



CABRA PBSA, FAUSSAGH AVENUE, CABRA

**PRELIMINARY SOIL NAIL SLOPE DESIGN FOR FUTURE WIDENING OF
RAILWAY CUTTING**

Rev 0

for

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PRELIMINARY SOIL NAIL SLOPE DESIGN FOR FUTURE WIDENING OF RAILWAY CUTTING

1 INTRODUCTION

AGL Consulting has been requested by Lohan & Donnelly Consulting Engineers carry out a preliminary soil nailed slope design to allow for potential future widening of the railway cutting that is adjacent to the Cabra PBSA site at Faussagh Avenue, Cabra in Dublin 7. The assessment carried out herein is to demonstrate that the proposed building development at the site will not prevent or hinder potential future works to the railway.

The proposed residential development at the site includes CFA pile supported buildings and the removal of circa 2m of made ground which is currently present at the site.

To allow for future widening of the railway located at the bottom of the slope, consideration has been given to a new cut slope with a steepened angle of 70 degrees that extends to the site boundary at the crest of the slope.

Soil nailing is a construction technique that is commonly used to stabilise existing or cut slopes. It comprises a grid of steel bars drilled and grouted into the slope face combined with a flexible or a rigid facing to support the soil between the nails.

The objective of this assessment is to determine:

- Preliminary design details for steepened soil nail reinforced slopes **at the existing undeveloped site**. The cut slopes are to allow widening of the railway cutting and extend to the property boundary.
- Preliminary design details for steepened soil nail reinforced slopes for the site **following development of the site with pile supported buildings** and lowered ground levels. The soil nails are required to extend below the buildings and therefore the arrangement of soil nails allows for the proposed CFA pile layout. .

The location of the site is shown in Figure 1-1 and the proposed site layout is shown in Figure 1-2. The overall plan length slope at the western site boundary is approx. 124m. The overall slope height ranges from 5.2 to 6.7m and from the railway track level to the site boundary.

This preliminary Geotechnical Design Report (GDR) provides the design of the soil nailed slope and facings including the design assumptions and the construction requirements. The soil nails have been designed in accordance with the principles of IS EN 1997-1:2005. The UK code of practice BS 8006-2:2011+A1:2017 is used for guidance.



Figure 1-1: Site location (Google Earth)

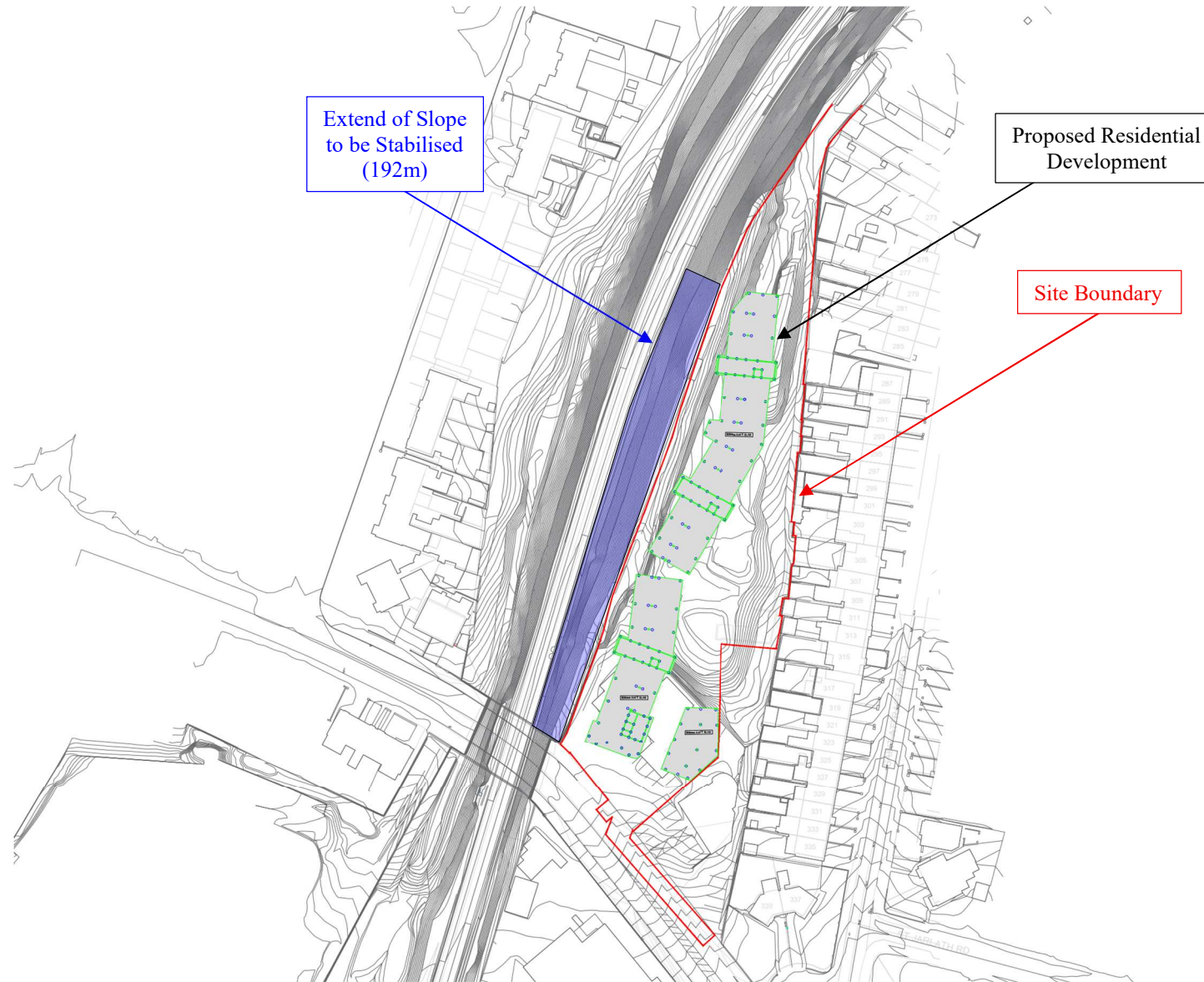


Figure 1-2: Site layout

2 SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 Site description

The Cabra PBSA site is located adjacent to the Pheonix Park Tunnel railway line, which is within an existing cutting that extends along the eastern site boundary. The plan length of the slope is 124m and has an overall height in the range 5.2 to 6.7m the ground level at the railway line to the slope crest at the site boundary. The existing slope inclination is approximately 36 to 42°.

The Cabra PBSA site comprises a relatively flat area at the top of the slopes, although currently mounds of stockpiled material are present. It is intended to remove the stockpiled material as part of the proposed development. The elevation of the ground surface at the site is in the range approx. +34.0 to +38.0mOD. The ground level at the railway line is approx. +27.0 to +28.0mOD.

A low concrete retaining wall, approximately 1 to 1.5m high is present at the crest of the slope at the southern end of the site. The plan length of the wall is approximately 27m. The wall is outside the site boundary and the wall is therefore to be retained in the proposed development.

A street view taken of the existing slope in September 2024 is shown in Figure 2-1. This shows the existing slope and the low retaining wall.



Figure 2-1: Photograph from Faussagh Avenue bridge facing north shown railway cutting slope (Google Maps, September 2024)

Sections showing the geometry of the existing railway cutting slopes and ground levels at the site are shown in Figure 2-2. The location of the section lines are shown in Figure 2-3 and are referred to as Sections A to I.

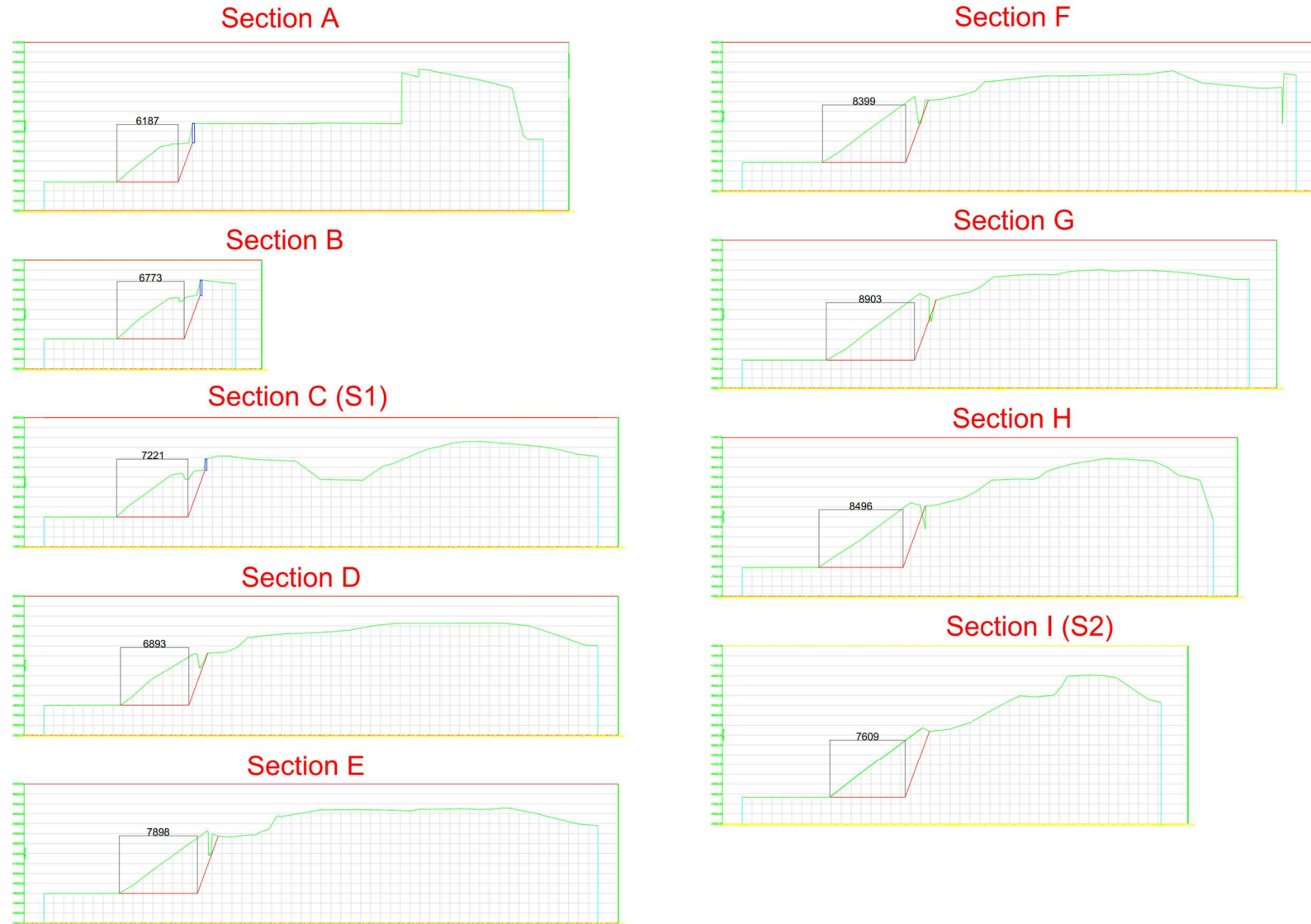


Figure 2-2: Sections showing existing railway cutting slope and proposed cut slope for widening railway cutting

2.2 Proposed development at the site

The proposed development comprises pile supported buildings with plan layout as shown in Figure 2-3. The buildings are to be supported on piled slab foundations. This comprises 600mm diameter CFA supported in the ground below the railway. The nearby ground investigation information this to comprise the Dublin Black Boulder Clay, which is a hard, highly competent bearing stratum.



Figure 2-3: Plan layout of proposed buildings and CFA piles

2.3 Future railway widening works

This assessment has allowed for potential future widening of the railway with steepened cut slopes of 70 degrees taken from the site boundary. These are shown at selected sections in

Figure 2-2. The sections show that the proposed arrangement allows the widening of the railway cutting by approx. 6.2 to 8.4m.

Note that Section C and Section I in Figure 2-2 have been used for analysis of the soil nailing and referred to as Section 1 and 2, respectively in the design, as described in Section 8 of this report.

3 PROPOSED SOIL NAIL REINFORCEMENT OF FUTURE RAILWAY WIDENING WORK

The proposed method to reinforce the steepened cut slopes is by soil nailing. This comprises steel bar tendons that are drilled and grouted into the face to provide mass stability to the slope.

Soil nails provide a mass stabilising method. This means that it does not rely on any individual nail but functions as a group effect to reinforce a slope and to allow steep cut slopes to remain stable in the long term. This gives flexibility in the soil nail layout to adjust for the location of obstructions, or in this instance CFA piles that support the proposed buildings. The as-built location of the piles can be recorded and the soil nail layout adjusted to avoid clashes, where required. AGL have provided a layout of soil nails that takes consideration of the pile locations with a minimum an offset distance of 0.5m from the soil nails to the edge of the pile.

A permanent facing comprising reinforced shotcrete that is secured to the face by the soil nails is proposed to retain the face, prevent shallow slips from between the nails and to prevent surface erosion. A shotcrete facing reduces the potential for future movement of the slope by provided a stiff facing element that spans between the soil nail heads and is not susceptible to erosion or ground loss between the soil nails,

The proposed works comprise the following elements:

- *Soil nails:* The proposed soil nails comprise hollow “self-drill” bars with 32mm external diameter that can be drilled using a sacrificial drill bit and a weak grout-water flush pumped through the hollow bar. A final thick grout is placed when the design length has been achieved. The soil nails are installed at a slight downward inclination to allow the grout to fully fill the hole and for design efficiency.
 - Soil nails to comprise R32-280 bars installed with 100 mm diameter drill bits. These are to comprise full length full length galvanised steel bars with further corrosion protection provided by grout cover and sacrificial steel.
 - Soil nail spacing is 1.25 vertical and 1.5m horizontal
 - Soil nails installed at 20 degrees downwards to the horizontal.
- *Nail bearing plates:* 300x300x20mm galvanised bearing plates, which are placed at the outside face of the shotcrete and secured to the nail head and slope face with a galvanised steel hemispherical washer and domed nut.
- *Shotcrete Facing:* A permanent facing 150mm thick with 1 No. layer of A393 steel mesh. Minimum lap of panels is 400mm. Shotcrete to comprise C40/50 shotcrete.
- *Drainage:*
 - 1 no. row of drilled in drains at +29.0mOD to +29.5mOD.
 - Length is 10m
 - Inclined upwards at 10°
 - Horizontal spacing = 3.0m
 - Comprises 60mm dia. slotted PVC piped wrapped in geosynthetic filter.

Full details of the soil nail, flexible facing and drainage are outlined in Sections 8 to 12.

4 INFORMATION PROVIDED

The following relevant information for the site was provided to AGL:

Ground Investigations:

- Ground Investigation Ireland (2025), Project No. 14499-02-25, titled 'Faussagh Avenue - Waste Classification Report', Rev A, dated 35 April 2025.

Drawings:

- Ait Urbanism & Landscape, AutoCAD Drawing No. 25D06-DR-1000, titled 'Cabra PBSA – Landscape Plan', dated 17th June 2025.
- CIE Property Boundary (AutoCAD Drawing)
- Topographic Map (AutoCAD Drawing)

5 Desk Study Information

A desk study has been carried out for the site at Faussagh Avenue, Cabra, Dublin 7. The key features which are considered relevant to the project are summarised below:

- The GSI Quaternary map shows the site to be underlain by **Till derived from Limestones**.
- The GSI 1:100,000 bedrock geology map shows the solid geology at the site comprises of **dark limestone & shale ('calp') of the Lucan Formation**.
- The GSI karst database mapping has not identified karst features to be present in the vicinity of the site.
- The Bedrock Aquifer map shows the site to be located within a **Locally Important Aquifer** which is described as **Bedrock which is Moderately Productive only in Local Zones**.
- The GSI Groundwater Vulnerability map shows the area to be of **Low (L)** vulnerability.
- The 6-inch first edition (1829–1841) historical maps show the site to be undeveloped.
- The 25-inch (Year 1897 – 1913) historical map shows the site to be undeveloped with a railway line running along the western boundary of the site.
- Aerial photographs dating back to 2002 show the site configured as it appears today.

6 GROUND CONDITIONS AND INVESTIGATION

6.1 Ground Investigation Information

The following ground investigation information has been provided to AGL for the site:

- Ground Investigation Ireland (2025), Project No. 14499-02-25, titled 'Faussagh Avenue - Waste Classification Report', Rev A, dated 35 April 2025.

Due to the limited site-specific ground investigation information available in the provided report, the following ground investigation reports from the GSI database have also been used:

- IGSL (1990), Report No. 940/1399, titled 'Site Investigation at Cabra Road, Dublin', dated October 1990.
- IGSL (1983), Report No. 19/2604, titled 'Site Investigation at Bachelors, Cabra, Dublin', dated August 1983.
- IGSL (2006), Report No. 11697, titled 'Site Investigation – Quarry Road, Cabra', dated April 2006.

The location of the GII (2025) SI locations is given below on Figure 6-1 and a combined location map of the GSI database ground investigations is shown in Figure 6-2.

The borehole logs obtained from the GSI database are included in Appendix B.

The ground conditions encountered in the exploratory holes were found to comprise the following strata:

- Topsoil
- Made Ground
- Brown Boulder Clay
- Black Boulder Clay

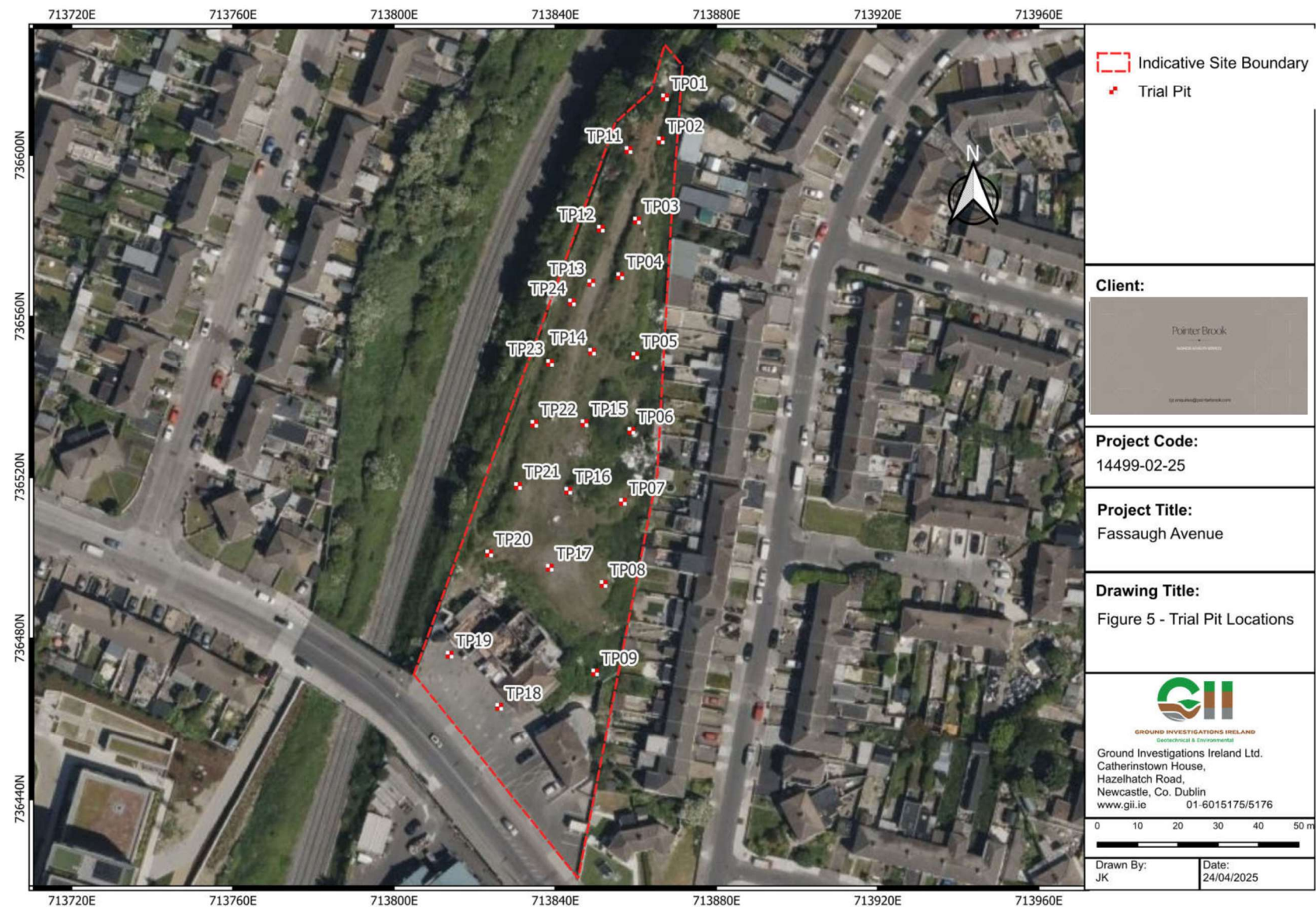


Figure 6-1: Location of the GI points and slope face (GII, 2020)

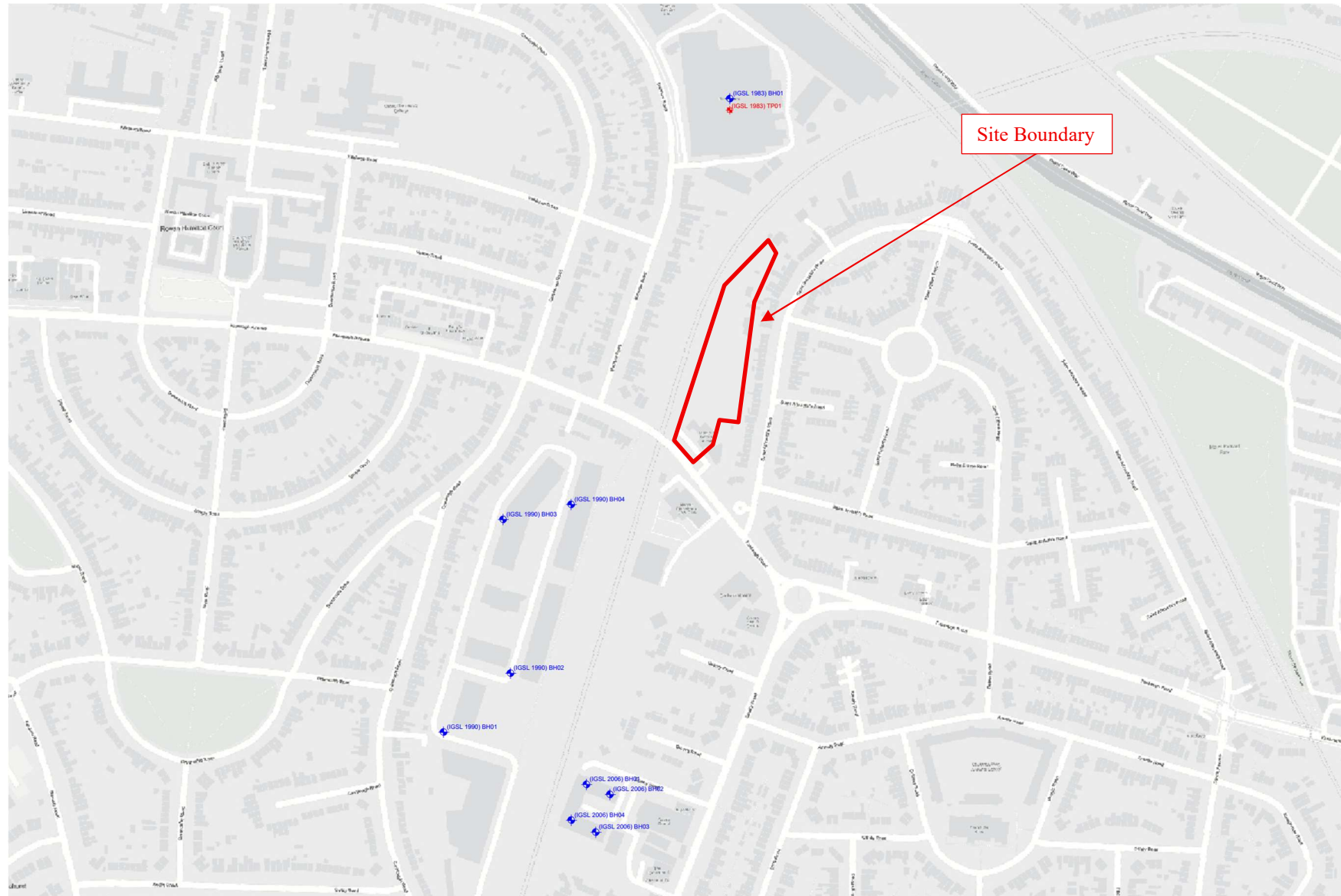


Figure 6-2: Combined Location Plan of GSI database Ground Investigations

6.2 Topsoil

A layer of Topsoil was encountered in most exploratory holes at ground level with a thickness of 0.2 to 0.35m.

