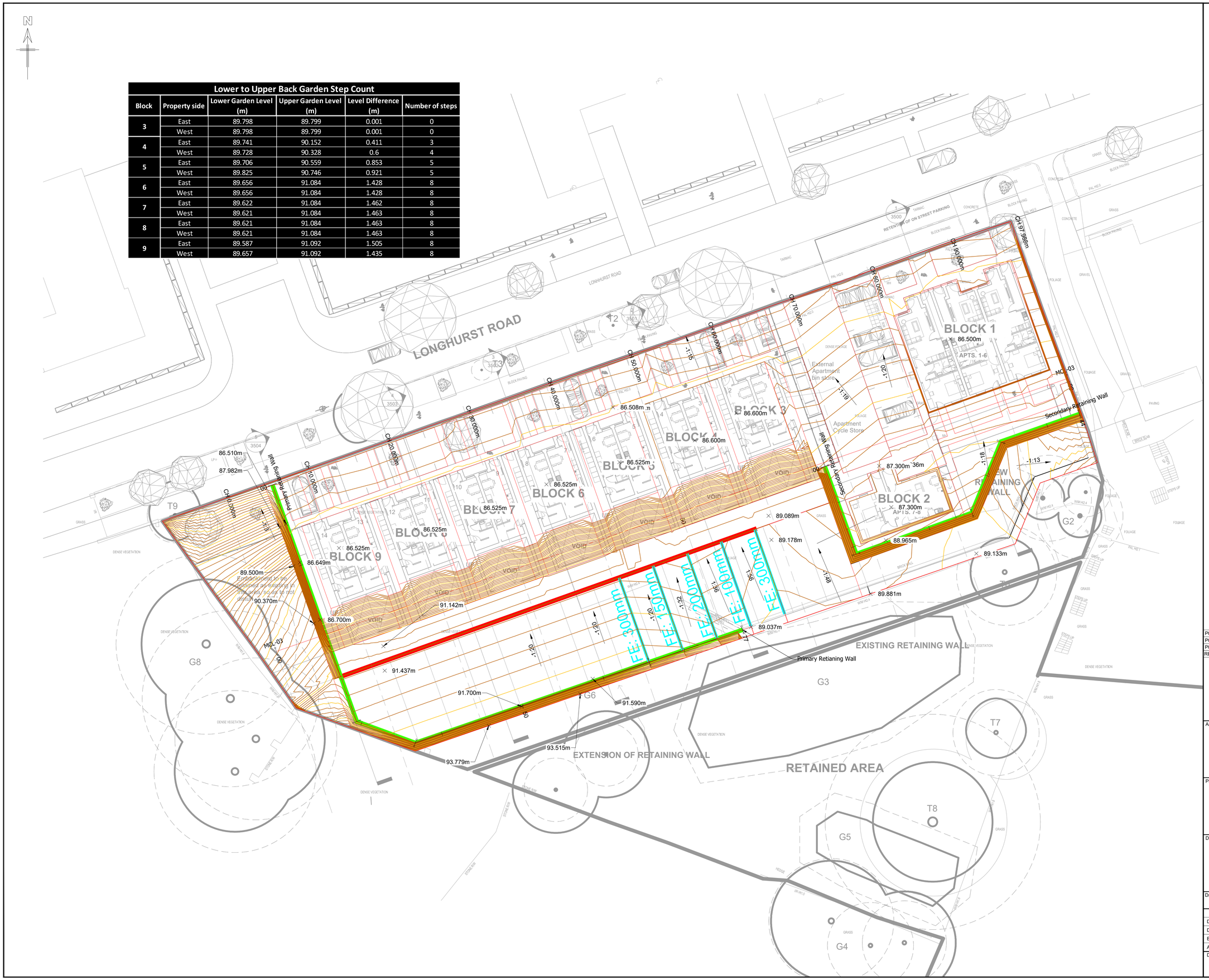
Lower to Upper Back Garden Step Count					
Block	Property side	Lower Garden Level (m)	Upper Garden Level (m)	Level Difference (m)	Number of steps
3	East	89.798	89.799	0.001	0
	West	89.798	89.799	0.001	0
4	East	89.741	90.152	0.411	3
	West	89.728	90.328	0.6	4
5	East	89.706	90.559	0.853	5
	West	89.825	90.746	0.921	5
6	East	89.656	91.084	1.428	8
	West	89.656	91.084	1.428	8
7	East	89.622	91.084	1.462	8
	West	89.621	91.084	1.463	8
8	East	89.621	91.084	1.463	8
	West	89.621	91.084	1.463	8
9	East	89.587	91.092	1.505	8
	West	89.657	91.092	1.435	8



GENERAL NOTES

G1. DO NOT SCALE THIS DRAWING.