No laboratory or in-situ testing was carried out on or within this layer.

6.3 Made Ground

Made ground was encountered in all exploratory holes at ground level or beneath the Topsoil at a depth of 0.2 to 0.35mbgl and with a thickness of 0.15 to 6.7m. The Made Ground stratum was described on the logs as:

- Dark brown slightly sandy slightly gravelly to gravelly CLAY with low to medium cobble and boulder content and rare fragments of plastic, ash, mortar, ceramic, red brick, concrete, glass, porcelain tile, geotextile, metal wire, rootlets, timber, organics and rubber.

3 No. moisture content tests were carried out on the Made Ground with values recording 23.8 to 36.2%. 2 No. Atterberg tests were carried out with values recording liquid limits= 28 to 35%, plastic limit= 18 to 22% and plastic index= 10 to 13%. This classifies the Made Ground as a CLAY of low plasticity.

10 No. Standard penetration tests (SPT) reported N-values of 9 to 46. This corresponds to an undrained shear strength of 45 kPa to 230 kPa using the empirical expression $c_u = 5 \times N_{SPT}$ (Stroud, 1989). This classifies the stratum as medium to very high strength material in accordance with ISO 14688-2.

6.4 Brown Boulder Clay

Brown Boulder Clay was encountered in most exploratory holes beneath the Made Ground at a depth of 0.15 to 6.7mbgl with a proven thickness of 1.7 to 3.2m. The Brown Boulder Clay stratum was described on the logs as:

- Firm to stiff brown slightly sandy to sandy slightly gravelly to very gravelly silty CLAY with occasional cobbles and boulders.

5 No. moisture content tests were carried out on the Brown Boulder Clay with values recording 12.2 to 14.05%. 5 No. Atterberg tests were carried out with values recording liquid limits= 24 to 30%, plastic limit= 11 to 16% and plastic index= 12 to 15%. This classifies the Brown Boulder Clay as a CLAY of low plasticity.

12 No. Standard penetration tests (SPT) reported N-values of 13 to 26. This corresponds to an undrained shear strength of 65 kPa to 130 kPa using the empirical expression $c_u = 5 \times N_{SPT}$ (Stroud, 1989). This classifies the stratum as medium to high strength material in accordance with ISO 14688-2.

1 No. Particle size distribution test on samples of Brown Boulder Clay reported a fines content of 31.2%, sand content of 23.7%, gravel content of 45.1%. Based on these fractions, the

material would be described as a slightly sandy gravelly CLAY in accordance with BS5930:2015.

6.5 Black Boulder Clay

Black Boulder Clay was encountered in all boreholes beneath the Brown Boulder Clay at a depth of 2.1 to 9.0mbgl. The Black Boulder Clay stratum was described on the logs as:

- Stiff to hard grey black sandy very gravelly silty to very silty CLAY with occasional cobbles and boulders.

4 No. moisture content tests were carried out on the Black Boulder Clay with values recording 8.82 to 10.2%. 4 No. Atterberg tests were carried out with values recording liquid limits= 19 to 26%, plastic limit= 9 to 15% and plastic index= 9 to 14%. This classifies the Black Boulder Clay as a CLAY of low plasticity.

25 No. Standard penetration tests (SPT) reported N-values of 38 to refusal (refusal = 100). This corresponds to an undrained shear strength of 190 kPa to >500 kPa using the empirical expression $c_u = 5 \times N_{SPT}$ (Stroud, 1989). This classifies the stratum as high to extremely high strength material in accordance with ISO 14688-2.

1 No. Particle size distribution test on samples of Black Boulder Clay reported a fines content of 30.4%, sand content of 21.3%, gravel content of 48.3%. Based on these fractions, the material would be described as a slightly sandy gravelly CLAY in accordance with BS5930:2015.

6.6 Groundwater

4 No. groundwater monitoring standpipes were installed in the IGSL (2006) ground investigation. Table 6-1 summarises the recorded groundwater monitoring result. Groundwater depths of 4.0 to 6.0 mbgl were reported.

Table 6-1: Groundwater monitoring in standpipes IGSL (2006)

Borehole	Response zone (mbgl)	Depth (mbgl)	Elevation* (mOD)	Date
(IGSL 2006) BH01	1.0 to 4.9	4.6	-	18/04/2006
(IGSL 2006) BH02	1.0 to 4.8	4.0	-	18/04/2006
(IGSL 2006) BH03	1.0 to 5.3	4.0	-	20/04/2006
(IGSL 2006) BH04	1.0 to 6.2	6.0	-	19/04/2006

* No elevations were provided.

Groundwater strikes for all ground investigations is summarised in Table 6-2. Groundwater strike depths of 2.5 to 6.2mbgl were reported. Exploratory holes were reported as dry in the IGSL (1983), IGSL (1990) and GII (2025) ground investigations.

Table 6-2: Groundwater strike summary

SI ID	Depth (mbgl)	Elevation* (mOD)	Comments
(IGSL 2006) BH01	2.6	-	Rose to 2.1mbgl
(IGSL 2006) BH01	4.9	-	Rose to 3.6mbgl
(IGSL 2006) BH02	2.6	-	Rose to 1.7mbgl
(IGSL 2006) BH02	4.8	-	Rose to 3.2mbgl
(IGSL 2006) BH03	5.2	-	Rose to 3.7mbgl
(IGSL 2006) BH04	2.5	-	Rose to 2.1mbgl
(IGSL 2006) BH04	6.2	-	Rose to 2.1mbgl

* No elevations were provided.

6.7 Soil and groundwater chemistry

The water-soluble sulphate (SO₄) of soil samples was measured to be in the range 0.5 to 660.0 mg/l and the pH range was 6.58 to 8.68.