G2. ANY DIMENSIONAL DISCREPANCIES SHOULD BE NOTIFIED TO THE ENGINEER IMMEDIATELY.

G3. ALL DIMENSIONS ARE IN MILLIMETRES - (mm)
ALL LEVELS ARE IN METRES - (m) AND ARE ABOVE ORDNANCE DATUM AT NEWLYN, CORNWALL UNLESS NOTED OTHERWISE.

G4. NORTH SHOWN INDICATIVE ONLY

G5. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL RELEVANT; SPECIFICATIONS; DRAWINGS; DETAILS AND OTHER DESIGN INFORMATION.

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G8. WHERE THE CONTRACTOR UNDERTAKES OR ENGAGES A THIRD PARTY TO UNDERTAKE TEMPORARY WORKS DESIGN, OR VARIES THE PELL FRISCHMANN DESIGN IN ANY WAY, THEN THE CONTRACTOR WILL TAKE FULL RESPONSIBILITY AND LIABILITY FOR ALL DESIGN ASPECTS, INCLUDING A DESIGN RISK ASSESSMENT. THE CONTRACTOR SHALL INFORM PELL FRISCHMANN OF ANY PROPOSED VARIANCES TO THE DESIGN.

CUT AND FILL NOTES

1. EXISTING GROUND MODEL BASED ON TOPOGRAPHICAL SURVEY 58510_02_01 DATED 11/04/2022
2. COMPARISON MODEL IS BETWEEN FINISHED SURFACE AND EXISTING GROUND -150mm TO ALLOW FOR TOPSOIL REMOVAL
3. THE CUT AND FILL DOES NOT TAKE INTO ACCOUNT ANY EXISTING OR PROPOSED UNDERGROUND SERVICES WITHIN THE EXTENT OF THE WORKS OR WORKS TO THE BOUNDARY WALL.

KEY

- MAJOR CONTOUR 0.5M
- MINOR CONTOUR 0.1M
- RETAINING WALL SEE DRAWING 106240-PEL-HML-01-DR-D-0003 FOR FURTHER DETAILS
- FLAG ON EDGE SEE DRAWING FOR DETAILS
- BRICK WALL SEE TABLE FOR DETAILS

REV	DESCRIPTION	DRN	CHK	APP	DATE
P03	REDUCED RETAINING WALL	PC	AH	MD	24.03.23
P02	NEW SITE LAYOUT	PC	AH	MD	01.11.22
P01	FIRST ISSUE	PC	AH	MD	29.06.22

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Architect/Client/Contractor

greatplaces HOUSING GROUP

Project

LONGHURST ROAD

Drawing Title

CONTOURS AND SECTION PLAN

Drawing Status

PRELIMINARY

	Name	Date
Drawn	P. CLARK	29.06.22
Designed	P. CLARK	29.06.22
Eng Chk	A. HALL	29.06.22
Approved	M. DAVIES	29.06.22

Status Code: S3
Scale: 1:200
Revision: P03

Drawing No.

106240 - PEL - HML - 01 - DR - D - 0002

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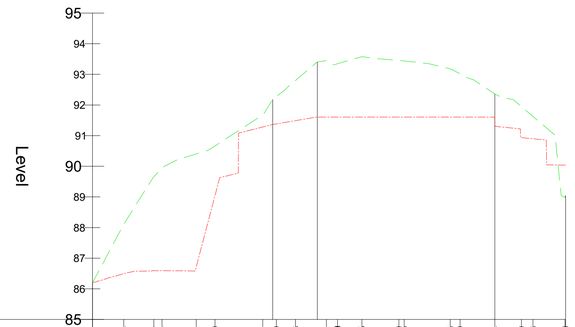
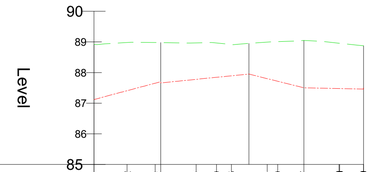
PRIMARY RETIANING WALL - LONGSECTION
SCALE: H 1:500,V 1:100. DATUM: 85.000



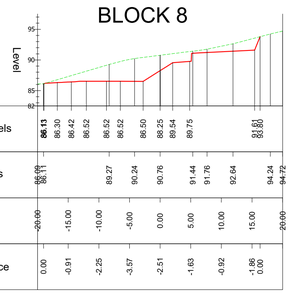
CHAINAGE	05.094	10.000 11.326	16.922	20.000	27.781	30.000	33.108	38.181 40.000	43.965	50.000 50.000	58.383 60.000	65.674	70.000 71.888	77.220
GROUND LEVELS (mAOD) EXISTING	88.110	89.653 89.941	90.405	90.667	91.674	92.273	92.795	93.446 93.345	93.579	93.458 93.458	93.184 93.020	92.358	91.951 91.628	88.043
PROPOSED LEVELS	86.493	86.581 86.581	86.584	89.050 89.671	91.265	91.383	91.477	91.606 91.606	91.606	91.605	91.605 91.605	91.271	90.937 90.887	
LEVEL DIFFERENCE	1.619	3.062 3.360	3.701	1.617	0.392	0.890	1.306	1.840 1.744	1.973	1.843 1.843	1.579 1.415	1.029	1.014 0.729	-1.069