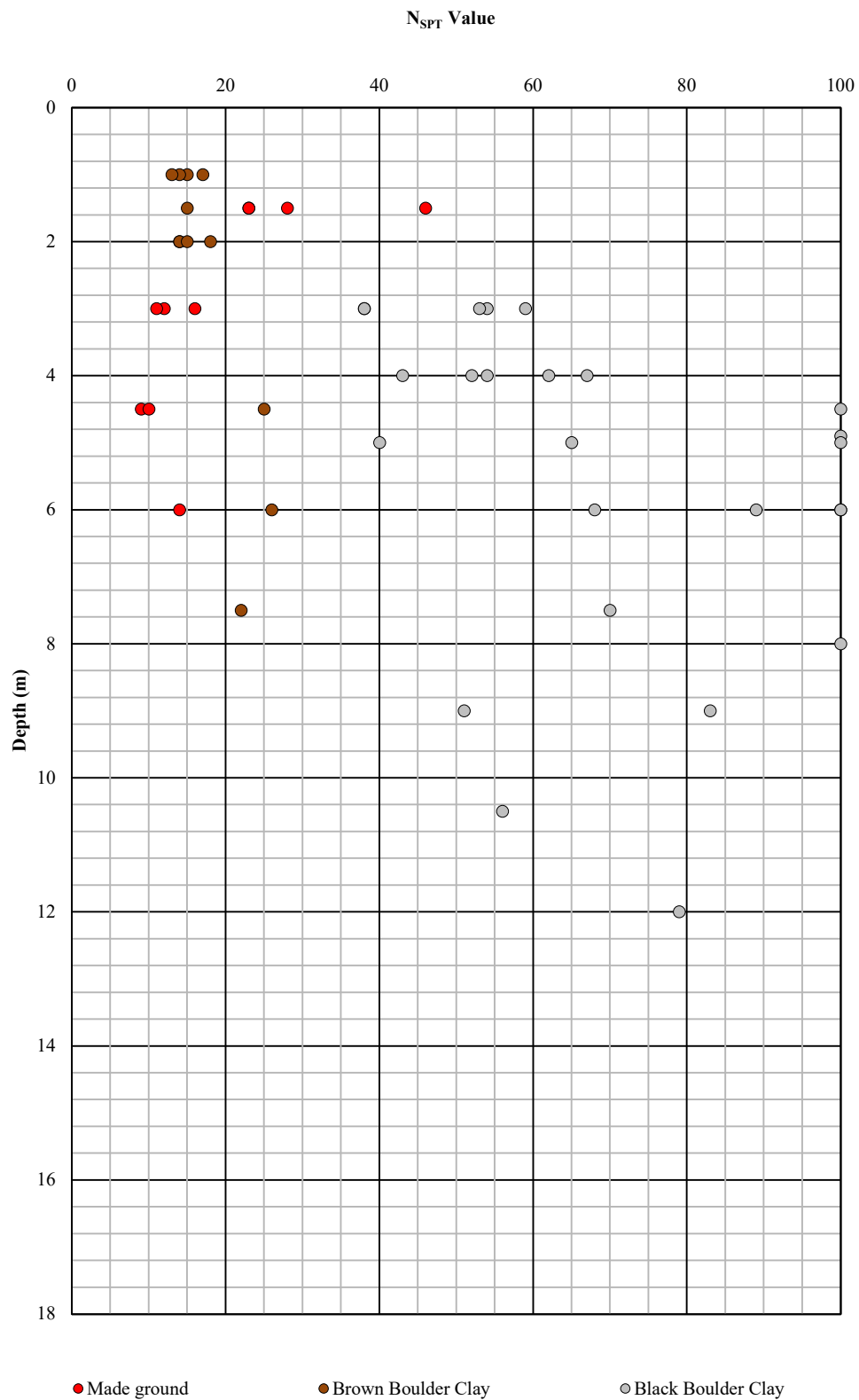


Figure 6-3 N_{SPT} vs depth

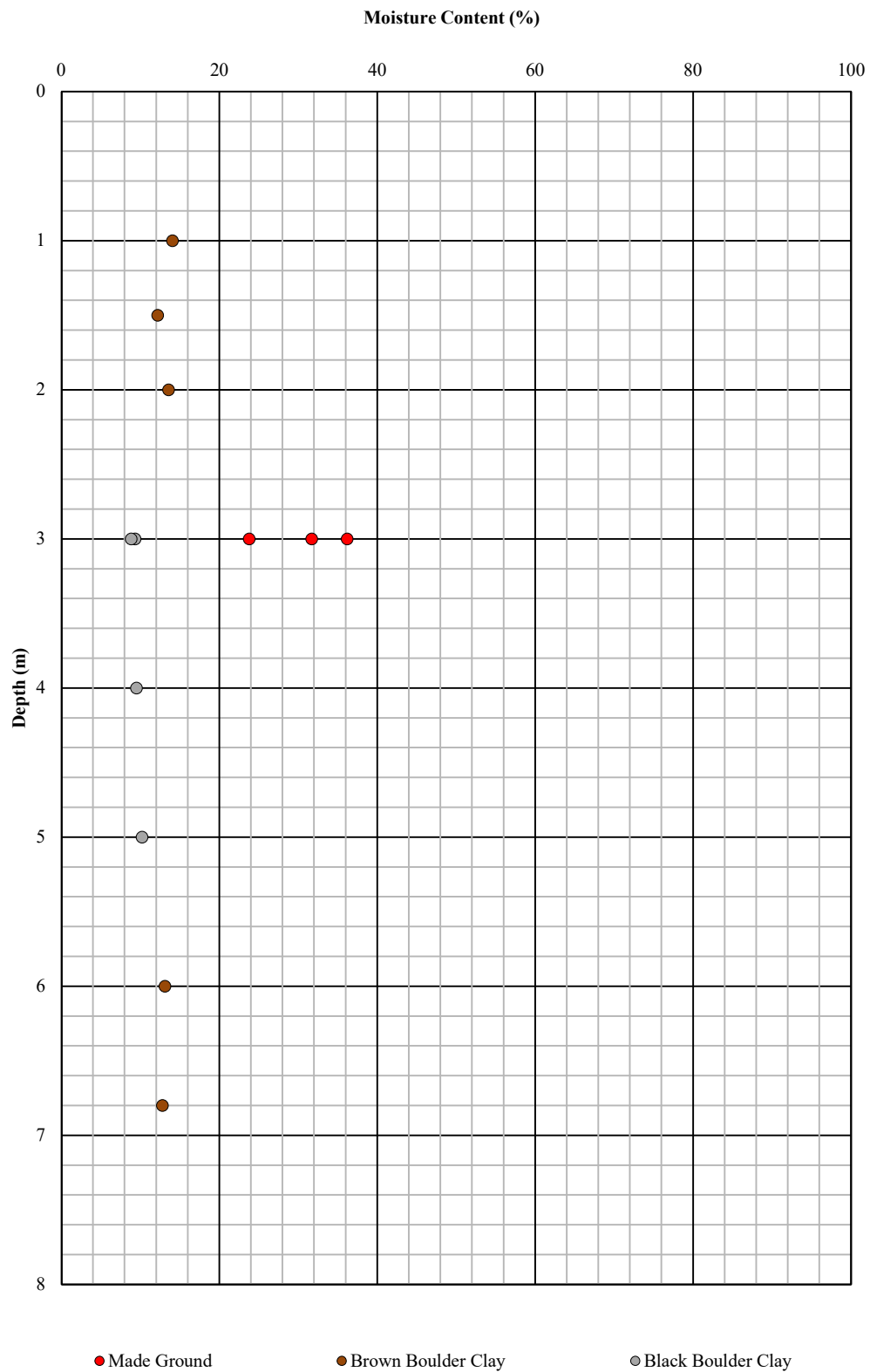


Figure 6-4 Moisture content vs depth

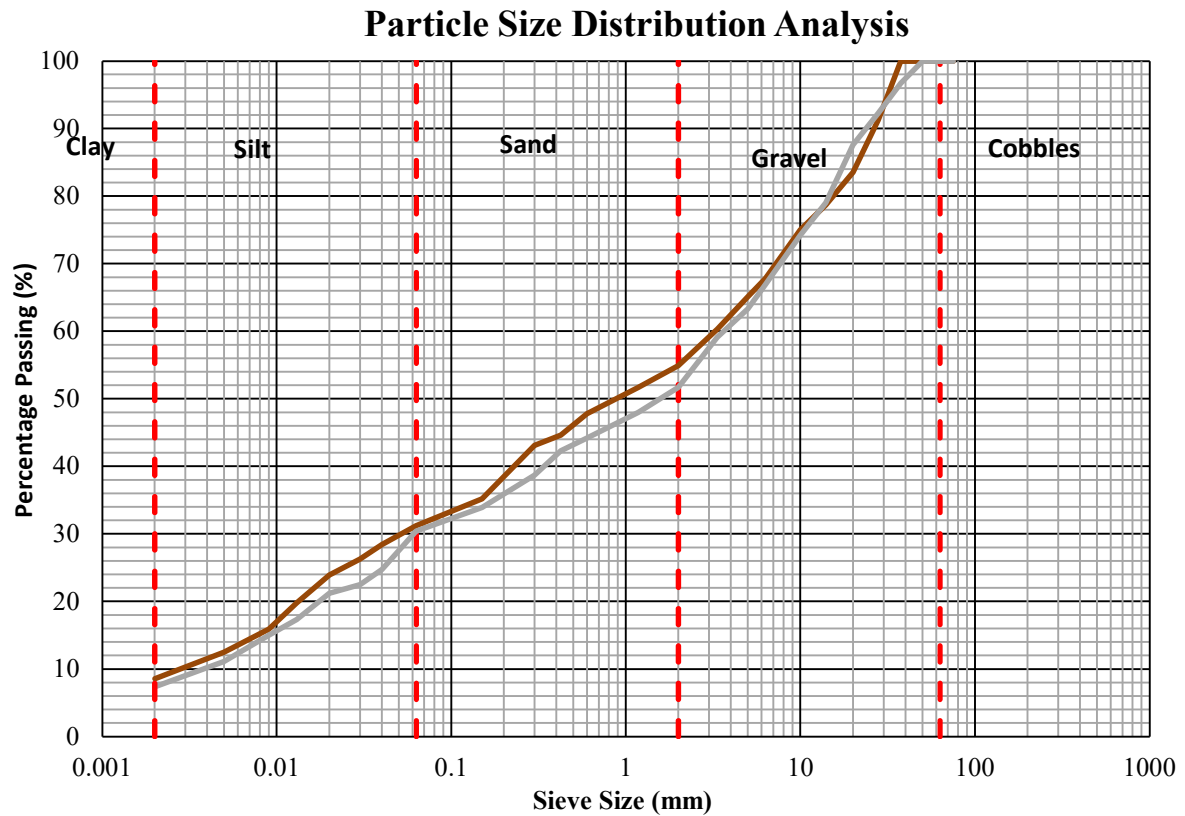


Figure 6-5: Particle size distribution

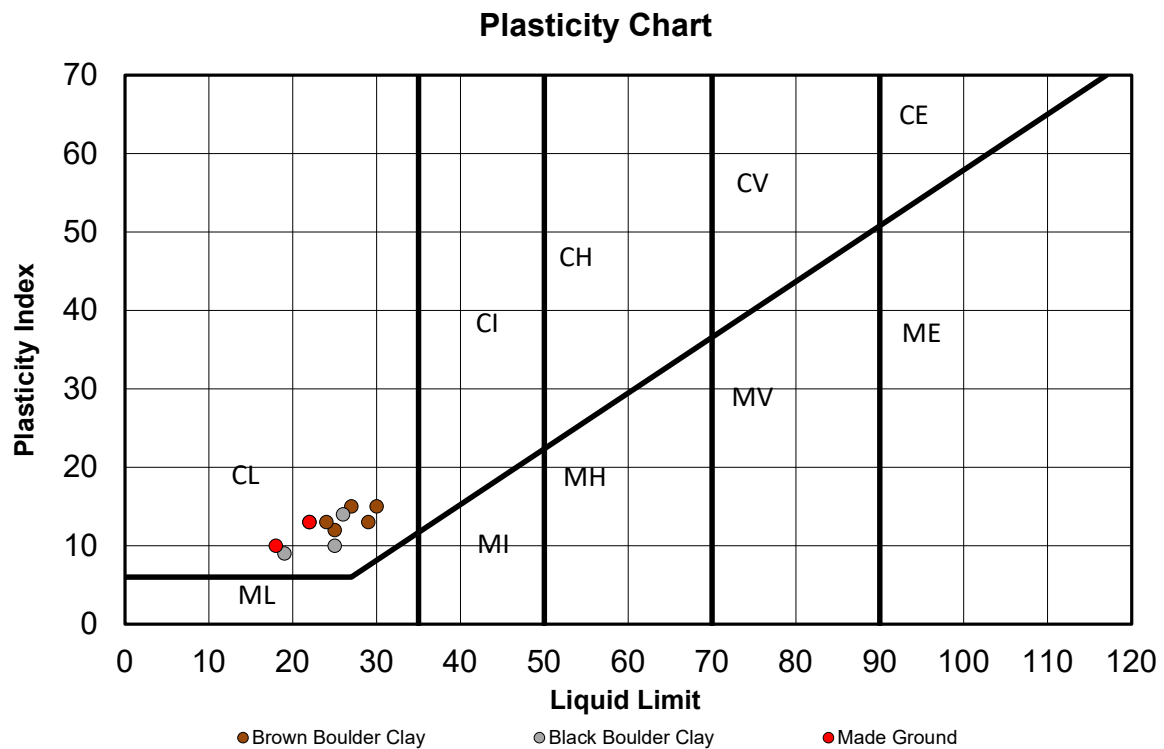
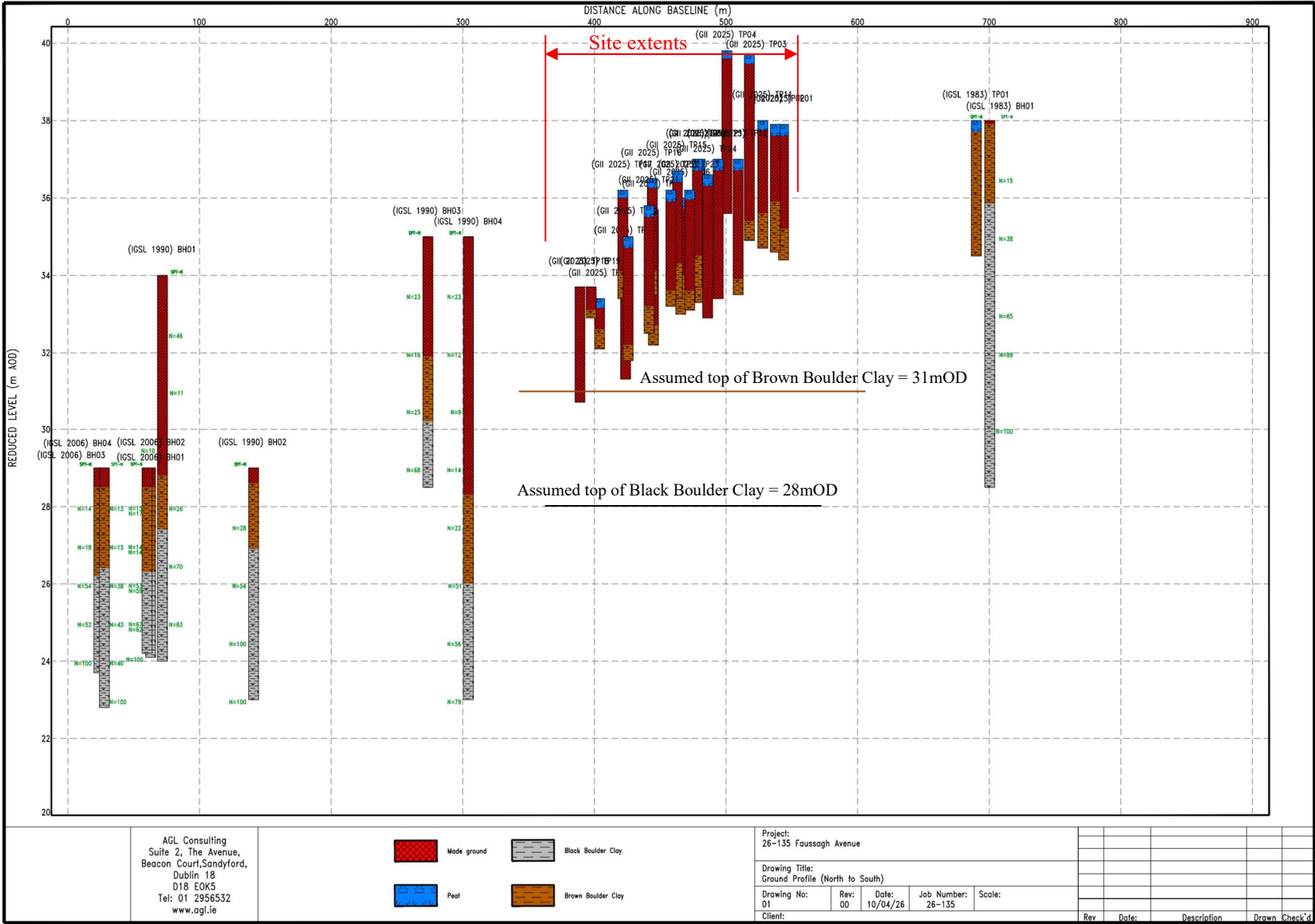


Figure 6-6: Plasticity Chart



7 CHARACTERISTIC PARAMETERS AND GROUND MODELS

7.1 Characteristic Soil Parameters

The characteristic properties for each stratum have been derived using the relevant corresponding lab test data. Where no data are available, conservative assumed values have been assigned which are based on experience and engineering judgement.

The characteristic material properties are determined for the following strata which are deemed relevant to the design of the secant pile wall:

- Made Ground
- Brown Boulder Clay
- Black Boulder Clay

7.2 Made Ground

Made ground was encountered in all exploratory holes at ground level or beneath the Topsoil at a depth of 0.2 to 0.35mbgl and with a thickness of 0.15 to 6.7m. The Made Ground stratum was described on the logs as dark brown slightly sandy slightly gravelly to gravelly CLAY with low to medium cobble and boulder content and rare fragments of plastic, ash, mortar, ceramic, red brick, concrete, glass, porcelain tile, geotextile, metal wire, rootlets, timber, organics and rubber.

No bulk density tests were carried out on samples of Made Ground. The SPTs in the Made Ground range from 9 to 46, which equates to an undrained shear strength $c_u = 45\text{kPa}$ to 230kPa . Therefore, the following cautious characteristic bulk unit weight and undrained shear strength has been assigned to the stratum:

$$\gamma_b = 20 \text{ kN/m}^3$$

$$c_{u,k} = 50 \text{ kPa}$$

No effective stress tests were performed on samples of Made Ground. However, a characteristic drained effective shearing resistance, $\phi_k' = 30^\circ$ is considered to be representative.

7.3 Brown Boulder Clay

This stratum was described on the logs as firm to stiff brown slightly sandy to sandy slightly gravelly to very gravelly silty CLAY with occasional cobbles and boulders.

No bulk density tests were carried out on samples of Brown Boulder Clay. Standard penetration tests (SPT) of Brown Boulder Clay in boreholes reported N-values of 13 to 26. This corresponds to an undrained shear strength of 65 kPa to 130 kPa using the empirical expression $c_u = 5 \times N_{\text{SPT}}$ (Stroud, 1989). Therefore, the following cautious characteristic bulk unit weight and undrained shear strength has been assigned to the stratum:

$$\gamma_b = 21 \text{ kN/m}^3$$

$$c_{u,k} = 75 \text{ kPa}$$

No effective stress tests were performed on samples of Brown Boulder Clay. However, a characteristic drained effective shearing resistance, $\phi_k' = 32^\circ$ is considered to be representative based on previous experience and published data for the Dublin Brown Boulder Clay.

7.4 Black Boulder Clay

This stratum was described on the logs stiff to hard grey black sandy very gravelly silty to very silty CLAY with occasional cobbles and boulders.

No bulk density tests were carried out on samples of Black Boulder Clay. Standard penetration tests (SPT) of Black Boulder Clay in boreholes reported N-values of 38 to refusal (refusal = 100). This corresponds to an undrained shear strength of 190 kPa to 500 kPa using the empirical expression $c_u = 5 \times N_{SPT}$ (Stroud, 1989). Therefore, the following cautious characteristic bulk unit weight and undrained shear strength has been assigned to the stratum:

$$\gamma_b = 23 \text{ kN/m}^3$$

$$c_{u,k} = 250 \text{ kPa}$$

No effective stress tests were performed on samples of Black Boulder Clay. However, a characteristic drained effective shearing resistance, $\phi_k' = 38^\circ$ is considered to be representative based on previous experience and published data for the Dublin Black Boulder Clay.

7.5 Summary of characteristic soil parameters

The characteristic soil properties that have been used in design are presented on Table 7-1.

Table 7-1 Summary of characteristic soil parameters

Stratum	Bulk Unit Weight (kN/m ³)	Effective Angle of shearing resistance, ϕ' (°)	Undrained Shear Strength (kPa)
Made Ground	20	30	50
Brown Boulder Clay	21	32	75
Black Boulder Clay	23	38	250

7.7 Ground Model

The topographical survey shows the highest elevation is +39.84mOD located at the eastern side of the slope. The elevation at the crest of the slope is approx. +31.8 to +36.4mOD. The slope extends to the railway line to the west to an elevation of circa +27.4 to +27.9mOD.

Ground model adopted for the analysis is presented in Table 7-2.

Table 7-2: Ground model stratigraphy GM-1

Stratum	Top of the stratum (mbgl)	Elevation at Top of Stratum (mOD)	Thickness (m)
Made Ground	0.0	+39.8	8.8
Brown Boulder Clay	8.8	+31.0	3.0
Black Boulder Clay	11.8	+28.0	-

7.8 Groundwater Model

No site-specific groundwater was encountered.

Nearby ground investigations recorded groundwater monitoring for the site to be between 4.0 to 6.0 mbgl. Groundwater strikes were recorded at 2.5 to 6.2mbgl.

The long term groundwater level has been conservatively taken at +31mOD (3-4mbgl) at the proposed site close to the crest of the slope and then falls to the toe of the slope, as shown in Figure 8-2. This is considered to be cautious given the highest groundwater recorded was 4.0mbgl.

Drilled-in drains will be installed in the slope at +29.0 to +29.5mOD to assist with drawdown of the groundwater level should a rise occur and to intercept potential perched groundwater.

8 SOIL NAIL DESIGN ANALYSIS

8.1 Method of analysis and design assumptions

The analyses have been carried out in accordance with the principles of IS EN1997-1:2005 (EC7) for the Ultimate Limit State and the Serviceability Limit states as outlined below. The bond skin friction of the soil nails has been calculated using characteristic effective stress parameters with a as per BS8006-2:2011.

8.2 Geotechnical category

In accordance with IS EN1997-1:2005 the proposed geotechnical structure is classified as Geotechnical Category 2, which includes conventional structures with no abnormal risk.

8.2.1 Ultimate Limit State (ULS)

The ultimate limit state analyses for the permanent conditions have been carried out in accordance with Design Approach 3 of Eurocode 7: Geotechnical Design Part 1 - General Rules (IS EN 1997-1:2004). Using this approach, the design soil parameters are obtained by reducing the characteristic soil parameters by a partial factor and the design loads are obtained by increasing the characteristic loads by a partial factor. The partial factors for Design Approach 3 are presented in Table 8-1. A margin of safety > 1.0 is required for an analysis with the factored material parameters and actions.

The bond skin friction of the soil nails has been calculated using characteristic effective stress parameters with a partial factor of 1.5 applied to derive the design bond stress as per BS8006-2:2011. The calculations are presented on the spreadsheets included in Appendix D.

The characteristic soil parameters used in the analyses are presented in Table 7-1.

Table 8-1: Partial factors for EC7 DA3

	$\gamma_{G\text{unfav}}^*$	$\gamma_{Q\text{unfav}}^*$	$\gamma_{G\text{fav}}$	$\gamma_{Q\text{fav}}$	$\gamma_{c'}$	$\gamma_{\phi'}$	γ_{cu}
DA3	1.0	1.3	1.0	0	1.25	1.25	1.4

* For slope and overall stability analyses structural actions on the soil are treated as geotechnical actions.

The Design Situations that have been considered are presented in Section 8.3.

The following design assumptions have also been made:

- A partial factor on ultimate strength of soil nail = 1.15 on yield strength of steel (as per EC2 & BS8006).
- Grout comprises a Portland Cement grout with a minimum 28 day compressive strength of 40 N/mm².
- Bond skin friction of the soil nails has been calculated by taking the minimum obtained from characteristic effective stress parameters and total stress parameters (where relevant) with a partial factor of 1.5 applied to derive the design bond stress as per BS8006-2:2011
- Drained and undrained conditions are analysed.

The analysis was carried out using the slope stability program Slope/W (GeoStudio 2019 R2) which is a 2-D plane-strain limit equilibrium slope stability analysis program. Slope/W has a

facility for incorporating soil nails. The design capacity of the nails is distributed evenly over the full length of the nail and only that portion of the nail that falls outside the failure surface contributes to the stability of the slope. The design was checked to ensure that there was an adequate margin of safety for internal and global stability, i.e., where the potential failure surface passes through or outside the soil nailed zone. Conservatively, the shear strength of the nails has been ignored.

The failure surfaces which resulted in the minimum calculated margin of safety for these conditions are included in Section 8.5. Note that a margin of safety > 1.0 is acceptable.

8.3 Design Situations & Soil Nail Design

All design situations consider a 70° slope angle cut from the boundary of the site down towards the railway line. The soil nailed slope design has considered the most onerous conditions, including maximum slope height, proximity of the future buildings and an existing retaining wall. This comprises two section profiles, referred to as Sections 1 and 2. The location of Sections 1 and 2 are shown in Figure 8-1.

- Section 2 considers the maximum slope height and shortest distance from the crest of the slope to the edge of the building, which is 4m.
- Section 1 includes an existing low retaining wall of 1 to 1.5m in height which is just outside the property boundary. To ensure the future stability of the wall, which is at the crest of the 70° slope, it is proposed to install a row of soil nails through the face of the wall. Depending on the condition of the wall the shotcrete facing could also extend over the face of the wall to ensure its structural longevity.

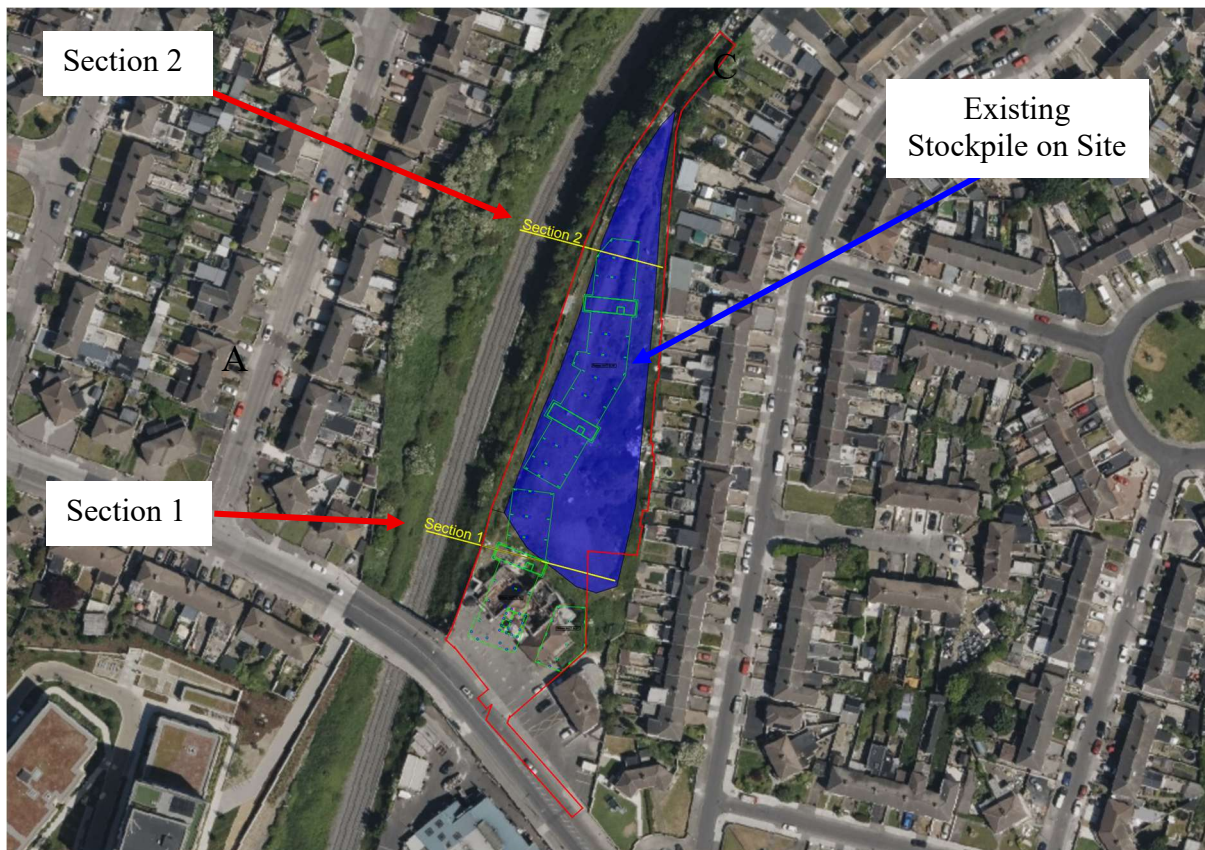
The details of the soil nail rows, and nail lengths are summarised on Table 8-2.

The soil nail elevations, spacing and length to give the required margin of safety in accordance with EC7 DA3 have been determined from slope stability analysis, as described in Section 8.2.1.

For Section profile, 2 No. cases were analysed, each with the current condition of the site with the stockpile of made ground in place (referred to as “Current Ground Profile”) and when the construction of the proposed development is complete (referred to as “Future Ground Profile”). This allows for lowering of ground levels below the buildings to the underside of the piled slab with a constant elevation level.

Table 8-2: Summary of Design Situations Analysed

Relevant Design Situation	Section	Ground Model Condition	Maximum Slope Elevation (mOD)	Max. Slope Height (m)	Average Slope Angle (°)
DS-1	1	Current Ground Profile – Ground model to reflect current conditions on site	+33.9	5.9	70
DS-2	1	Future Ground Profile – Ground model to reflection conditions on site after construction	+33.9	5.9	70
DS-3	2	Current Ground Profile – Ground model to reflect current conditions on site	+34.7	6.7	70
DS-4	2	Future Ground Profile – Ground model to reflection conditions on site after construction	+34.7	6.7	70

**Figure 8-1: Location Plan of Section 1 & 2**

8.4 Surcharge loads

8.4.1 General surcharge

A characteristic construction plant/general surcharge of 10kPa was assumed at the top of the slope in all design situations to allow for vehicular traffic during or after construction.

The future building loads have not been included as these are supported on piles. A general surcharge load of 10 kPa has been included conservatively below the building footprint.

8.4.2 *Soil Nail Inclination*

A nail inclination of 20 degrees to the horizontal has been applied for all rows of soil nails.

8.4.3 *Soil Nail Layout and length*

The soil nail arrangement has been adjusted as follows to allow for the proposed buildings at the site:

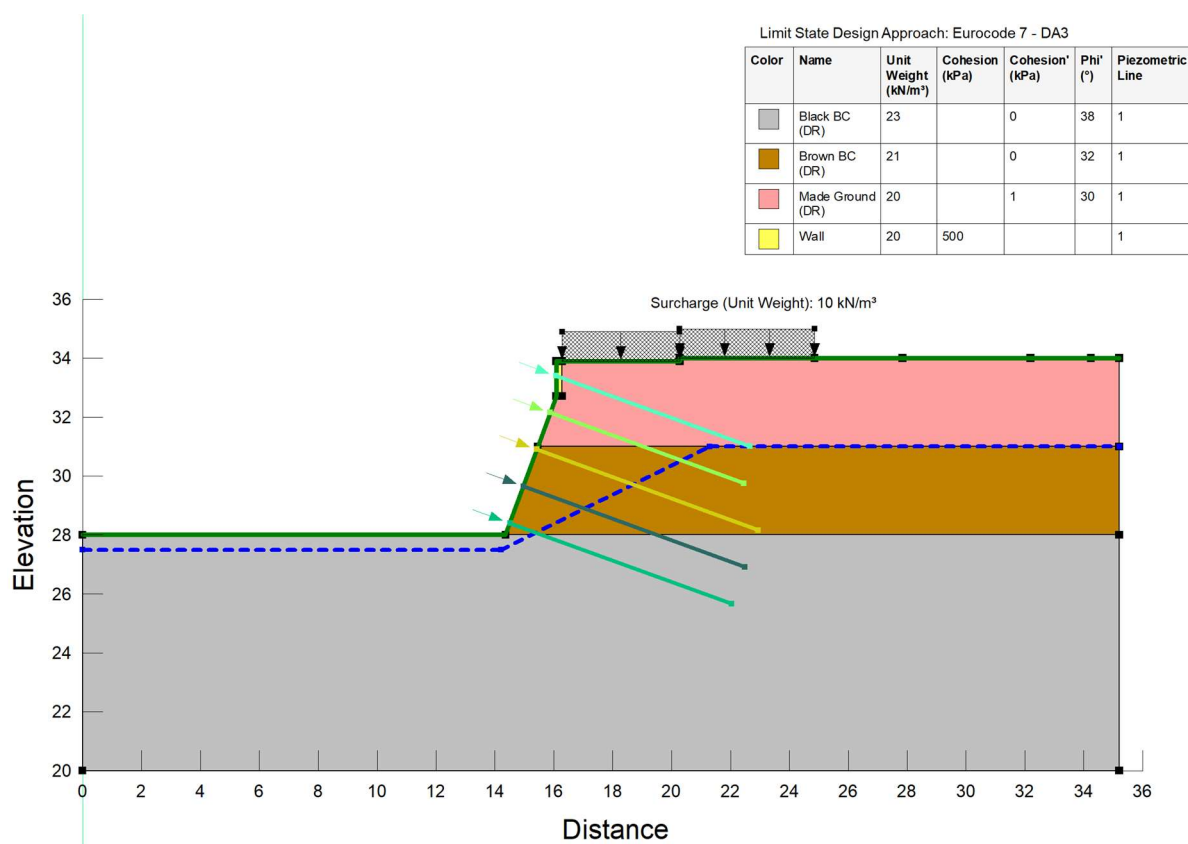
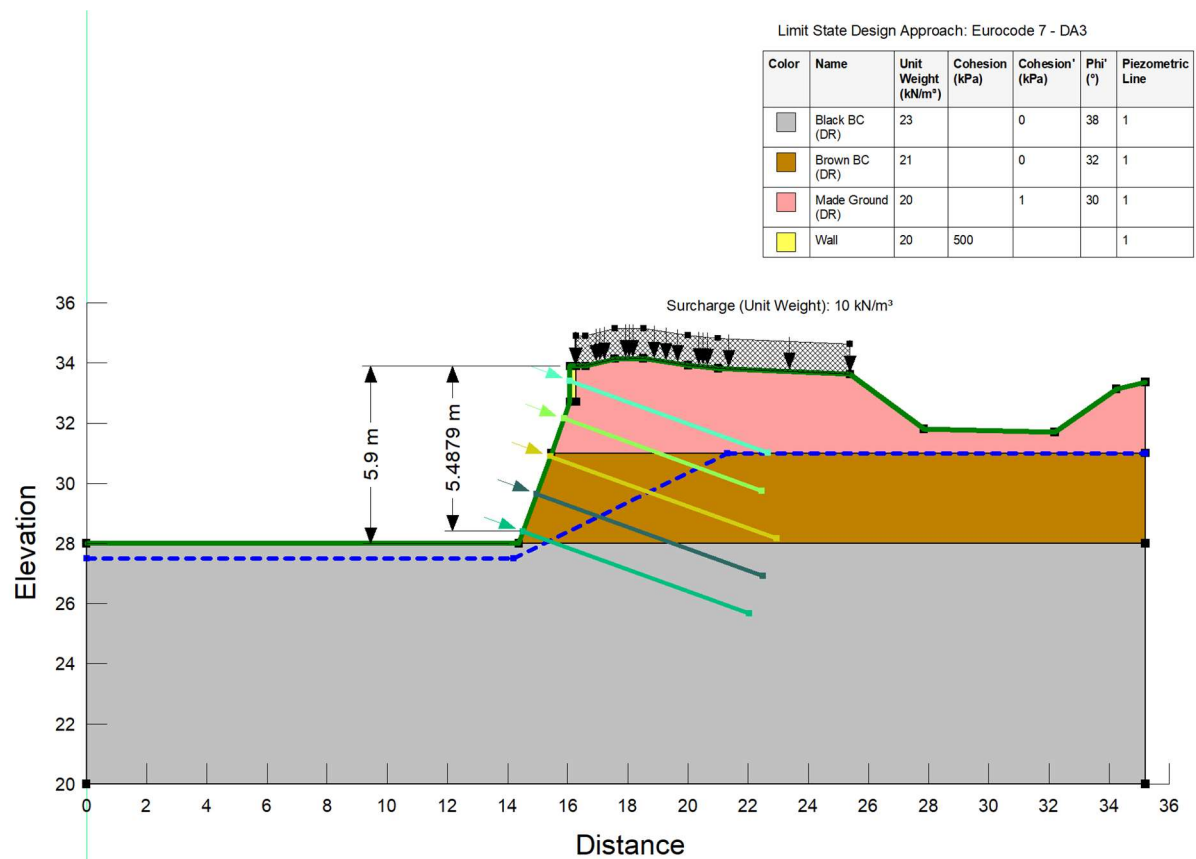
- A minimum offset distance of 0.5m from the centre of a soil nail to the edge of the 600mm diameter piles has been assumed. This locally increases the nail spacing to 1.6m for the soil nails located on either side of the piles.
- The maximum length of 7m has been assumed for the upper 2 No. nail rows. This ensures that the nail layout is only affected by a single line of CFA piles at the edge of the building. The soil nails are $\geq 0.5\text{m}$ from a second line of piles within the building. This ensures that only the “edge” piles need to be considered during soil nail installation.

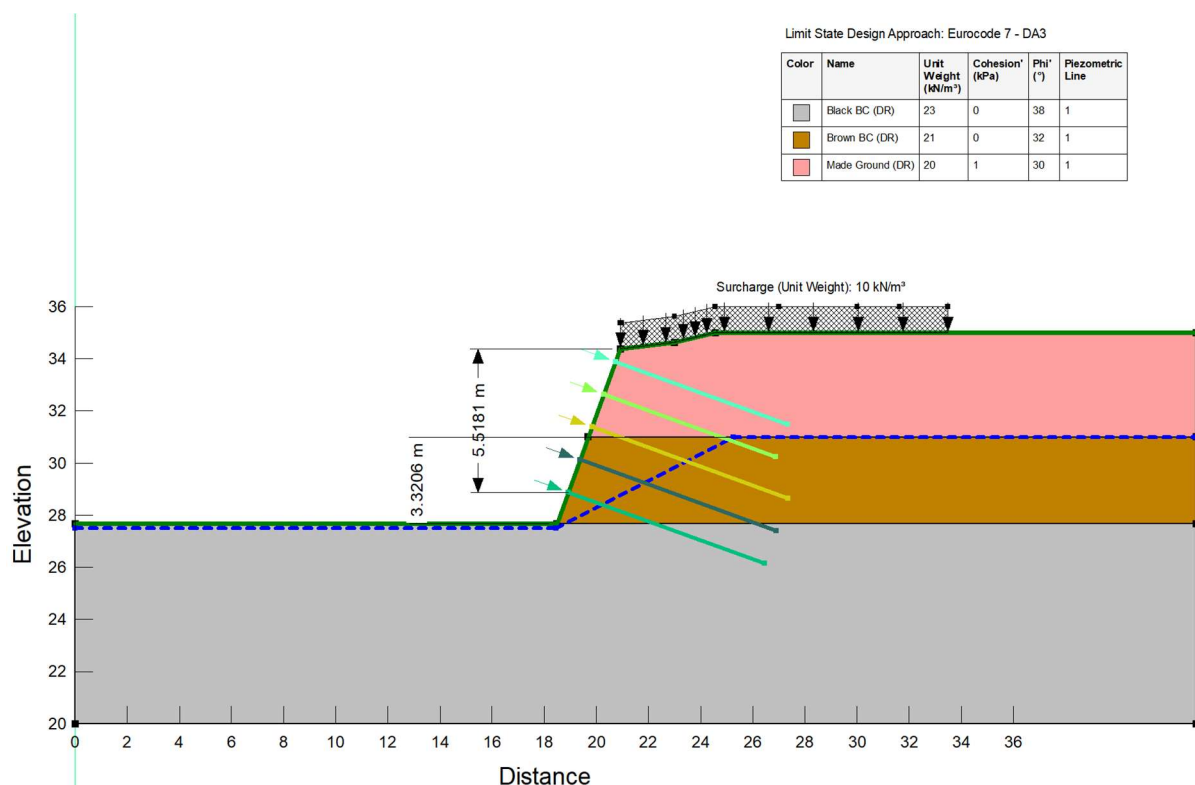
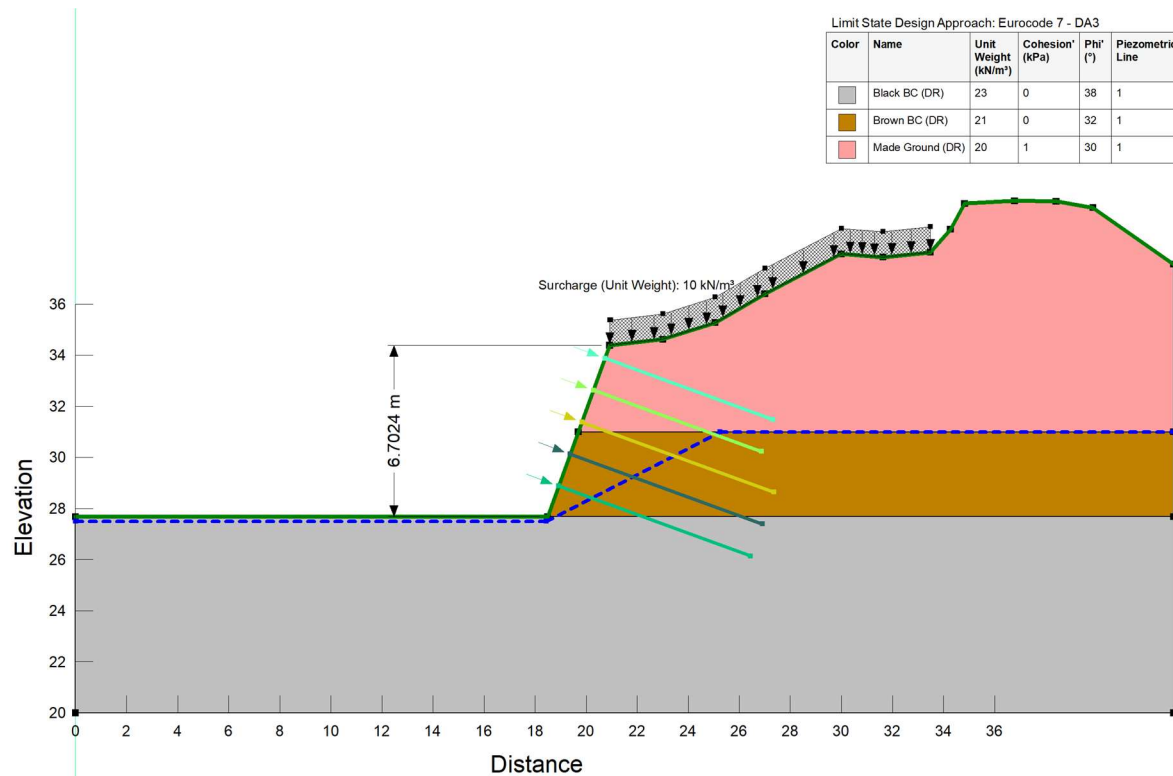
8.5 **Ultimate Limit State (ULS) Analysis & Results**

8.5.1 *Slope/W Input Model*

As mentioned in Section 8.2.1, the slope stability of the proposed design has been assessed using the slope stability program Slope/W. The stability of the soil nail slope has been assessed using the ground properties and design situations outlined in Section 7.5 and 8.3, respectively for both drained and undrained conditions.

The input models used for DS-1 to DS-4 are shown on Figure 8-2 to Figure 8-5. All input models are presented in Appendix E.





8.5.2 Slope/W Results

The results of the Slope/W analyses are presented in Appendix E and the Over Design Factor (ODF) is presented in Table 8-3. The margin of safety with factored parameters or the Over

Design Factor (ODF) for all the analyses was greater than 1.0 which indicates that the design complies with the requirements of EC7 for stability.

Note that very shallow slip surfaces in the slope face are ignored as these are supported by the facing.

The results for DS-1 to DS-4 are presented in Figure 8-8 to Figure 8-9 for drained conditions, which is the critical analysis. The results for all cases are presented in Appendix E.

Table 8-3: Over Design Factors for each slope analysed

Design Situation	Section	Over Design Factor	
		Drained	Undrained
1 (Current Ground Profile)	1	1.300	2.292
2 (Future Ground Profile)	1	1.325	2.331
3 (Current Ground Profile)	2	1.054	1.206
4 (Future Ground Profile)	2	1.228	1.234

It is found that that using the same soil nail arrangement the margin of safety increases for the “Future Ground Profile” compared to the “Current Ground Profile”. Therefore the Future Ground Profile is less onerous which potentially would allow shorter nail lengths, with greater efficiency. However, for simplicity the same nail arrangement for both current and future conditions have been considered.

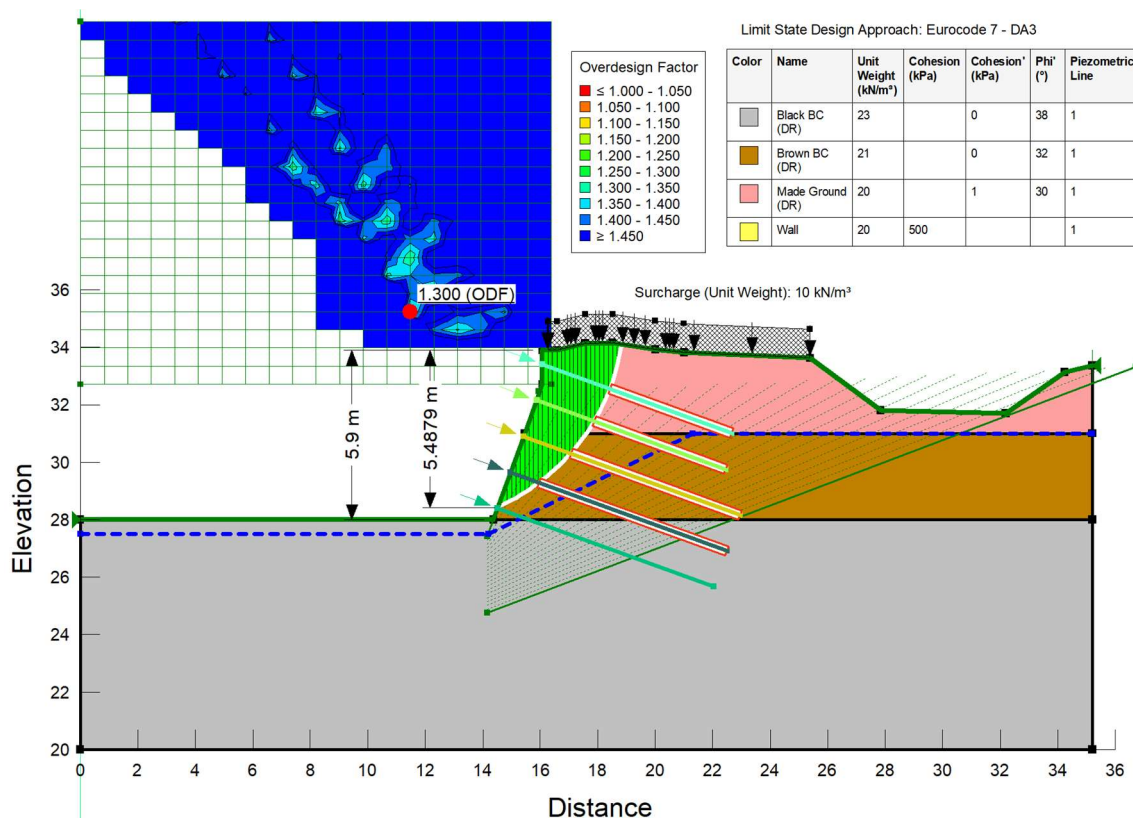


Figure 8-6:: Slope/W critical case results – Section 1 – Current Ground Profile (DS-1) - Drained

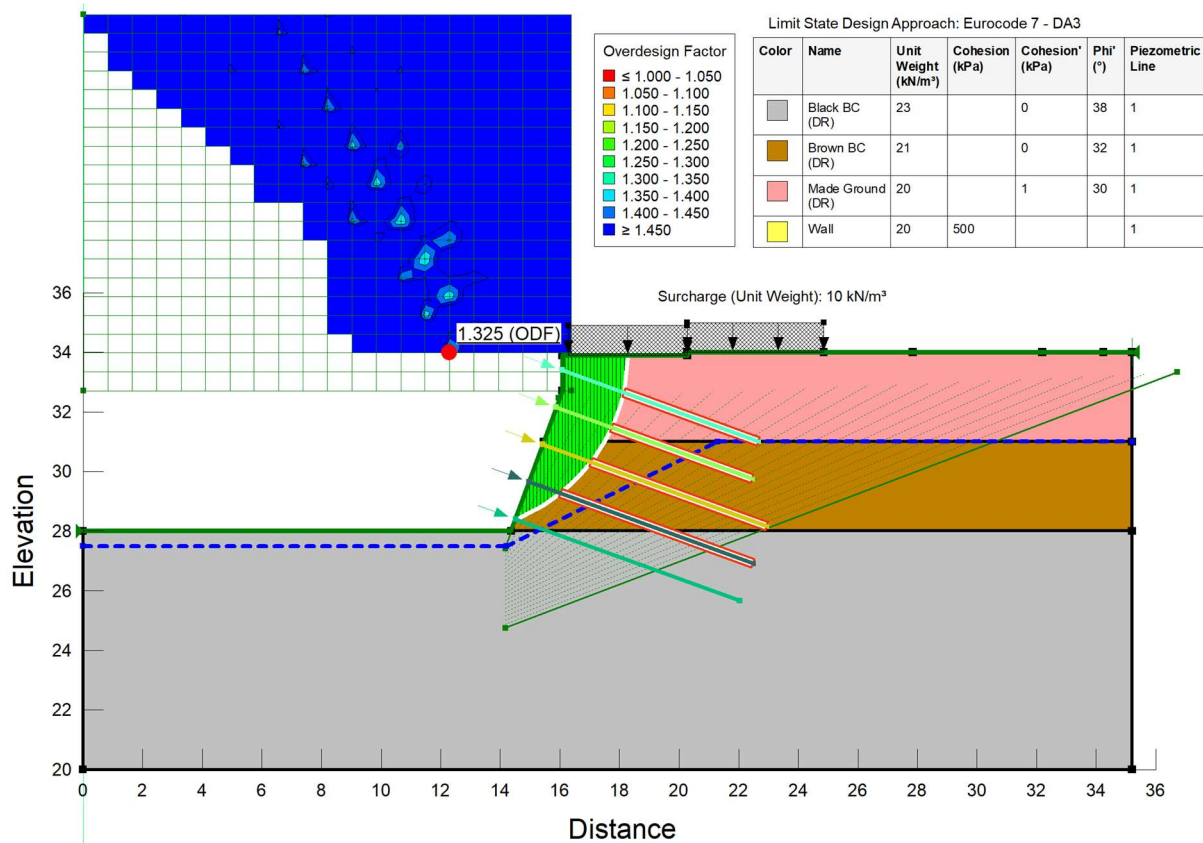


Figure 8-7: SlopeW Results – Section 1 – Future Ground Profile (DS-2) – Drained

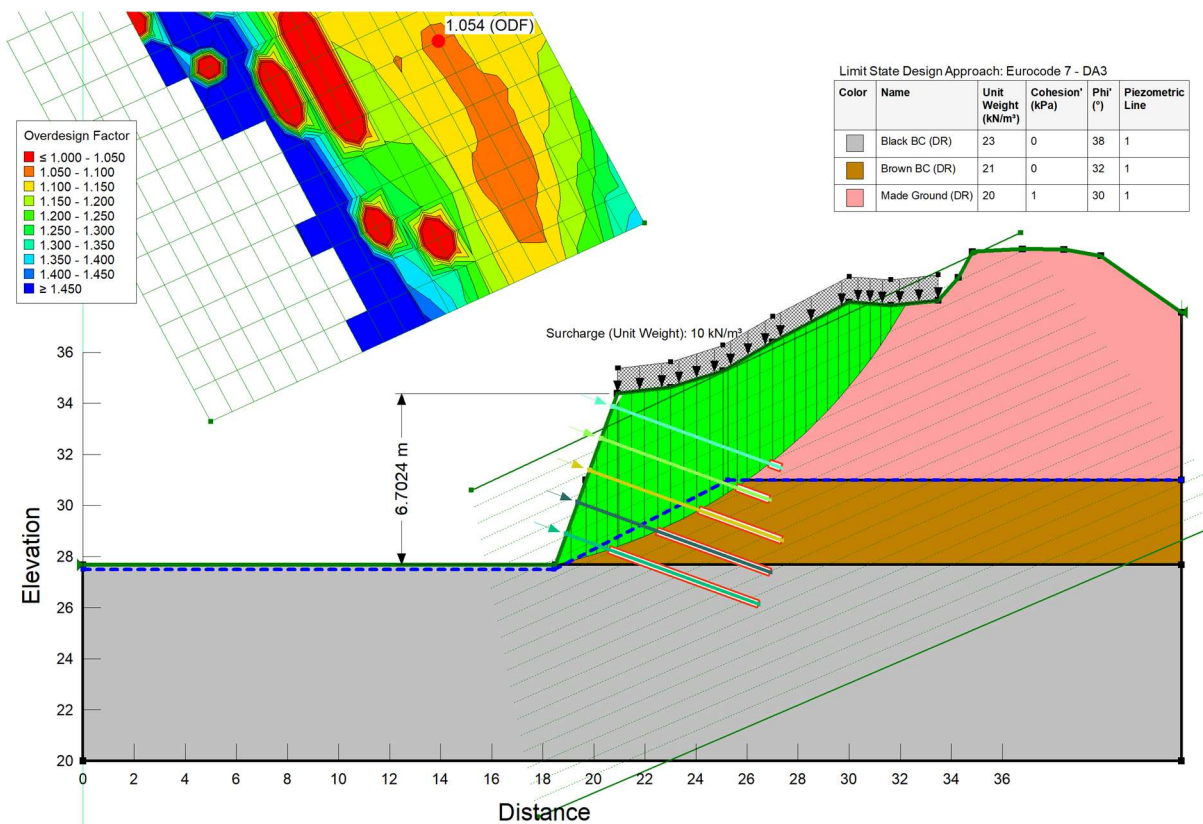


Figure 8-8: Slope/W critical case results – Section 2 (DS-3) – Current Ground Profile – Drained

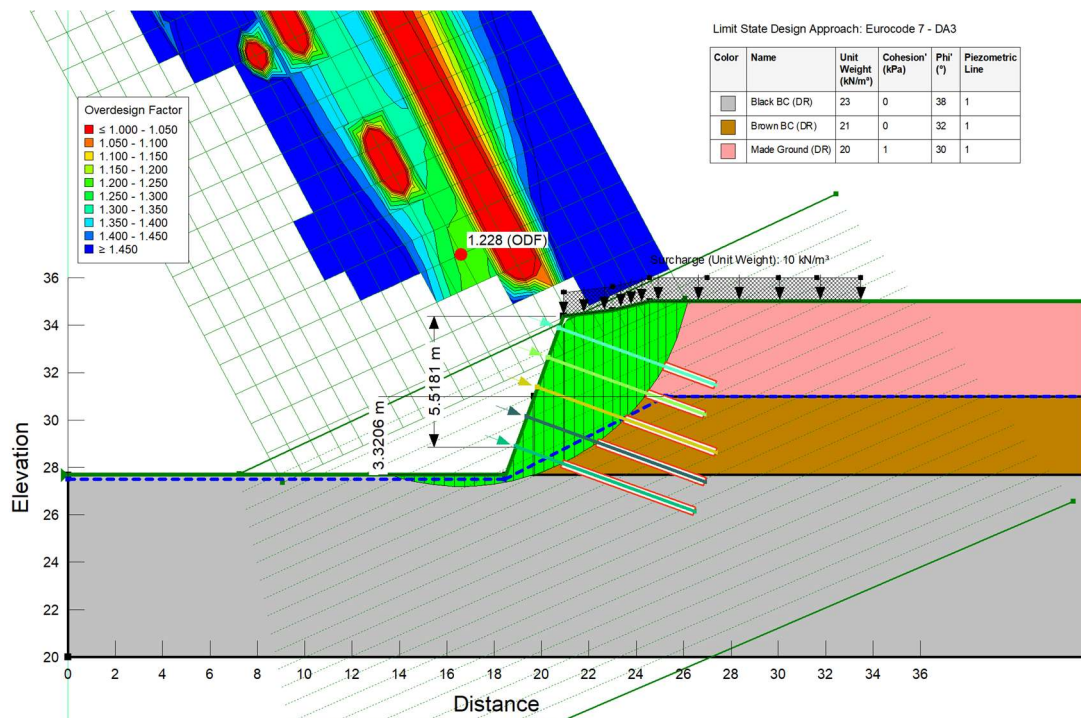


Figure 8-9: Slope/W critical case results – Section 2 (DS-) – Future Ground Profile – Drained

8.6 Serviceability Limit State (SLS)

The maximum horizontal and vertical deflections due to excavation of the soil nailed slope have been estimated using the empirical method of Clouterre (1993) and Byrne (1998) as recommended in FHWA-GEC-007 (2015) and CIRIA C637 (2005).