SECONDARY RETAINING WALL
SCALE: H 1:500,V 1:100. DATUM: 85.000



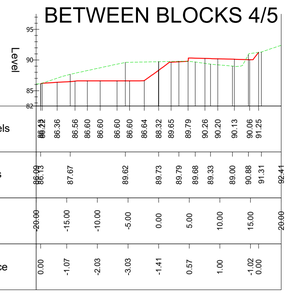
CHAINAGE	05.412	10.000 10.897	16.680	20.000 22.446	28.291 30.000	34.267	40.000	44.009
GROUND LEVELS (mAOD) EXISTING	88.921	88.989	88.981	88.962	88.965 88.938	89.001 89.009	89.048	88.958	88.895
PROPOSED LEVELS	87.409	87.652 87.706	87.778	87.845 87.894	87.801 87.716	87.503	87.478	87.500
LEVEL DIFFERENCE	-1.603	1.544	1.329	1.203	1.121 1.085	1.229 1.293	1.546	1.479	1.429

BLOCK 8



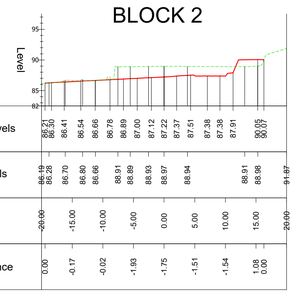
Chainage 20.000																					
Proposed Levels	86.15	86.30	86.42	86.52	86.52	86.50	88.25	89.54	89.75	93.81 93.80
Existing Levels	86.09 86.11	89.27	90.24	90.76	91.44	91.76	92.64	94.24	94.72
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-0.91	-2.25	-3.57	-2.51	-1.63	-0.92	-1.88 0.05	

BETWEEN BLOCKS 4/5



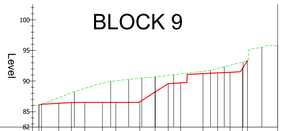
Chainage 50.000																					
Proposed Levels	86.12	86.36	86.56	86.60	86.60	86.60	86.64	88.32	89.65	89.79	90.26	90.20	90.13	90.06	91.25
Existing Levels	86.09 86.13	87.67	89.62	89.73	89.79	89.88	89.33	89.00	90.88	91.31	92.41
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-1.07	-2.03	-3.03	-1.41	0.57	1.00	-1.02 0.00	

BLOCK 2



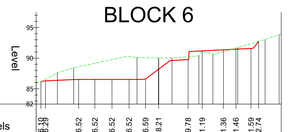
Chainage 80.000																					
Proposed Levels	86.21 86.30	86.41	86.54	86.66	86.78	86.89	87.00	87.12	87.22	87.37	87.51	87.38	87.38	87.91	90.05 90.07
Existing Levels	86.19 86.28	86.70	86.80	86.66	88.91	88.89	88.83	88.97	88.94	88.91	88.98	91.87
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-0.17	-0.02	-1.93	-1.75	-1.51	-1.54	1.08 0.00	

BLOCK 9



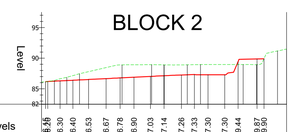
Chainage 10.000																					
Proposed Levels	86.22 86.30	86.38	86.50	86.53	86.53	86.53	86.62	88.22	89.58	89.71	91.21	91.33	91.44	93.03 93.04
Existing Levels	86.06 86.09	88.25	89.96	90.29	90.52	90.66	91.05	91.73	92.34	93.10	95.47	95.77
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-1.15	-2.75	-3.69	-2.44	-1.51	-0.66	-0.65	

BLOCK 6



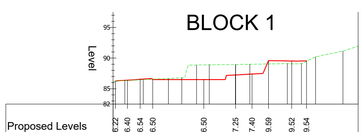
Chainage 40.000																					
Proposed Levels	86.10 86.29	86.35	86.52	86.52	86.52	86.52	86.59	88.21	89.78	91.19	91.38	91.46	91.59	92.74
Existing Levels	86.09 86.10	87.62	88.42	90.10	90.03	90.04	90.38	90.65	91.73	93.05 93.04
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-1.61	-2.72	-3.78	-1.82	0.82	0.42	-0.76 0.00	