For vertical soil nail walls with typical nail-length-to-wall-height ratios of $L/H > 0.7$ designed with reasonable factors of safety, the peak wall displacements at the top of the wall tend to vary from 0.1 %H or less for weathered rocks and very competent and dense soils (e.g., glacial tills), to 0.2%H for granular soils, and up to 0.4%H for fine-grained clay type soils. These movements are considered to be relatively small and generally correspond to those expected for well braced systems and for tieback walls. Deformation also reduces for slopes with shallower face angles

Allowing for made ground and boulder clay to be present the maximum displacements have been estimated from $\delta_v = \delta_h = 0.15\% H$. This gives an estimated maximum settlement and lateral deflection of 10mm for a slope height of up to 6.7m.

The maximum movement occurs at the crest of the slope and reduce with distance from the slope as illustrated in Figure 8-10. The extend of ground movement from the slope crest is given by:

$$D_{DEF} = C(1 - \tan\alpha)H.$$

Taking $C = 1.0$ for firm to stiff soil, this gives $D_{DEF} = 4.2\text{m}$. Therefore the extend of movement is close to the wall and generally does not extend to the proposed buildings. The future buildings will be pile supported, so will be unaffected by expected ground movements for widening of the railway where soil nail reinforced slopes are used.

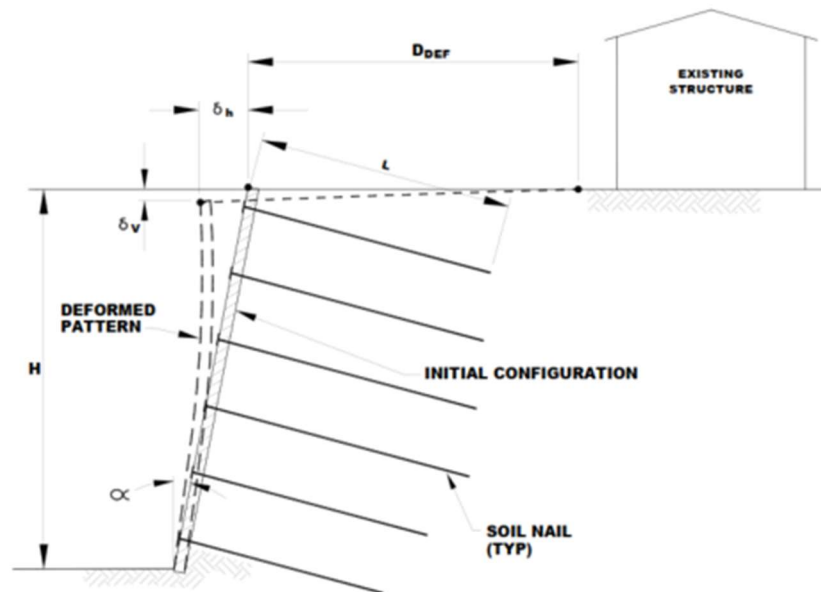


Figure 8-10: Deformation of soil nail wall (after Clouterre (1991) & Byren et al (1998) from FHWA-GEC-007 (2015))

8.7 Structural Design of Nails

8.7.1 Design for bar corrosion

The corrosiveness of the soil at the site has been assessed according to IS EN 14490:2010 *Execution of special geotechnical works - soil nailing* and TRL Report 380 (1993) *The development of specifications for soil nailing* and BS8006-2:2011 *Code of practice of strengthened/reinforced soils Part 2 Soil nail design*.

The assumed design life for the structure is 100 years.

Grout cover

The index value determined from Table B.2 of IS EN 14490:2010 is presented in Table 8-4. This gives a “slightly corrosive” or “non-aggressive” soil category (Classification IV). Table B.4 indicates that, for a 1000-year design life for an ungalvanized bar special consideration needs to be given for the grout cover.

Table 8-4 Corrosiveness assessment as per Table B.2 of IS EN 14490:2010

Criterion	Features	Weight A of criterion
Type of soil	Sandy gravelly clay	1
Resistivity	Estimated to be in the range 100-250 ohm-m (10,000 to 25,000 ohm-cm) based on O'Connor et al (2016) for glacial till.	0
Moisture content	Above water table and $w_n < 20\%$	0
pH	> 6	0
Global Index		1

The bar and coupler diameter for the R32-280 self-drilling bar is 32mm and 42mm, respectively. Therefore, the minimum grout cover in a 100 mm dia. hole will be 34mm and 29mm to the bar and couple, respectively. The grout cover achieved does not comply with grout cover required in Table B.4 for the design life of the structure (i.e., 100 years). Therefore, the grout cover has been ignored and allowance is made of loss of steel section due to corrosion. Note that due the method of drilling it would be expected that overbreak will occur and the drillhole diameter will typically exceed 100mm, so this assessment is conservative.

Loss of Steel due to Corrosion

The ranking value determined from Table 3 of TRL 380 is presented in Table 8-5. TRL380 indicates the ground conditions for the glacial till to be “mildly aggressive”. For a plain bar in a grouted hole TRL 380 allows the grout to be considered as enhancing the soil environment so that the requirements for sacrificial allowance may be reduced to a lower category of aggressiveness. For a 100 year design life and for non-aggressive soil type the sacrificial thickness is given by TRL 380 as 0.6mm per exposed surface as shown on Figure 8-11.

From BS8006-2:2011, assuming a typical pH in the range of 4.5 and 9 indicates neutral conditions in which degradation rates are controlled by cathodic reduction of oxygen and corrosion rates can be assessed from Table 3 of the standard. This gives a mean corrosion rate of 15 microns/year for unprotected undisturbed ground (except the proximal end). Allowing for a design life of 100 years this gives a steel loss of 1.5mm per side.

For soils of low aggressivity Dywidag recommends sacrificial corrosion of 1.4mm for a design life of 120 years. Allowing for >30mm grout cover and galvanised bar and a sacrificial steel loss of **1.4mm** has been cautiously allowed for to give a design life of 100 years.

Table 8-5 Corrosiveness assessment as per Table 3 of TRL 380

Criterion	Features	Weight A of criterion
Type of soil	Low plasticity sandy gravelly clay	-2
Groundwater level	Well drained area	+1
Resistivity	Estimated to be in the range 100-250 ohm-m (10,000 to 25,000 ohm-cm) based on O'Connor et al (2016) for glacial till.	-3
Moisture content	<20%	0
pH	>6	0
Soluble sulphate	40ppm (0.04 g/l)	0
Ranking Value		-4

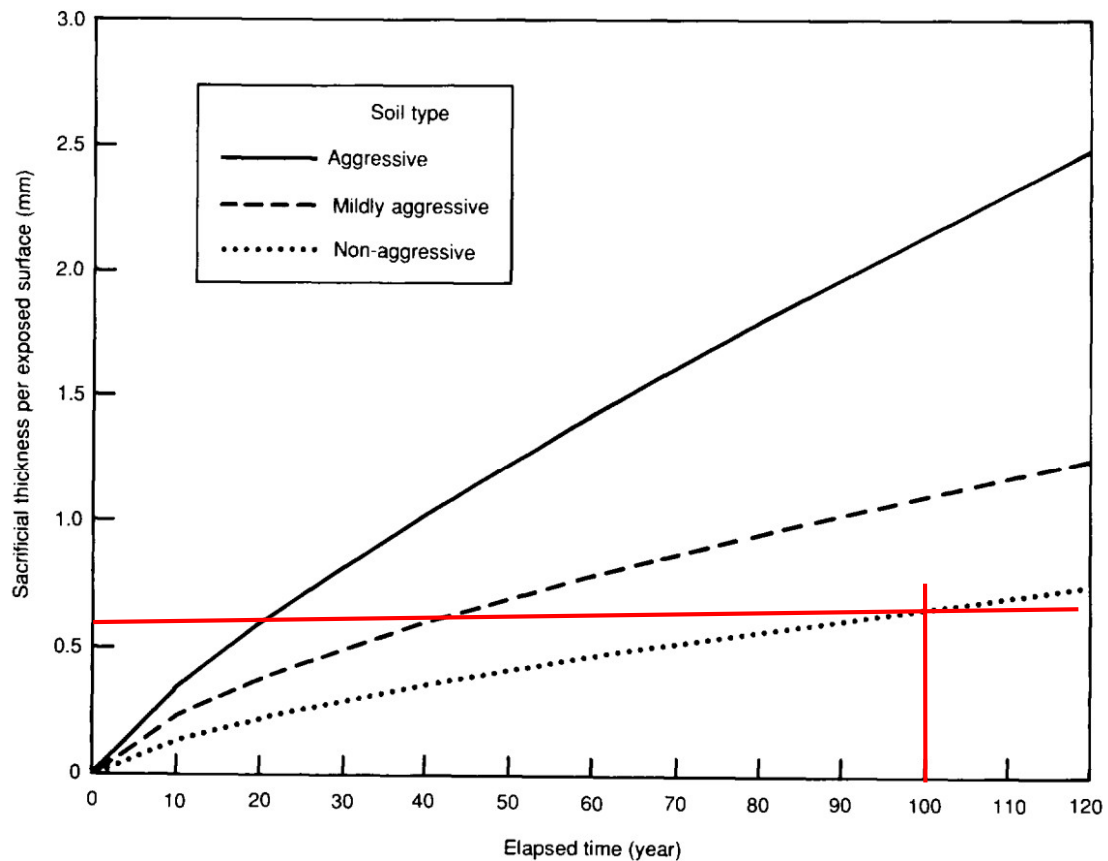


Figure 2. Sacrificial thickness of buried steel surfaces

Figure 8-11 Sacrificial thickness of buried steel surfaces (after TRL 380)

Conclusion

It is recommended that the soil nails be galvanised over their full length. In addition the nails have been cautiously designed for a loss of steel of 1.4mm due to corrosion for an ungalvanised bar for a 100-year design life (which requires no grout cover). These additional protective measures against corrosion and the grout cover of 29-34mm provides a robust design against loss of steel due to corrosion.

8.7.2 Bar properties

The self-drill hollow steel bars that are to be used are R32-280. The properties of the bar are presented in Table 8-6. The bars are to be galvanised to IS EN ISO 1461:2009 over their full length.

Table 8-6: Properties of soil nail bar

Bar Type	Nominal Thread Diameter (mm)	Internal Diameter (mm)	Ultimate Strength (kN)	Yield Strength (kN)	Steel Grade Yld / Ult (N/mm ²)	Corroded Effective External Diameter (mm) Note 2	Corroded Cross Section Area (mm ²) Note 2	Design yield strength, (kN) Note 1&2
R32-280	32	18	280	220	520 / 670	29.2	287.3	134.9

#Notes

1. Steel yield strength factor, γ_s , of 1.15 applied
2. Sacrificial corrosion based on design life of 100 years, the calculation sheet is attached in Appendix D.

The bar properties are also used in the Slope/W analysis, with the program checking the limiting values for the bar design tensile strength and geotechnical pull out.

8.7.3 Bar Rupture

The design load in soil nails has been checked to ensure it does not exceed the tensile resistance of the bar, and thus, rupture. The design pullout resistance of the soil nail, calculated using either effective stress or undrained parameters, is checked against the bar yield force.

The calculation method in Slope/W also takes into account the bar yield load in the calculations and the user can identify whether the nail structural resistance or geotechnical pull-out governs the design.

8.7.4 Grout/Tendon Interface

The ultimate bond stress at the Grout/Tendon interface is not to exceed 5000N/mm² for a clean threadbar with coarse pitch threadform according to BS 8081:2015 + A2:2018.

For a 32mm diameter bar this length bar this gives a pull-out capacity of the soil nail at the grout/tendon interface of 520 kN/m (i.e., per metre length). Allowing for a partial factor of 1.5 the pull-out resistance is 336 kN/m. Therefore, the grout/tendon interface does not control the pull-out resistance of the soil nails.

8.7.5 Grout-soil bond resistance

The bond skin friction and structural capacity of the soil nails have been calculated in accordance with BS8006-2:2011 using the spreadsheet included in Appendix D. The bond skin friction of the soil nails has been determined for drained conditions as follows:

$$\tau_{bu} = \lambda_f \times k_r \times \sigma_v' \times \tan(\phi_k) \text{ (kPa)}$$

Where,

- σ_v' = vertical effective stress. This is calculated at the $\frac{3}{4}$ length of the bar.

- ϕ_k = characteristic angle of shearing resistance
- k_r is a factor relating the average radial effective stress around the nail to the vertical and has a value typically in the range 0.55 to 0.9, depending on the relative density of the soil and degree of stress reduction due to slope movements in the active zone of the slope. A value of 1.0 has been assumed in the design based on the experience in Irish glacial soil.
- λ_f is the interface factor, should be taken to be between 0.7 and 1.0 with the lower value relating to smooth interfaces and the higher value relating to rough interfaces. A value of 1.5 has been assumed in this design based on the experience in Irish glacial soil.

The bond skin friction of the soil nails has been determined by total stress methods for the fine-grained soils as follows:

$$\tau_{bu} = \alpha c_u \text{ (kPa)}$$

Where,

- $\alpha = 0.75$ for the Made Ground and Boulder Clay
- c_u = characteristic undrained shear strength, taken to be 50kPa for the Made Ground, 75kPa for Brown Boulder Clay and 250kPa for Black Boulder Clay (to give $\alpha c_u = 37.5$ kPa for Made Ground, 56.3kPa for Brown Boulder Clay and 188kPa for Black Boulder Clay). This is taken as the maximum bond stress.

The maximum design bond grout-soil stress used in the Black Boulder Clay in the analysis is 100kPa, as presented in Appendix D.

The assumed characteristic bond stress is to be confirmed by sacrificial investigation tests to be carried out at the start of the works. The test details are presented in Section 11.

8.8 Shotcrete Facing Design

A structural shotcrete facing is proposed to support the soil between the nail bearing plates. The design of the facing has been carried out in accordance with CIRIA C367 *Soil nailing – best practice guidance* and FHWA-SA-96-069R *Manual for the design and construction monitoring of soil nail wall*. The facing design calculations are presented in Appendix D.

The facing is proposed to comprise grade C40/50 shotcrete, with minimum thickness of 150mm. A single layer of A393 mesh to be positioned at the centre of the layer with a minimum overlap of 400mm.

The following structural design calculations have been carried out for the shotcrete concrete facing:

1. Tension force in the soil nail
2. Bearing resistance of concrete behind soil nail bearing plates.
3. Shear resistance shotcrete around soil nail bearing plates.
4. Flexural resistance and bending resistance of facing.

These are discussed in the following sections.

8.9 Tension Force in Soil Nail

The load taken by the nail and transferred to the facing has been calculated using the method recommended in FHWA-SA-96-069R *Manual for the design and construction monitoring of soil nail wall*.

Using the method recommend in FHWA, the force on the facing is calculated as 0.5 times the force developed in the nail head due to the active earth pressures, i.e. the force on the facing, $T_F = 0.5K_a\gamma H S_H S_V + \text{water pressure (U)}$. This calculation assumes a vertical slope. The total depth to the lowest nail head is taken as 5.52m. A general variable surcharge load of 10kPa is applied at the slope surface. The water pressures are conservatively taken to be 1m above the lowest nail row. Note that the slope drainage measures and the limited extend of shotcrete facing would be expected to ensure than only minor water pressures can occur.

The facing design allows for a local increase in nail spacing at the pile locations to 1.6m.

The Maximum Design Force (factored) in Nail Head, $T_{0,ULT} = 84.7 \text{ kN}$ has been calculated for each design situation. The horizontal spacing of the nails is 1.5m. This allows for a local increase of 1.6m at piles with a reduced spacing of 1.4m on either side of the affected nails. Therefore the design yield strength of the bars is divided by the horizontal spacing to give the factored tensile strength. The design bar resistance of $134.9/1.5 = 89.9 \text{ kN}$ exceeds $T_{0,ULT}$, so is acceptable.

8.10 Bearing resistance of shotcrete behind soil nail bearing plates

Design Bearing Pressure on Concrete Behind Soil Nail Bearing Plates

Check 300 x 300 x 20mm bearing plate on shotcrete facing:

$$\text{Bearing Stress} = 84.7 \times 10^3 / (300 \times 300) = 0.94 \text{ N/mm}^2$$

Bearing Resistance of Shotcrete Behind Soil Nail Bearing Plates

Concrete Grade = C40/50

$$\begin{aligned} \text{Concrete Bearing Stress capacity} &= 0.3 f_{cd} \text{ (cl. 10.9.4.3, IS EN 1992-1-1)} \\ &= 0.3 \times (f_{cu} / \gamma_m) = 0.3 \times (40 \times 0.8/1.5) = 6.4 \text{ N/mm}^2 \\ 6.4 &> 0.94 \text{ N/mm}^2 \end{aligned}$$

Bearing Resistance > Design Bearing Pressure, therefore, plate dimensions are adequate.

8.11 Punching Shear Resistance of Shotcrete at Bearing Plates

Punching Shear Resistance (R_{fp})

Punching shear resistance, $R_{fp} = 4\xi_s v_c (L_{hb} + h)d$

Where, ξ_s = depth factor = 1.5

L_{hb} = bearing plate width = 300mm

h = depth of shotcrete = 150mm (*plates to be placed against outside face of shotcrete*)

$d = \text{effective depth} = 75\text{mm}$

$v_c = \text{shear resistance of concrete} = (0.27/\gamma_c)(100A_s/bd)^{1/3}f_{cu}^{1/3}$

As presented in Appendix D, $R_{ip} = 182.1 \text{ kN}$, which is $> T_{0, \text{ULT}}$ (84.7 kN) so is therefore acceptable for punching shear.

8.12 Flexural and Moment Resistance

The calculation of the flexural resistance R_{ff} and the bending moment resistance in the horizontal and vertical direction M is presented in Appendix D. The flexural resistance is calculated to be 475.9kN, which is greater than $T_{0, \text{ULT}}$, so is acceptable. The facing moment resistance in the vertical direction, $M_{vm} = 11.9 \text{ kNm/m}$ is also shown to be significantly greater than the cantilever moments at the top and bottom of the facing beyond the soil nails, so are acceptable.

8.13 Drilled in slope drains

To assist with drainage of the slope and to ensure the long-term water level complies with the design it is proposed to install 10m long drainage holes. These are to be located at 3m centre at an elevation of +29.0 to +29.5mOD.

The holes shall be drilled at an angle of 10 degrees upward to the horizontal. A 60mm diameter slotted PVC pipe wrapped in a geotextile separator (Thrace S8NW/PB100, non-woven geotextile, or similar approved) shall be installed in the drainage holes. The drillhole diameter shall be in the range 70 to 90mm.

8.14 Drainage for Shotcrete Facing

The slope drainage in the shotcrete area is to comprise:

- Macdrain W 1051 drainage composite strips placed directly against the 45 degree slope face. Drains to be 300mm wide with spacing of 1.5m. The Macdrain is to extend below the bottom of the shotcrete and is to be run into the drainage trench.

9 CONSTRUCTION DETAILS

9.1 Construction Drawings

The details of the soil nail slope are shown on the Construction Drawings.

9.2 Soil Nail Details

9.2.1 *Soil nail rows*

The estimated total number of soil nails is 444.

The nail lengths and bar type are summarised in Table 9-1 for each design situation.

Table 9-1: Summary of nail lengths, nail rows and type for each Design Situation

Design Situation	Total Slope Height (m)	Height of Shotcrete Face (m)	Horizontal Nail Spacing (m)	Vertical Nail Spacing (m)	No. of nail Rows (m)	Nail Length / Bar Type
DS-1/DS-2	5.9	5.9	1.5	1.25	5	Row 1 = 7m, R32-280 Row 2 = 7m, R32-280 Row 3 = 8m, R32-280 Row 4 = 8m, R32-280 Row 5 = 8m, R32-280
DS-3/DS-4	6.7	6.7	1.5	1.25	5	Row 1 = 7m, R32-280 Row 2 = 7m, R32-280 Row 3 = 8m, R32-280 Row 4 = 8m, R32-280 Row 5 = 8m, R32-280

9.2.2 *Horizontal and vertical Spacing*

The maximum horizontal spacing (s_h) is to be 1.5m (**locally 1.6m where soil nails are installed near piles**) along the full length of the slope and the vertical spacing (s_v) is 1.25m.

9.2.3 *Spacers*

Spacers are to be used in accordance with IS EN 14490:2010 to centralise the bar in the drillhole. The maximum spacing of spacers is to be 3m.

9.2.4 *Angle of inclination*

A nail inclination of 20 degrees to the horizontal has been applied for the soil nails.

9.2.5 *Length and location of soil nails*

The minimum soil nail length is presented in Table 9-1. The soil nails will be grouted over their full length. The depth/level to the head of the soil nail are given in the construction drawings.

9.2.6 *Soil nail type, hole diameter and installation method*

The soil nails shall be R32-280 Self Drilling Hollow Bars with galvanised bar, nuts and couplers. The bar shall have the properties as shown on Table 9-2.

Table 9-2: Properties of soil nail bar

Bar Type	Nominal Thread Diameter (mm)	Ultimate Strength (kN)	Yield Strength (kN)	Bar Coatings
R32-280	32	280	220	Galvanised along full length of bar

The soil nails are hollow stem bars which are installed using low strength grout as the flushing medium. The nails shall be grouted into a hole of minimum diameter of 0.1m.

The bars have a sacrificial drill head and the permanent grout is pumped through the hollow bar when the nail has been installed to the required depth.

The Contractor shall ensure that the grouting operations are such that the soil nail is grouted over its full length and that the grout surrounding the nail in the drillhole is clean and free from arisings.

9.2.7 *Setting out of nails*

Outside of zones with bearing piles:

- Nail positional tolerance is +/-100mm and deviation tolerance is 5 degrees.

Soil nails adjacent to CFAbearing piles

- Nail positional tolerance is +/-50mm and deviation tolerance is 2.5 degrees.

9.2.8 *Nuts and bearing plates*

The permanent bearing plates are to comprise:

- **Shotcrete facing: 300x300x20 mm galvanised steel plate.** Bearing plates to be located at the external face of the permanent shotcrete facing. The bearing plate and nut is to allow for articulation in the range of up to 20 degrees. The nail head to be covered with min 50mm of shotcrete.

9.2.9 *Nail grout mix*

Grout shall comprise a Portland Cement grout with a minimum 28 day compressive strength of 40 N/mm².

The grout mix shall be mixed with 40 litres of water for every 100kg of cement.

Testing shall comprise 1 No. set per rig per day with a set comprising 4 No. cubes. Each set of 4 no. cubes shall comprise 1 no. test at 7 days and 2 no. tests at 28 days, with 1 no. held in reserve.

9.3 Facing details

The facing is to comprise:

- The permanent facing is proposed to comprise grade C40/50 shotcrete, with minimum thickness of 150mm. A single layer of A393 mesh to be positioned at the centre of the layer with a minimum overlap of 400mm.

9.4 Surface drains

The site surface drainage should ensure that any surface water is collect and directed away from the slope.

A surface drain is required at the toe of the slope. This would typically comprise a 0.5m deep gravel filled drain with perforated collector pipe. The shotcrete facing would extend below the top of the drain. The Macdrain composite strips would also extend into the drain.

9.5 Drilled-in Slope Drains

1 No. rows of 10m long drainage holes shall be drilled in the slope face at maximum 3m centres. The drains shall be located at +29.0 to +29.5mOD

The holes shall be drilled at an angle of 10 degrees upward to the horizontal. A 60mm diameter slotted PVC pipe wrapped in a geotextile separator (Thrace S8NW/PB100, non-woven geotextile, or similar approved) shall be installed in the drainage holes. The drillhole diameter shall be in the range 70 to 90mm.

10 CONSTRUCTION SEQUENCE

Construction sequence given below shall be followed during the construction works. This sequence reflects the current excavation that has been carried out of the full height of the of the slope face.

1. A crest drain or bund is required to prevent surface water flowing down the slope face both during construction and for the permanent condition. The design of the crest drain is by others.
2. The soil nailed slope is to be cut to its final profile progressing from slope crest to toe in typically in maximum lifts of 1.25m. This will allow 1 No. Row of nails to be installed at each excavation stage. The cut slope should not be exposed more than 24hrs ahead of the soil nailing.
3. Install and grout the upper rows of soil nails, with the upper row beginning at 0.3 to 0.5m below the crest of the slope.
4. Place Macdrain, A393 mesh, bearing plates and nuts. Plates to be located at outside face of shotcrete. Spray shotcrete facing min 150mm thick*.
5. Excavation of the next (lower) bench level to be carried out a minimum 48 hrs after shotcrete installation in the rows above.
6. Soil nailing, facing works and excavation to proceed in maximum 1.5m lifts/benches to base of slope.
7. Construct 10m long drilled in drains and weep holes to the stater layer as the excavation of the slope progresses.
8. Carry out working soil nail tests a minimum of 7 days after nail installation.
9. Installed permanent 150mm thick shotcrete on the 45 degree slope face
10. Install drain at toe of slope.

* Note that as an alternative sequence a thin (75mm thick) temporary shotcrete facing with A142 mesh and temporary bearing plates can be installed in sequence to the base of the slope. Once the soil nailing is completed the permanent 150mm thick shotcrete, A393 mesh, bearing plates and nuts can be installed on the final cut face. This can give a better finish and avoids this work being carried out from height.

11 MONITORING, DESIGN VERIFICATION AND QUALITY ASSURANCE

11.1 Testing requirements

The testing that is required to be carried out on the soil nails is detailed in Section 11.2 and 11.3.

11.2 Sacrificial Nail Tests

11.2.1 General

It is proposed to carry out 2 No. sacrificial load tests

11.2.2 Sacrificial testing

2 No. Sacrificial Tests are to be carried out on soil nails installed in the glacial till to demonstrate the ultimate bond resistance. The nails are to be installed with head level at approx. 10mOD and 13mOD, respectively. The tests shall comprise:

- 2 No. sacrificial test with total length = 5m, upper 1m to be debonded; $P_p = 230$ kN. R32-320 bar. Nail inclination to be 20deg. Drill bit dia. = 100mm.

The tests shall be carried out a minimum of 7 days after nail installation.

The load test schedule for the Sacrificial test is given on Table 11-1.

Table 11-1: Loading Procedure Sacrificial Test

Load (kN)	Minimum Hold Time	Reading @ (min)
First Cycle		
0	-	0
5	10 min	0, 1, 2, 5, 10
35	10 min	0, 1, 2, 5, 10
65	10 min	0, 1, 2, 5, 10
110	20 min	0, 1, 2, 5, 10, 15, 20
150	20 min	0, 1, 2, 5, 10, 15, 20
5	10 min	0, 1, 2, 5, 10
Second Cycle		
150	10 min	0, 1, 2, 5, 10
170	10 min	0, 1, 2, 5, 10
190	20 min	0, 1, 2, 5, 10, 15, 20
210	20 min	0, 1, 2, 5, 10, 15, 20
0	10 min	0, 1, 2, 5, 10

Note:

1. If creep is > 2 mm from 2 to 20 min, at the maximum load for Load Cycles 1 & 2, then the load is to be held for 60 mins

11.3 Production Nail Tests

Production tests will be carried out on 2% of working nails at locations to be agreed. Tests to be carried out a minimum of 7 days after installation. The maximum load (P_p) is 150% of

the Design Nail Load (P_{des}) based on the partial factor of 1.5 on the design bond stress. However in most instances the test load is limited by the bar tensile resistance

Tests are to be carried out in one cycle at 5 or 10kN (initial load), 50% P_p , 75% P_p , 100% P_p and then unload to 0 kN. The load test schedule for the Production tests is given on Table 11-2.

Table 11-2: Loading Procedure and Acceptance Criteria: Production tests

Soil Nail Row	Nail Length (m)	Proof test Load, P_p (kN)	Initial Load	Load Increments for Production Tests (kN)		
				1 st Increment (50% P_p)	2 nd Increment (75% P_p)	3 rd Increment (100% P_p)
1	7	82	5	41	62	82
2	7	100	5	50	75	100
3	8	125	5	62.5	94	125
4	8	140	5	70	105	140
4	8	170	5	85	127	170

Readings shall be taken at the following intervals:

- 5 kN 0, 1, 2, 5, 10 minutes
- 50% P_p 0, 1, 2, 5, 10 15, 20 minutes
- 75% P_p 0, 1, 2, 5, 10 15, 20 minutes
- 100% P_p 0, 1, 2, 5, 10, 15, 20 minutes
- 0 kN 0, 1, 2, 5, 10 minutes

A Soil Nail Production test shall be considered acceptable if < 2mm of creep occurs within one log cycle of time (i.e., from 1 to 10, or 2 to 20 mins).

11.4 Monitoring

Daily inspections of the slope crest and face shall be carried out during the works to observe for any signs of movements, such as settlement or tension cracks at the slope crest and for shallow slips in the slope face.

12 MAINTENANCE REQUIREMENTS

There are no specific maintenance requirements for this structure. All buried and exposed steel is designed for 100 year design life with grout/concrete cover and without any further maintenance required.

General requirements include ensuring surface drains are not blocked and these inspections as can be conducted as part of standard maintenance requirements.

13 DESIGNER'S RISK ASSESSMENT

A Designers Risk Assessment for the temporary soil nail slope is included in Appendix G.

Some of the particular risks are described below. These risk items need to be assessed by the Contractor during the construction of the works.

13.1.1 *Strike of CFA bearing pile*

A sacrificial “dummy” piles is to be installed on the railway side of each bearing pile that is within the plan extent of the soil nailing. This ensures that the piles supporting the building can't be damaged by the soil nailing works. The as-built location of the CFA piles is to be recorded and clearly marked on the slope face. The soil nails are to be set out to allow for minimum 0.5m offset from the CFA piles.

13.1.2 *Collapse of existing retaining wall at crest of slope on to the railway*

The existing retaining wall is to be stabilised by a row of soil nails to ensure that a future collapse on to the railway cannot occur. The shotcrete facing may also extend over the wall depending on its structural condition.

13.1.3 *Fall from height*

The construction of the soil nails slope is required to be carried out on a slope face up to 7m high with slope angle of 60 degrees. Therefore, there is a risk of falling to personnel working at the crest of the slope and on the slope face.

13.1.4 *Collapse of slope during excavation*

The slopes are to be inspected daily for signs of instability, such as tension cracks in the ground at the slope face or other signs of movement. An exclusion zone shall be maintained at the base and the behind the crest of the slope. Only personnel carrying out the slope stabilisation works shall be permitted in this zone. Materials shall not be stockpiled at the crest of the slope.

13.1.5 *Settlement of Ground at Top of Slope - Soil Nail Grouting*

During installation it shall be ensured that the grout has fully filled the drillhole and that no voids are present by the grout to overflow onto the slope face. Any voids remaining within the hole can cause settlement behind the slopes due to collapse of voids, as well as reduce the bond skin friction available to stabilise the slope.

13.1.6 *Collapse of drillholes - Soil Nail Grouting*

During installation it shall be ensured that the grout has fully filled the drillhole and that no voids are present by the ensuring that grout overflows onto the slope face. Any voids remaining within the hole can cause settlement behind the slopes due to collapse of voids, as well as reduce the bond skin friction available to stabilise the slope.

13.1.7 *Nails do not achieve min length due to Cobble/Boulders obstructions*

Cobbles and boulders may be encountered in the glacial deposits. This will have to be accounted for in the type of drilling equipment used to install the soil nails into the slope.

13.1.8 *Underground/over ground services*

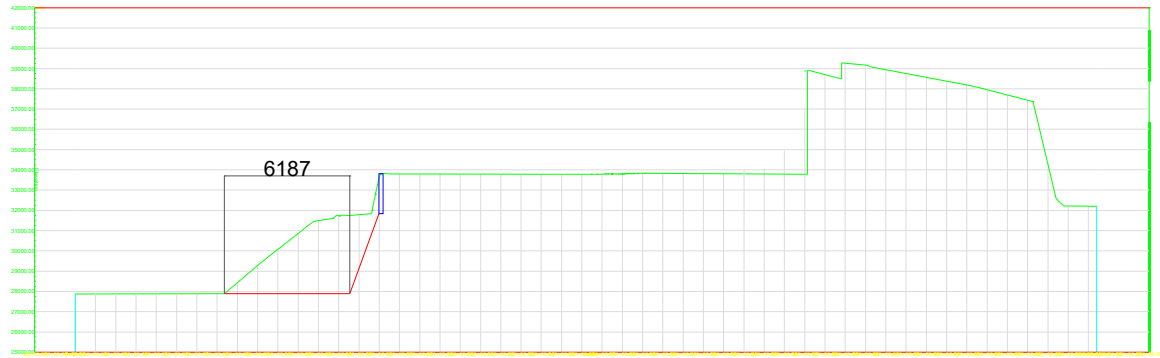
Underground/overground services are not expected to be encountered, however, appropriate care is to be taken during the works.

14 SOIL NAIL CONSTRUCTION STANDARDS

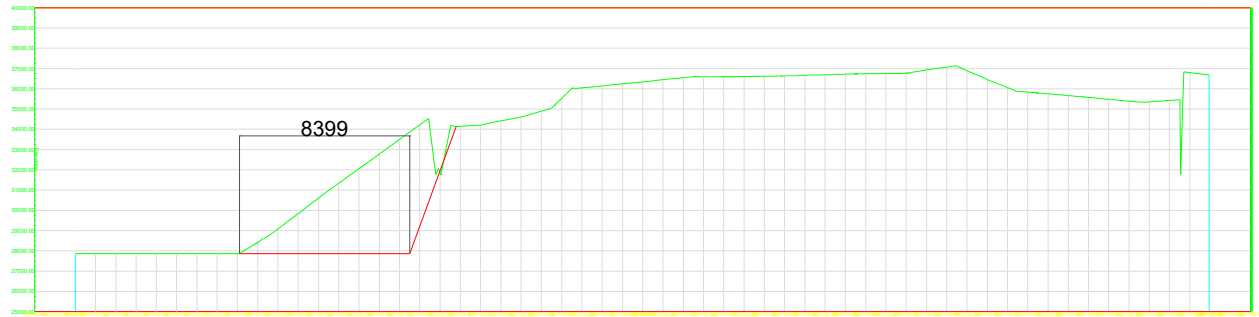
The soil nails are to be constructed in accordance with IS EN 14490:2010 execution of special geotechnical works – Soil nailing.

APPENDIX A
SECTION DRAWING

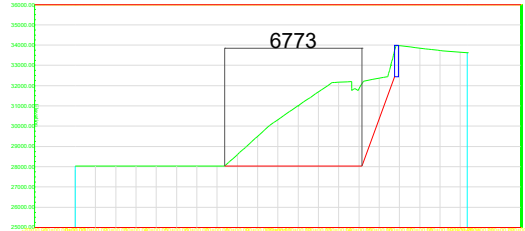
Section A



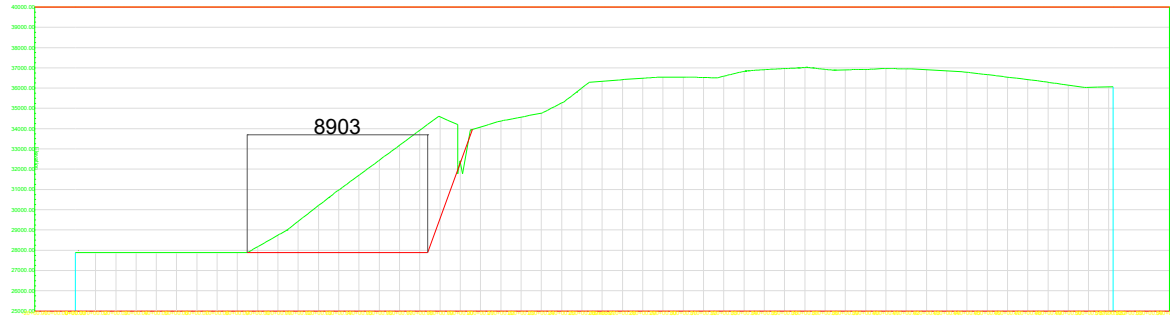
Section F



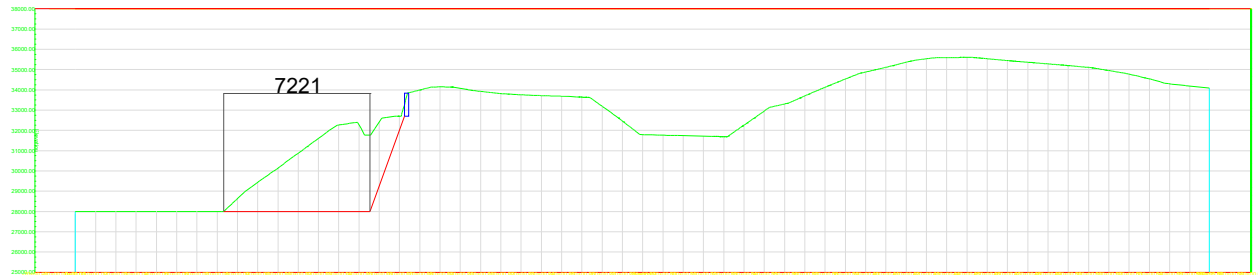
Section B



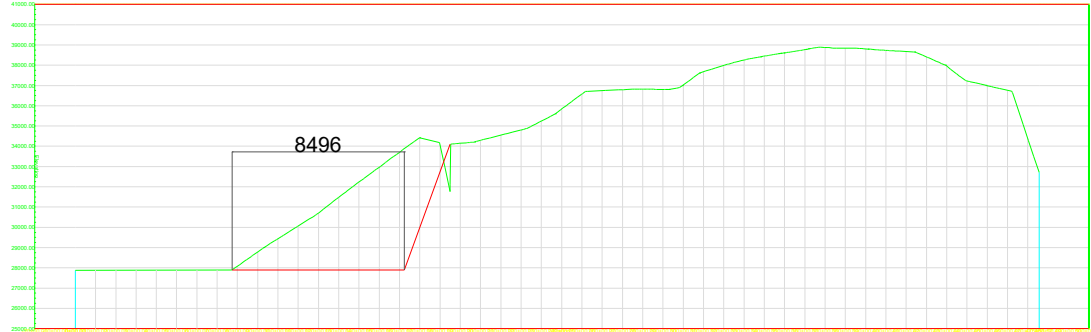
Section G



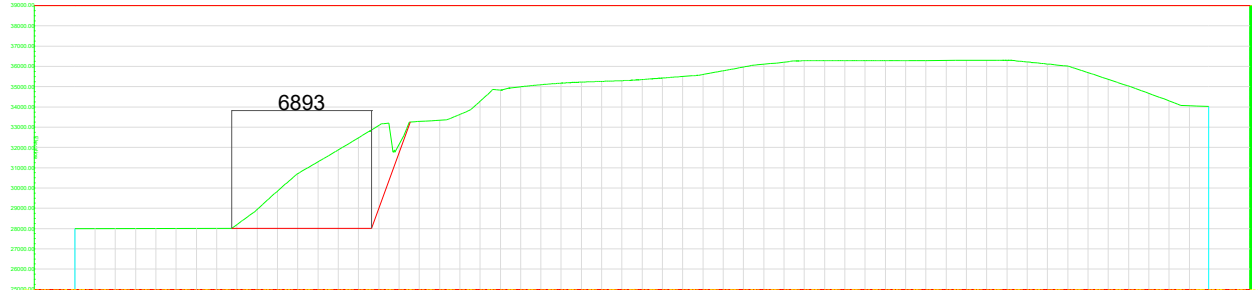
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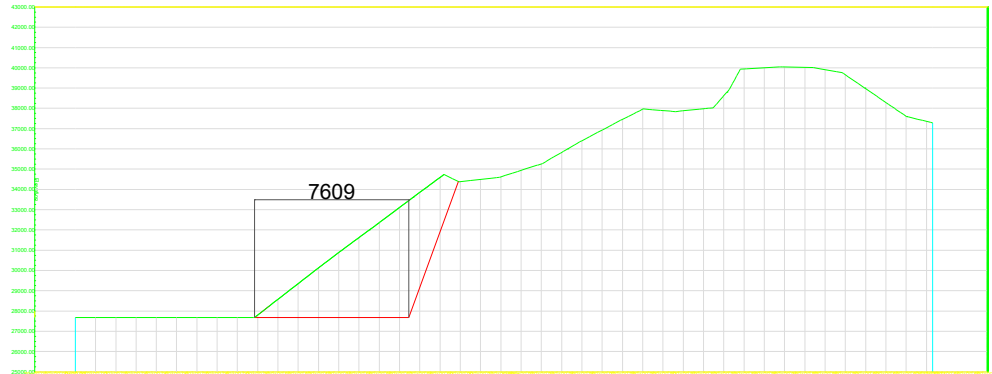
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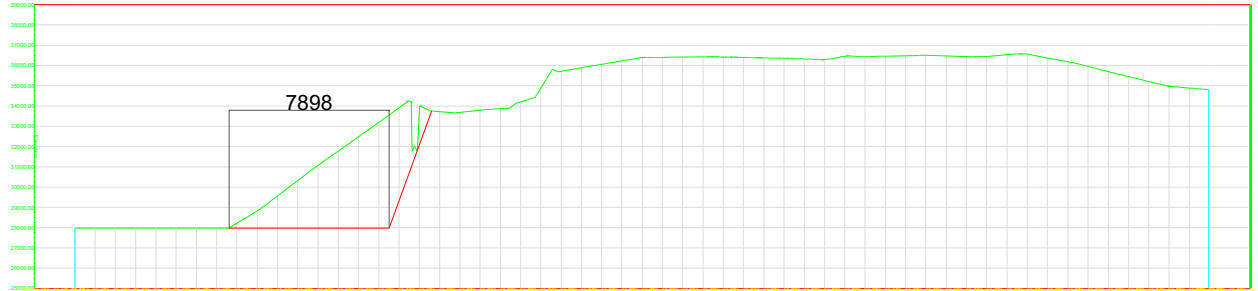
Section D



Section I (S2)



Section E



APPENDIX B

DESK STUDY

04. 1990.

13 99 .

29/23 SE

SERIAL No. 1473.

Report No. 940

Box No. 45

Investigation ID. 62629 - 62632
(4)

**SITE INVESTIGATION AT
CABRA ROAD , DUBLIN**

**JOHN DOYLE & ASSOC.
CONSULTING ENGINEERS.**

CONTENTS

- I. Introduction**
- II. Fieldwork**
- III. Testing**
- IV. Discussion**

APPENDICES

- I. Boring Records**
- II. Test Results**
- III. Site Plan**

FOREWORD

Notes on Site Investigation Procedure

The following notes should be read in conjunction with the Report. Any modifications to the procedures outlined below are indicated in the main text.

GENERAL :

The recommendations made and opinions expressed in the Report are based on the Boring Records, an examination of samples and the results of the site and laboratory tests. No responsibility can be held for conditions which have not been revealed by the boreholes, for example, between borehole positions. Whilst the Report may express an opinion on a possible configuration of strata both between borehole positions and below the maximum depth of the investigation, this is for guidance only and no liability can be accepted for its accuracy.

BORING TECHNIQUE :

Unless otherwise stated, the 'Shell and Auger' technique of soft ground boring has been employed. Whilst this technique allows the maximum data to be obtained on strata conditions, a degree of mixing of some layered soils, (e.g. thin layers of coarse and fine granular material) is inevitable. Specific attention is drawn to this factor where evidence of such a condition is available.

GROUND WATER :

The ground water conditions entered on the Boring Records are those ascertained at the time of the investigation. The normal rate of boring does not usually permit the recording of an equilibrium water level for any one water strike. Moreover, ground water levels are subject to variations caused by seasonal effects or changes in local drainage conditions. The table of each Boring Record shows the ground water level at the quoted borehole and casing depths, usually at the start of the day's work. The word 'None' indicates that ground water was sealed off by the borehole casing.

ROUTINE SAMPLING :

Undisturbed samples of predominantly cohesive soils are obtained in a 102 mm diameter open-drive sampler, complying with the requirements of the British Standard Code of Practice B.S. 5930. Large disturbed samples of granular soils, or of soils in which undisturbed sampling is not possible or appropriate, are taken from the boring tools and sealed into polythene bags. Small disturbed samples are taken at frequent intervals of depth and sealed into 0.5 kg. glass jars or polythene bags for subsequent visual classification. Where encountered in sufficient quantity, samples of ground water are taken.