BLOCK 2



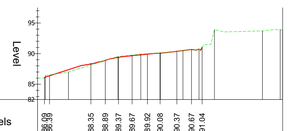
Chainage 70.000																					
Proposed Levels	86.12 86.29	86.30	86.40	86.53	86.67	86.78	86.90	87.03	87.14	87.26	87.33	87.30	87.30	89.44	89.87 89.90
Existing Levels	86.12 86.15	86.35	86.88	87.46	88.91	88.94	88.91	88.92	88.96	88.97	88.95	91.18 91.48
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-0.73	-1.70	-2.03	-1.78	-1.62	-1.67	0.92 0.00	

BLOCK 1



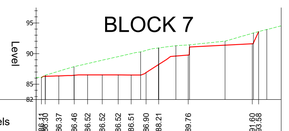
Chainage 90.000																					
Proposed Levels	86.22	86.40	86.54	86.50	86.50	87.25	87.40	89.59	89.52	89.54
Existing Levels	86.21	86.32	86.49	86.65	86.79	88.86	88.90	88.95	88.99	89.10	89.12	90.21	91.14	91.93
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	0.09	-0.22	-2.39	-1.70	-0.31	0.45 0.07	

BLOCK 0



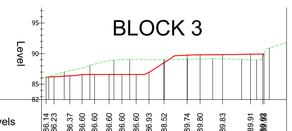
Chainage 0.000																					
Proposed Levels	86.09 86.09 86.09	88.35	88.89	89.37	89.67	89.92	90.08	90.37	90.67	91.04	93.71 93.66
Existing Levels	86.00 86.43	86.88	89.54	89.93	90.10	90.48	90.48	93.92	93.71 93.66
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	0.38	0.10	-0.11	-0.02	0.01 0.00			

BLOCK 7



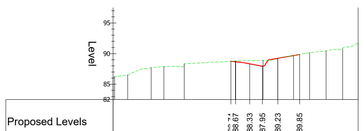
Chainage 30.000																					
Proposed Levels	86.14 86.30	86.37	86.46	86.52	86.52	86.51	86.90	88.21	89.76	91.60 91.59
Existing Levels	86.10 86.48	87.87	90.28	90.92	91.29	91.43	92.03	93.37	93.83	94.53
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-1.11	-2.21	-3.38	-2.71	-1.44	-0.61	-1.64 0.00	

BLOCK 3



Chainage 60.000																					
Proposed Levels	86.14 86.20	86.23	86.37	86.60	86.60	86.60	86.60	86.93	88.52	89.74	89.80	89.83	89.91 89.92
Existing Levels	86.10 86.20	87.01	88.10	88.93	89.07	89.02	89.08	89.17	89.09	88.94	90.91	91.82
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	0.00	-0.89	-2.01	-2.46	-0.50	0.69	0.70	0.97 0.00	

BLOCK 0



Chainage 97.988																					
Proposed Levels	88.67 88.67	88.33	87.95	89.23	89.85
Existing Levels	86.53	87.69	87.89	88.34	88.78	88.88	89.15	89.65	90.15	90.45	90.76	91.72
Offset	-20.00	-15.00	-10.00	-5.00	0.00	5.00	10.00	15.00	20.00
Level Difference	-0.12	-0.41	0.65 0.65	

GENERAL NOTES

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G2. ANY DIMENSIONAL DISCREPANCIES SHOULD BE NOTIFIED TO THE ENGINEER IMMEDIATELY.

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CUT AND FILL NOTES

1. EXISTING GROUND MODEL BASED ON TOPOGRAPHICAL SURVEY 58510_02_01 DATED 11/04/2022
2. COMPARISON MODEL IS BETWEEN FINISHED SURFACE AND EXISTING GROUND -150mm TO ALLOW FOR TOPSOIL REMOVAL
3. THE CUT AND FILL DOES NOT TAKE INTO ACCOUNT ANY EXISTING OR PROPOSED UNDERGROUND SERVICES WITHIN THE EXTENT OF THE WORKS OR WORKS TO THE BOUNDARY WALL.

KEY

EXISTING GROUND LEVELS

PROPOSED GROUND LEVELS

REV	DESCRIPTION	DRN	CHK	APP	DATE
P03	RETAINING WALL REDUCTION	PC	AH	MD	24.03.23
P02	CUT AND FILL QUANTITIES REISSUE	PC	AH	MD	26.06.22
P01	FIRST ISSUE	PC	AH	MD	29.06.22

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Architect/Client/Contractor

greatplaces HOUSING GROUP

Project

LONGHURST ROAD

Drawing Title

CROSS SECTIONS

Drawing Status

PRELIMINARY

	Name	Date
Drawn	P. CLARK	29.06.22
Designed	P. CLARK	29.06.22
Eng Chk	A. HALL	29.06.22
Approved	M. DAVIES	29.06.22

Status Code S3

Scale H-1:500 V-1:100

Revision P03

Drawing No.

106240 - PEL - HML - 01 - DR - D - 0003

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