Unless otherwise stated in the main text, disturbed soil samples may not be at their natural water content.

* * * * *

**REPORT ON A SITE INVESTIGATION
AT CABRA ROAD , DUBLIN
FOR JOHN DOYLE & ASSOCIATES.**

REPORT NO.1399

OCTOBER 1990

I. INTRODUCTION

It is proposed to develop a site at the junction of Carnlough Road and Cabra Road in Dublin, the site adjoins the railway tracks and is in C.I.E property.

An investigation of sub soil conditions has been ordered and four boreholes have been sunk using conventional 200mm cable tool methods , with associated in - situ testing and sampling .

This report determines the findings of the investigation and relates these findings to the design of safe and economic foundations for the proposed structure .

I.

II. FIELDWORK

The site location and borehole positions thereon are shown on the location map found in Appendix III of this report .

Descriptions and depths of strata encountered are given on the individual boring records contained in Appendix I . These records give details of samples taken and in - situ tests carried out as well as ground water conditions pertaining at the time of boring . Any obstruction to normal boring, necessitating the use of a chisel are also detailed .

The records show a covering of made ground varying from 3.10 to 6.70 metres in thickness at Boreholes 1 , 3 and 4 and 0.40 metres in thickness at Borehole 2 .

The fill consists in general of stony clays with brick , rubble and concrete , however , organic traces were noted particularly from 2.40 to 6.70 in Borehole 4

In Boreholes 3 and 4 a thin stratum of mottled grey brown silty clay underlies the fill . Below this silty clay in Boreholes 3 and 4 and directly below the fill in Boreholes 1 and 2 is a stratum of brown silty sandy stony clay containing cobbles and boulders (Brown glacial till or Boulder clay).

This material is rapidly succeeded by hard grey black stony clay (Black glacial till or boulder clay) and boreholes were terminated in this stratum .

No free ground water was noted during the course of this boring operation .

III. TESTING

Disturbed samples were taken at regular intervals in each borehole and samples sent to the laboratory for examination and testing . In - situ Standard penetration tests were also carried out at approximately 1.50 metres intervals in each borehole.

III.

The natural moisture contents of both clayey fill and underlying natural boulder clays has been determined . High natural moisture contents (23 - 26%) reflect the softer nature of the fill while values of from 10 to 13% were obtained in the underlying clays .

Liquid and plastic limits have also been determined and results plotted on the standard Casagrande chart , a copy of which is included in Appendix II.

Results indicate that the clay fill and the boulder clays are of similar origin , being of very low to low plasticity .

Chemical analyses of two samples indicates a low sulphate concentration and no special precaution needbe taken to protect foundation concrete .

Standard in - situ penetration tests were carried out to give an indication of in - situ soil strength . The test involves driving a conical probe a measured distance (300mm) into the soil , recording the number of blows to achieve the required penetration (N value)

Tests in the fill at 1.50m gave N values of from 23 to 46 , indicating a stiff upper crust of material . Results of from 9 to 14 below this upper crust imply a softening of the fill material .

Tests in the underlying boulder clays give N values of 22 to 25 in the brown clays and values in excess of 50 in the grey black boulder clays .

IV. DISCUSSION

Fill material , up to 6.70 metres in thickness, is noted on the site . The fill is of greatest thickness at Boreholes 1 , 3 and 4 along the Carnlough Road boundary and thinnest at Borehole 2.

Because of the variability of the fill , with some organic material noted , and the lack of information on the placement, the use of foundations placed on the fill can not be recommended .

It will be necessary, therefore, to transfer foundation loadings to the underlying competent sub soils or to improve the bearing capacity of the fill by a recognised geotechnical process.

The brown boulder clay should be capable of supporting an increase in load for normal footings of 200 KN/m² while an increase of 300 KN/m² can be taken in the black boulder clay .

The depth of excavation required through the fill may determine the economic viability of strip or pad foundations and in areas of deepest fill this will be obviously impractical.

In these deep fill areas the use of piling methods can be considered . Various piling methods are available . A precast driven concrete pile , founded in the grey black boulder clay should provide load bearing capacities of up to 100 tonnes .

The treatment of the fill by vibroflotation or its consolidation by dynamic compaction could also be considered . Both methods will improve the overall strength of the fill while reducing settlement characteristics .

Following treatment , conventional foundations , close to ground level can be considered .

Specialist contractors , with geotechnical experience should be consulted to determine the possible suitability of each process .

VII.

APPENDIX I - Boring Records

APPENDIX II - Test Results

Report No. 1399		CLASSIFICATION TEST RESULTS						IGSL
Contract								
CABRA								
Borehole No.	Sample No.	Depth (Metres)	Sample Description	Percentage Passing 425 μ m Sieve	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Water Content %
1	1606	3.00	Clay Fill		28	18	10	23.8
	1607	6.00	Brown silty stony CLAY		25	13	12	13.1
2	1602	1.50	Brown silty stony CLAY		24	11	13	12.2
	1604	4.00	Black silty stony CLAY		22	9	13	9.5
3	1599	3.00	CLAYEY FILL		35	22	13	31.7
	1601	5.00	Black silty stony CLAY		19	10	9	10.2
4	1610	3.00	FILL GLASS PLASTIC CLAY					36.2
	1612	6.80	Brown stony CLAY		27	12	15	12.8

Report No.

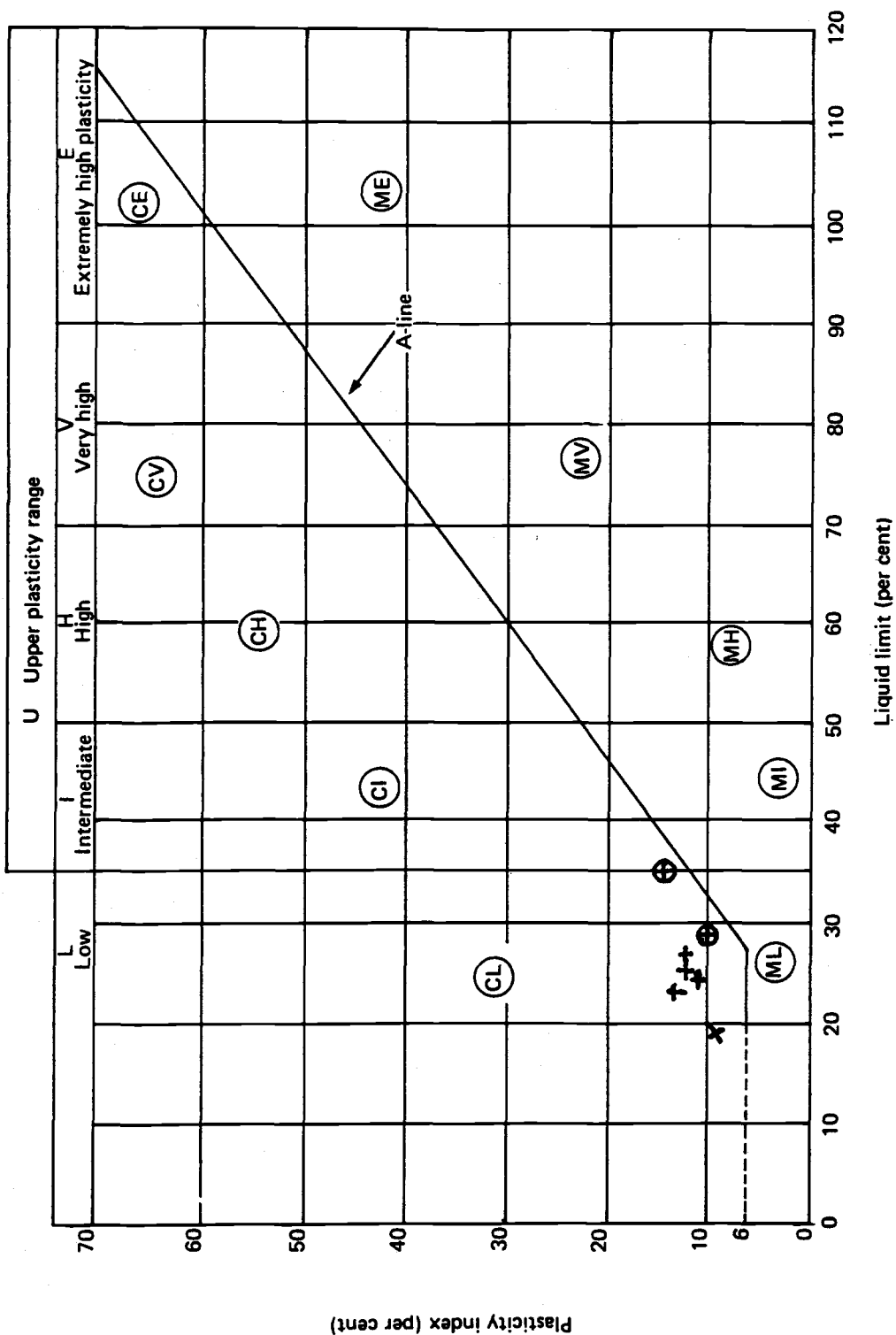
PLASTICITY CHART

IGSL

Contract:

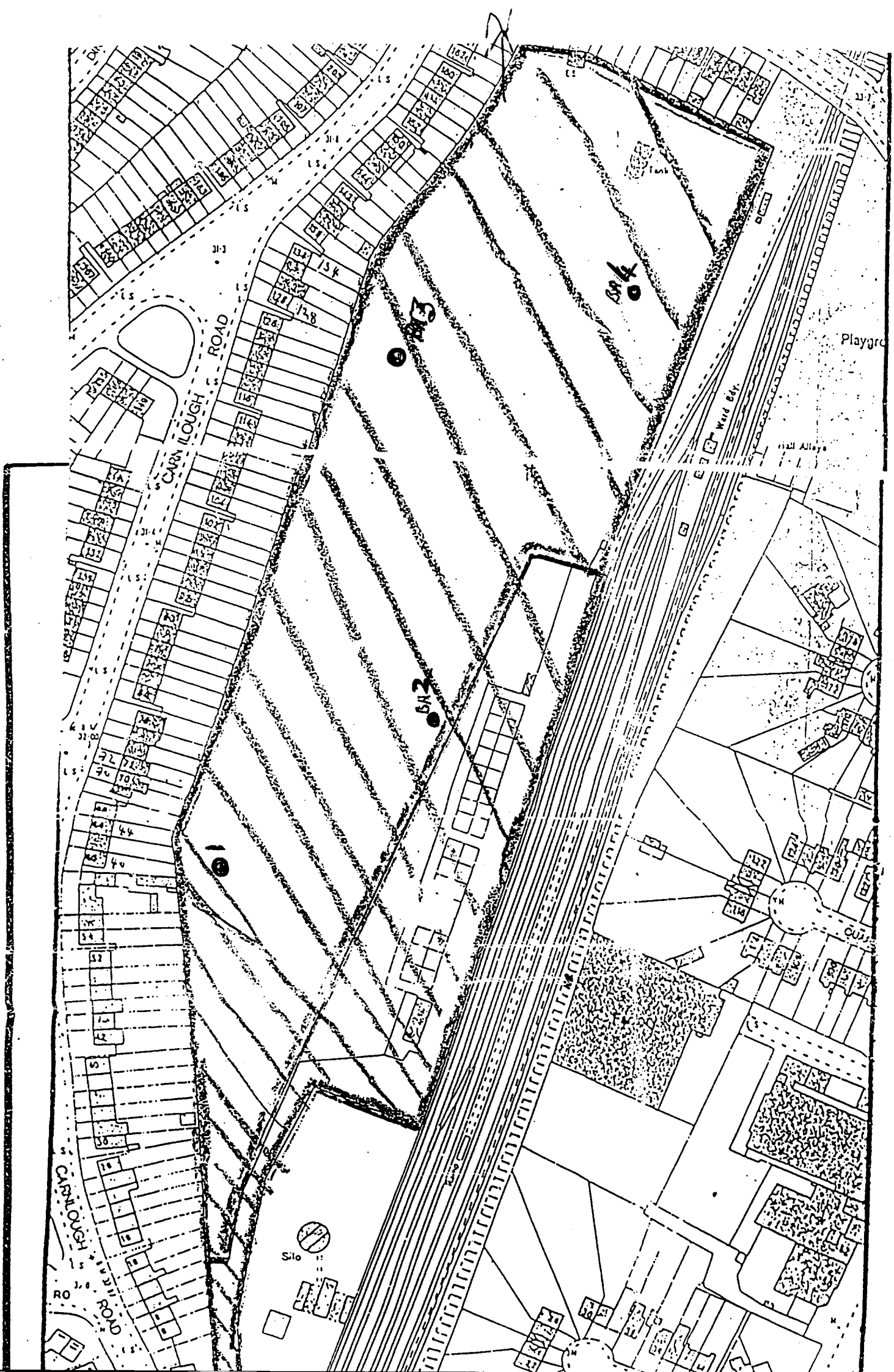
CABRA

⊕ FILL MATERIAL
+ BOULDER CLAY



Report No. 1399		CHEMICAL ANALYSIS				IGSL	
Contract CABRA							
Borehole No.	Sample No.	Depth (metres)	Sample Type	Test Code	Sulphur Trioxide		pH Value
					Parts SO_3 per 100,000 Water	Per Cent SO_3 Soil	
1	1606	3.00	FILL	S		NEGLIGABLE	7.6
4	1610	3.00	FILL	S		0.08	7.5
Test Code W – Water S – Soil A – Aqueous soil extract							

APPENDIX III - Site Plan



Report No. 1399		BORING RECORD				IGSL		
Contract CABRA CIE DEPOT					Borehole No. 3 Sheet			
Location CABRA ROAD				Type and Diameter CABLE TOOL 200mm				
Client JOHN DOYLE CONSULTING ENGINEER				Ground Level Date 23.10.89 24.10.89				
Description		Reduced Level	Legend	Depth	Samples Ref. No. Type Depth		Field Records And Tests	
FILL: GREY AND BROWN SILTY STONY CLAYS, WITH BRICK GLASS AND ORGANIC TRACES					1598	D	1.50	1.50 N 23
Stiff mottled grey brown silty stony CLAY				3.10	1599	D	3.00	3.00 N 16
Stiff to hard grey black very silty very stony CLAY with cobbles and boulders. (Boulder Clay)				4.80	1600	D	4.00	4.50 N 25
				6.50	1601	D	5.50	6.00 N 68
Water Level Observations during Boring				Remarks				
Date	Hole Depth	Casing Depth	Depth to Water	Remarks				
			DRY	Chiselling in fill 1½ hrs " in boulders 3 hrs				
				Sample/Test key C-Cone Penetration Test U-Tube Sample N-Blows/0.3 metres D-Disturbed Sample R-Refusal W-Water Sample V-Vane S-Standard Penetration Test				

Report No.		1399		BORING RECORD				IGSL		
Contract				CABRA CIE DEPOT				Borehole No. 4		
Location				CABRA ROAD				Type and Diameter		
Client				JOHN DOYLE CONSULTING ENGINEER				CABLE TOOL 200mm		
				Ground Level				Date		
								27.10.89 28.10.89		
Description		Reduced Level	Legend	Depth	Samples			Field Records		
					Ref. No.	Type	Depth	And Tests		
FILL: Brown stony CLAY with traces of brick and concrete					1609	D	1.50	1.50	N 23	
FILL: Dark silty CLAY with brick, plastic, glass etc.				2.40	1610	D	3.00	3.00	N 12	
								4.50	N 9	
Stiff mottled grey brown silty stony CLAY				6.70	1611	D	6.00	6.00	N 14	
					1612	D	6.80		7.50	N 22
Stiff to hard grey black very silty very stony CLAY with cobbles and boulders. (Black boulder Clay)				9.00	1613	D	9.50		9.00	N 51
					1614	D	11.50		10.50	N 56
				12.00				12.00	N 79	
Water Level Observations during Boring					Remarks					
Date	Hole Depth	Casing Depth	Depth to Water	Remarks	Chiselling concrete 1 h Chiselling boulders at 9.70, 10.80 and 11.30 4 hours					
			DRY	No free water						
Sample/Test key					C-Cone Penetration Test					
U-Tube Sample					N-Blows/0.3 metres					
D-Disturbed Sample					R-Refusal					
W-Water Sample					V-Vane					
S-Standard Penetration Test										

62629

Report No.	1399	BORING RECORD				IGSL	
Contract					Borehole No.		
CABRA CIE DEPOT					1		
Location					Type and Diameter		
CABRA ROAD					CABLE TOOL 200mm		
Client					Ground Level		
JOHN DOYLE CONSULTING ENGINEER					Date		
					26.10.89 27.10.89		

Description	Reduced Level	Legend	Depth	Samples			Field Records And Tests
				Ref. No.	Type	Depth	
FILL CLAY WITH STONES , CONCRETE, BRICK, GLASS ETC.		XXXXXX		1605	D	1.50	1.50 N 46
			1606	D	3.00	3.00 N 11	
						4.50 N 10	
Stiff brown silty stony CLAY (Boulder Clay)		XXXXXX	5.20	1607	D	6.00	6.00 N 26
			6.60				
Hard grey black very silty very stony CLAY with boulders		XXXXXX		1608	D	8.00	7.50 N 70
						9.00 N 83	
			10.00				

Water Level Observations during Boring					Remarks
Date	Hole Depth	Casing Depth	Depth to Water	Remarks	
			DRY	NO FREE WATER	CHISELLING IN FILL 1½ hr CHISELLING THROUGH BOULDER CLAY 3 hrs Sample/Test key U-Tube Sample D-Disturbed Sample W-Water Sample S-Standard Penetration Test C-Cone Penetration Test N-Blows/0.3 metres R-Refusal V-Vane

62630

Report No. 1399	BORING RECORD				IGSL			
Contract CABRA CIE DEPOT					Borehole No. 2			
Location CABRA ROAD					Type and Diameter CABLE TOOL 200mm			
Client JOHN DOYLE CONSULTING ENGINEER					Ground Level			
					Date 25.10.89 26.10.89			

Description	Reduced Level	Legend	Depth	Samples			Field Records And Tests	
				Ref. No.	Type	Depth		
FILL: BROWN STONY CLAY		[Hatched Box]	0.40					
Stiff brown silty stony CLAY		[Stippled Box]	1.50	1602	D	1.50	1.50	N 28
Hard grey black silty very stony CLAY with cobbles and boulders (Black boulder clay)		[Dotted Box]	2.10					
		[Dotted Box]	3.00	1603	D	3.00	3.00	N 54
		[Dotted Box]	4.50	1604	D	4.50	4.50	N 27/75mm and refusal
BOREHOLE COMPLETE AT 6.00		[Dotted Box]	6.00				6.00	N Refusal

Water Level Observations during Boring					Remarks
Date	Hole Depth	Casing Depth	Depth to Water	Remarks	
			DRY	NO FREE WATER	Chiselling boulders in the boulder clay 3 hours Sample/Test key U-Tube Sample C-Cone Penetration Test D-Disturbed Sample N-Blows/0.3 metres W-Water Sample R-Refusal S-Standard Penetration Test V-Vane

REPORT ON SITE INVESTIGATION
AT

BATCHELORS, CABRA, CO. DUBLIN

Sarnon Rd.

CONTENTS:

REPORT
BOREHOLE RECORD
SITE MAP.

Box 2

REPORT ON SITE INVESTIGATION AT BATCHELORS, CABRA, CO. DUBLIN

INTRODUCTION:

At the request of Messrs. Kaye, Parry and Partners, Architects a site investigation was carried out at Batchelors factory in Cabra during August 1983.

The site investigated was at the corner of the existing factory near the main entrance. It is intended to construct a sterilizer building on the site. The sterilizer unit is a high columnar structure which will be founded on a heavy reinforced concrete base.

INVESTIGATION:

A single borehole was put down near the centre of the proposed building. This hole was 200mm in diameter and was bored with a Pilcon Wayfarer 1500 drilling rig using standard cable-tool methods.

Disturbed samples were taken from each stratum encountered and examined in the laboratory. Standard Penetration tests were carried out in the borehole and the results of these as well as sample depths and numbers and details of strata encountered are shown on the accompanying borehole record.

SUB-SURFACE CONDITIONS:

Beneath the concrete slab a firm brown gravelly clay was found to 2.15m below ground level. This was followed by a hard black gravelly silty clay with cobbles and boulders (boulder clay). There were fairly frequent cobbles and boulders in this material requiring the use of the chisel to penetrate them.

The borehole ended in black boulder clay

PAGE 2

at 9.50m. Water was not encountered in the borehole.

IN-SITU TESTS:

Standard Penetration tests were carried out in the borehole and the resulting N values are shown on the borehole record.

An N value of 15 was recorded in the brown gravelly clay. This figure indicates a safe bearing capacity in this material of 140 kN/m^2 .

The N values in the black boulder clay are extremely high and give an allowable bearing capacity in excess of 350 kN/m^2 .

RESULTS AND RECOMMENDATIONS:

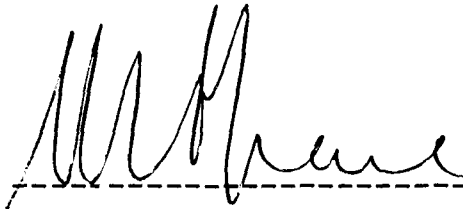
The foundations for the building and the sterilizer may be placed on either the brown clay at 1.0 - 1.50m using a bearing pressure of 140 kN/m^2 or on the black clay at about 2.20m using the higher bearing pressure of 350 kN/m^2 .

The configuration of the foundations will decide whether it is more economical to found near the existing ground level with lower bearing capacity or at the deeper level with the high bearing capacity. The stability of the sterilizer which is a high structure may also be a deciding factor.

It was not possible to carry out settlement tests due to the very stony nature of the soil. However, Dublin boulder clay is a highly overconsolidated material and settlements should be minimal under the

PAGE 3

recommended loading.

A handwritten signature in black ink, appearing to read 'M.F. Grace', is written over a horizontal dashed line.

M.F.GRACE BE MSc, DIC, CEng, MIEI.

BOREHOLE RECORD



Unsure of Location
Box:2
Report# 19

Inv ID 51569

Report No. 2604	TRIAL PIT RECORD					I.G.S.L.		
CONTRACT: Proposed factory development for Shamrock foods				Trial Pit No.: 5				
				Sheet : 1 of 1				
CLIENT: BeMRA, Consulting Engineers				Excavation Method: JCB				
				Date Started: 3.8.94				
LOCATION: West County Business Park County Dublin				Date Completed: 3.8.94				
				Ground Level (mOD):				
	Geotechnical Description	Legend	Depth (m)	Elevation (mOD)	samples			Field Tests
					Ref. No.	Type	Depth (m)	
-	Top Soil		0.30					
-	Firm brown gravelly CLAY with occ. cobble							
-1								
-2								
-3	Brown fine to coarse clayey sandy GRAVEL with cobbles and boulders.		2.50		3735	D	1.50	
-4								
-5								
-6								
			3.50		3736	D	3.00	
Groundwater Conditions: Seepage noted at 3.50m.								
Remarks: Trial pit unstable at 3.50m.								

Report No.	BORING RECORD								
Contract Batchelors Ltd						Borehole No. 1 Sheet 1			
Location Cabra					Type and Diameter Cable Tool 200mm				
Client Kaye Parry and Partners					Ground Level				
					Date 22-8-83				

Description	Reduced Level	Legend	Depth	Samples			Field Records And Tests
				Ref. No.	Type	Depth	
Concrete (1 hr. chiselling)		XXXX	0.15				
Firm brown gravelly silty clay				7256	D	1.50	N = 15
Stiff to hard black gravelly silty clay with cobbles and boulders			2.15	7257	D	3.00	N = 38
(chiselling 1 hr. at 6.00m)							
				7258	D	5.00	N = 65
				7259	D	6.00	N = 89
continued			8.00				

Water Level Observations during Boring					Remarks
Date	Hole Depth	Casing Depth	Depth to Water	Remarks	
				Groundwater not encountered	Total chiselling 3 hrs.
					Sample/Test key U-Tube Sample D-Disturbed Sample W-Water Sample S-Standard Penetration Test
					C-Cone Penetration Test N-Blows/0.3 metres R-Refusal V-Vane

[illegible]

QUARRY ROAD
CABRA

MURRAY PARTNERSHIP
DEVELOPERS

CONTENTS

I	INTRODUCTION
II	FIELDWORK
III	TESTING
IV	DISCUSSION

APPENDICES

I	BOREHOLE RECORDS
II	LABORATORY TEST DATA
III	SITE PLAN

FOREWORD

The following Conditions and Notes on Site Investigation Procedures should be read in conjunction with this report.

General.

Recommendations made, and opinions expressed in the report are based on the strata observed in the exploratory holes, together with the results of in-situ and laboratory tests. No responsibility can be held for conditions which have not been revealed by exploratory work, or which occur between exploratory hole locations. Whilst the report may suggest the likely configuration of strata, both between exploratory hole locations, or below the maximum depth of the investigation, this is only indicative, and liability cannot be accepted for its accuracy.

Unless specifically stated, no account has been taken of possible subsidence due to mineral extraction below or close to the site.

Boring Procedures.

Unless otherwise stated, the 'Shell and Auger' technique of soft ground boring has been employed. All boring operations sampling and/or logging of soils and in-situ testing complies with the recommendations of the British Standard Code of Practice BS 5930 (1981), 'Site Investigation' and BS 1377:1990, 'Methods of test for soils for civil engineering purposes'.

Whilst the technique allows the maximum data to be obtained in soft ground, some disturbance and variation of soft and layered soils is unavoidable. Attention is drawn to this condition, whenever it is suspected. Where cobbles and boulders are recorded, no conclusion should be drawn concerning the size, presence, lithological nature, or numbers per unit volume of ground.

Where peat has been encountered during siteworks, samples have been logged in accordance with the Von Post Classification (ref. Von Post, L. 1992. Sveriges Gologiska Undersoknings torvinventering och nogra av dess hittills vunna resultat (SGU peat inventory and some preliminary results) Svenska Mosskulturforeningens Tidskrift, Jonkoping, Swedden, 36, 1-37 & Hobbs N. B. Mire morphology and the properties of some British and foreign peats. QJEG, Vol. 19, 1986).

Routine Sampling.

Undisturbed samples of soils, predominantly cohesive in nature are obtained unless otherwise stated by a 104mm diameter open-drive tube sampler. In granular soils, and where undisturbed sampling is inappropriate, disturbed samples are collected. Smaller disturbed samples are also recovered at intervals to allow a visual examination of the full strata section.

In-Situ Testing.

Standard penetration tests, utilising either the standard split spoon sampler or solid cone and automatic trip-hammer are conducted unless otherwise where required by instruction. Subsequent to a seating drive of 150mm, a summation for the number of blows for 300mm penetration is recorded on the boring records together with the blow count for each 75mm penetration. In cases where incomplete penetration is obtained, the number of blows for the recorded value of penetration are noted. In coarse granular soils, a cone end is fitted to the sampler and a similar procedure adopted.

Groundwater.

The depth of entry of any influx of groundwater is recorded during the course of boring operations. However, the normal rate of boring does not usually permit the recording of an equilibrium level for any one water strike. Where possible drilling is suspended for a period of twenty minutes to monitor the subsequent rise in water level.

Groundwater conditions observed in the borings or pits are those appertaining to the period of investigation. It should be noted however, that groundwater levels are subject to diurnal, seasonal and climatic variations and can also be affected by drainage condition, tidal variation or other causes.

Retention of Samples.

After satisfactory completion of all the scheduled laboratory tests on any sample, the remaining material is discarded unless a period of retention of samples is agreed, it is our normal practice to discard all soil samples one month after submission of our final report.

REPORT ON A SITE INVESTIGATION
FOR A DEVELOPMENT
AT QUARRY ROAD
CABRA
FOR
MURRAY PARTNERSHIP

Report No. 11697

APRIL 2006

I Introduction

A new residential development is proposed for a site at Quarry Road in Cabra, Dublin. An investigation of sub soil conditions has been ordered by the project engineers on behalf of the Murray Partnership.

The ground investigation was completed over two phases with Phase 1 comprising a gas spike survey (IGSL report no. 11658) and Phase 2 comprised of the following

- ✓ The construction of four boreholes, to establish geotechnical criteria on which to base foundation design. Work was carried out in accordance with BS 5930, Code of Practice for Site Investigations (1999).
- ✓ The completion of six shallow window sample holes to recover samples for environmental analysis.
- ✓ A programme of geotechnical and environmental laboratory testing to confirm soil parameters.

This report includes all factual data pertaining to the project and comments on the geotechnical findings relative to foundation design.

II Fieldwork

The site is located at Quarry Road in Cabra at an existing commercial premises. Borehole and Window Sample locations are indicated on the site plan enclosed in Appendix IV. Borehole 3 and Window Samples WS 4, 5 and 6 were located within an existing building. At these locations the concrete surface was pre-drilled and reinstated on completion of the works.

a. Boreholes

The exploratory holes were bored with conventional 200mm cable-tool methods using a Dando Exploratory Rig.

Detailed geotechnical records are contained in Appendix I to this report - the records give details of stratification, sampling, in-situ testing and groundwater. Note is also taken of any obstructions to normal boring requiring the use of the heavy chisel for advancement. It was not possible to recover undisturbed samples because of the high stone/cobble content of the strata encountered.

Concrete floors and dense hardcore fill is noted at each location to approximately 0.50 metres. This is followed by a stratum of firm grey brown sandy gravelly CLAY which extends to approximately 2.80 metres. A thin (100mm) layer of fine sand is then noted at BHs1, 2 and 4. Below this sand layer and directly below the brown gravelly clay at BH 3, a very stiff to hard stratum of black gravelly CLAY is encountered.

This deposit has been penetrated to refusal depths varying from 4.90 to 6.20 metres, using the heavy chisel to assist advancement. The final bored depths are not indicative of rock horizon, borehole refusals may have been on boulders in the hard clay matrix.

The sub soils encountered represent the glacial till succession of the Dublin area, locally referred to as brown boulder clay overlying black boulder clay.

Ground water was noted in the boreholes, both in the thin sand stratum dividing the clay layers and at the base of some of the boreholes. Slotted standpipes were installed to record ground water variation on the site. The standpipes have been sealed at surface with Bentonite and are protected by a steel manhole cover.

Window Samples

Six shallow soil cores have been recovered at specified locations using window sampling techniques.

These undisturbed cores which are retained in plastic coreliner and were returned to IGSL's offices where they were split and examined. A record of completed window sample records is enclosed in Appendix II.

Representative sub samples were taken from these window samples and these were dispatched to Alcontrol's specialist environmental laboratory for detailed environmental analysis.

III Testing

(a) In-Situ Tests

Standard penetration tests were carried out at approximate 1.00 metre intervals in the geotechnical boreholes to measure relative in-situ soil strength. N values are noted in the right hand column of the boring records, representing the blow count required to drive the standard sampler 300mm into the soil, following initial seating blows. Where full test penetration was not achieved the blow count for a specific penetration is recorded, or refusal is indicated where appropriate.

The results of the tests are summarised as follows:

STRATUM	N VALUE RANGE	COMMENT
Brown Gravelly Clay	13 to 18	Firm
Black Gravelly Clay	38 to 67	Very Stiff / Hard

Some limited penetration SPT tests were recorded on cobbles or boulders in the hard glacial clay.

(b) Geotechnical Laboratory Tests

All geotechnical samples from the boreholes have been returned to the IGSL laboratory for initial visual inspection, a schedule of testing was prepared and tests as appropriate carried out (results are enclosed in Appendix III).

Testing included the following elements:

- a. Classification (Liquid and Plastic Limits)
- b. Grading Analysis
- c. Sulphate and pH determination

Classification Tests

The liquid and plastic limits were established for samples of the brown and black gravelly clays. Values are tabulated with relevant moisture contents and plotted on the standard Casagrande Chart, falling uniformly in the CL Zone. The results indicate that the clay is of uniformly low plasticity with a high sensitivity to moisture content variation.

Grading

Two samples had particle size distribution established by wet sieve analysis and hydrometer analysis. The curves are typically straight line in nature, reflecting the heterogeneous nature of the glacial material, smoothly graded from the clay to coarse gravel fraction.

Sulphate and pH

Chemical tests on the soils indicate low sulphate concentrations and neutral pH values. No special precautions are indicated to protect foundation concrete.

The laboratory data is consistent with the large body published data on the glacial soils of the greater Dublin area.

c. Environmental Laboratory

A total of four samples were sent to Alcontrol Geochem Ireland to establish any possible soil contamination relation to prior use of the site.

The samples were tested in accordance with the Murphy Suite of tests which covers a range of determinants specified by the Murphy Landfill Group which operates the main landfills in the Naul and at other locations in North County Dublin.

The results of the environmental tests are enclosed in Appendix IV to this report.

Murphy Test Suite Comparison Table

Murphy Landfill Summary Sheet

IGSL Contract Reference

Quarry Road Cabra

Parameter	Liquid:Soild=10 l/g mg/kg dry substance	Total Pollutant Content mg/kg dry substance	Limit for Acceptance mg/kg dry substance	Test Result Achieved Shanowen Road
Arsenic	x	n/a	0.5	< 0.01
Barium	x	n/a	20	3.65
Cadmium	x	n/a	0.04	< 0.01
Total Chromium	x	n/a	0.5	0.06
Copper	x	n/a	2	0.16
Mercury	x	n/a	0.01	< 0.0005
Molybdenum	x	n/a	0.5	0.35
Nickel	x	n/a	0.4	0.01
Lead	x	n/a	0.5	0.04
Antimony	x	n/a	0.06	< 0.01
Selenium	x	n/a	0.1	0.08
Zinc	x	n/a	4	0.3
Chloride	x	n/a	800	50
Fluoride	x	n/a	10	8
Sulphate	x	n/a	1000	463
Phenol Index	x	n/a	1	< 0.10
DOC	x	n/a	500	73
TDS	x	n/a	4000	1060
TOC	x	n/a	30000	0.91%
BTEX	n/a	x	6	<10.0
PCB (7 Concergens)	n/a	x	1	<1.0
Mineral Oil	n/a	x	500	<1.0
Total PAH	n/a	x	6	1.6

Note 1

Note 2

Note 3

Test results shown are the highest recorded values from the samples tested at Alcontrol

Tests carried out for Murphy Suite at Alcontrol Geochem Ireland

Where elevated levels have been recorded then the range of values has been show

IV Discussion

The investigation has indicated the presence of glacial till deposits underlying shallow surface fill. The glacial till consists of firm to stiff brown gravelly clay which extends to about 2.70 metres, overlying very stiff to hard grey black gravelly clay. A thin water bearing sand layer divides the brown and black till. Boreholes were terminated at depths between 5.00 and 6.00 metres, possibly reflecting the limestone rock horizon, although rock has not been proof cored.

Field and laboratory tests in the upper brown gravelly clay indicate an allowable bearing pressure of the order of 175 kN/sq.m. at depths ranging from 1.00 to 2.50 metres.

An increase in allowable bearing with depth of penetration to the black lodgement till is indicated by the test data and 400 kN/sq.m. can readily be supported at approximately 3.00 metres BGL.

The glacial till is over-consolidated and consequently settlement under the above loads will be very low, with negligible differential movement anticipated.

The heterogeneous nature of the glacial sub soils is emphasised and variation from hard clay to dense sand or gravel (sometimes water bearing) can occur randomly. Careful visual examination of excavated formation is advised to ensure uniformity and suitability of the founding medium. Any unsuitable material should be removed and replaced by low-grade concrete.

Ground water has been noted in the thin sand layer below the brown glacial till, this may cause some minor difficulties in deep excavation. Pumping from local sumps should permit construction in dry conditions. Ground water was also noted at the base of several of the boreholes, these water strikes are unlikely to affect traditionally constructed foundations.

Where basement construction is envisaged, support of vertical excavations may be required, particularly if sufficient space is not available for safe side slopes. The use of traditional steel sheet piles, penetrating to the black boulder clay is recommended.

A standing water table of about 1.50 metres BGL should be adopted in uplift design of tanked basement structures.

If sufficient space is available, temporary excavation slopes of 1.5 to 1.00 (Horizontal to Vertical) should be satisfactory.

SUMMARY

Glacial tills are present on this site below about 0.50 metres and can be used as the founding medium for direct foundations. The allowable bearing increases from about 175 kN./sq.m, in the upper brown boulder clay to 400 kN/sq.m. in the hard black lodgement till at about 3.00 metres BGL. Ground water ingress is light and will not be of major concern.

IGSL/JC
APRIL 2006

Appendix I – Cable Tool Borehole Records



GEOTECHNICAL BORING RECORD

REPORT NUMBER

11697

CONTRACT Quarry Road Cabra, Dublin

BOREHOLE NO. BH1

SHEET Sheet 1 of 1

CO-ORDINATES(_)

GROUND LEVEL (m)

BOREHOLE DIAMETER (mm) 200

DATE STARTED 18/04/2006

DATE COMPLETED 18/04/2006

CLIENT Murray Partnership
ENGINEER

BOREHOLE DEPTH (m) 4.90

CASING DEPTH (m) 4.90

BORED BY F McEvoy

PROCESSED BY C.Killaly

Depth (m)	Description	Legend	Elevation	Depth (m)	Samples			Field Test Results	Standpipe Details
					Ref. Number	Sample Type	Depth (m)		
0	MADE GROUND comprised of concrete			0.10	2118	B	1.00-1.50	N = 17 (3, 4, 4, 4, 4, 5)	
	MADE GROUND comprised of hardcore fill material			0.30					
	MADE GROUND comprised of black sandy gravelly CLAY with ash and red brick			0.50					
1	Firm grey brown sandy very gravelly CLAY with occasional subrounded, subangular cobbles and boulders				2119	B	2.00-2.50	N = 14 (2, 3, 4, 4, 3, 3)	
2									
	Medium dense fine brown SAND			2.60	2120	B	3.00-3.50	N = 59 (6, 8, 9, 11, 19, 20)	
3	Very stiff black sandy very gravelly CLAY with occasional subrounded, subangular and angular cobbles and boulders			2.70					
4					2121	B	4.00-4.50	N = 62 (9, 11, 12, 14, 17, 19)	
5	Obstruction End of Borehole at 4.90 m			4.85				N = 50/10 mm (25, 50)	
				4.90					
6									
7									
8									
9									

HARD STRATA BORING/CHISELLING

From (m)	To (m)	Time (h)	Comments
0	0.5	0.75	
4.7	4.9	2	

WATER STRIKE DETAILS

Water Strike	Casing Depth	Sealed At	Rise To	Time (hh:mm)	Comments
2.60	2.60	3.00	2.10		Moderate
4.90	4.90		3.60		

GROUNDWATER DETAILS

INSTALLATION DETAILS					Date	Hole Depth	Casing Depth	Depth to Water	Comments
Date	Tip Depth	RZ Top	RZ Base	Type	18-04-06	4.90	0.00	4.60	End of Borehole
18/04/2006	4.90	1.00	4.90	50mm SP					

REMARKS 1.5 hrs standing waiting for key to gain access to Site



GEOTECHNICAL BORING RECORD

REPORT NUMBER

11697

CONTRACT Quarry Road Cabra, Dublin

BOREHOLE NO.

BH2

SHEET

Sheet 1 of 1

CO-ORDINATES(_)

GROUND LEVEL (m)

BOREHOLE DIAMETER (mm) 200

DATE STARTED

18/04/2006

DATE COMPLETED

18/04/2006

CLIENT Murray Partnership

BOREHOLE DEPTH (m) 4.80

BORED BY

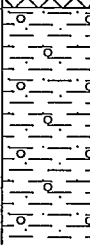
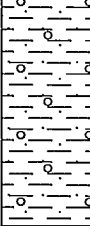
F McEvoy

ENGINEER

CASING DEPTH (m) 4.80

PROCESSED BY

C.Killaly

Depth (m)	Description	Legend	Elevation	Depth (m)	Samples			Field Test Results	Standpipe Details
					Ref. Number	Sample Type	Depth (m)		
0	MADE GROUND comprised of hardcore fill material				2122	B	1.00-1.50	N = 15 (2, 3, 3, 4, 4, 4)	
	MADE GROUND comprised of grey brown CLAY with ash, some sand, cobbles and red brick			0.20					
	Firm grey brown sandy very gravelly CLAY with occasional subrounded, subangular cobbles and boulders			0.50					
1									
2									
	Medium dense fine brown SAND		2.60	2124	B	3.00-3.50	N = 53 (6, 8, 9, 11, 16, 17)		
3	Very stiff black sandy very gravelly CLAY with occasional subrounded, subangular and angular cobbles and boulders		2.70						
4									
	Obstruction		4.75	2125	B	4.00-4.50	N = 67 (6, 9, 10, 14, 18, 25)		
5	End of Borehole at 4.80 m		4.80						
6									
7									
8									
9									

HARD STRATA BORING/CHISELLING

From (m)	To (m)	Time (h)	Comments
0	0.5	0.75	
4.45	4.8	2.5	

WATER STRIKE DETAILS

Water Strike	Casing Depth	Sealed At	Rise To	Time (hh:mm)	Comments
2.60	2.60	2.50	1.70		Moderate
4.80	4.80		3.20		

GROUNDWATER DETAILS

INSTALLATION DETAILS

Date	Tip Depth	RZ Top	RZ Base	Type	Date	Hole Depth	Casing Depth	Depth to Water	Comments
18/04/2006	4.80	1.00	4.80	50mm SP	18-04-06	4.80	0.00	4.00	End of Borehole

REMARKS

IGSL BH LOG 11697.GPJ IGSL.GDT 24/4/06



GEOTECHNICAL BORING RECORD

REPORT NUMBER

11697

CONTRACT Quarry Road Cabra, Dublin

BOREHOLE NO. BH3

SHEET Sheet 1 of 1

CO-ORDINATES(_)

GROUND LEVEL (m)

BOREHOLE DIAMETER (mm) 200

DATE STARTED 19/04/2006

DATE COMPLETED 20/04/2006

CLIENT Murray Partnership

BOREHOLE DEPTH (m) 5.30

BORED BY F McEvoy

ENGINEER

CASING DEPTH (m) 5.30

PROCESSED BY C.Killaly

Depth (m)	Description	Legend	Elevation	Depth (m)	Samples			Field Test Results	Standpipe Details
					Ref. Number	Sample Type	Depth (m)		
0	MADE GROUND comprised of concrete			0.10					
	MADE GROUND comprised of hardcore fill material with rounded and subrounded cobbles			0.50					
1	Firm grey brown sandy very gravelly CLAY with occasional subrounded, subangular cobbles and boulders				2131	B	1.00-1.50	N = 14 (3, 3, 3, 4, 3, 4)	
2					2132	B	2.00-2.50	N = 18 (2, 4, 4, 4, 5, 5)	
3	Very stiff black sandy very gravelly CLAY with occasional subrounded, subangular and angular cobbles and boulders			2.80	2133	B	3.00-3.50	N = 54 (6, 9, 10, 11, 17, 16)	
4					2134	B	4.00-4.50	N = 52 (4, 7, 8, 12, 15, 17)	
5	Obstruction End of Borehole at 5.30 m			5.25 5.30				N = 77/150 mm (6, 10, 27, 50)	
6									
7									
8									
9									

HARD STRATA BORING/CHISELLING

From (m)	To (m)	Time (h)	Comments
0	0.3	1	
5	5.3	2	

WATER STRIKE DETAILS

Water Strike	Casing Depth	Sealed At	Rise To	Time (hh:mm)	Comments
5.20	5.20		3.70		Moderate

GROUNDWATER DETAILS

INSTALLATION DETAILS					Date	Hole Depth	Casing Depth	Depth to Water	Comments
Date	Tip Depth	RZ Top	RZ Base	Type	20-04-06	5.30	0.00	4.00	End of Borehole
20/04/2006	5.30	1.00	5.30	50mm SP					

REMARKS 1 hr EOM due to difficult access



GEOTECHNICAL BORING RECORD

REPORT NUMBER

11697

CONTRACT Quarry Road Cabra, Dublin

BOREHOLE NO. BH4

SHEET Sheet 1 of 1

CO-ORDINATES(_)

GROUND LEVEL (m)

BOREHOLE DIAMETER (mm) 200

DATE STARTED 19/04/2006

DATE COMPLETED 19/04/2006

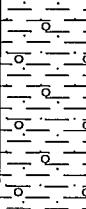
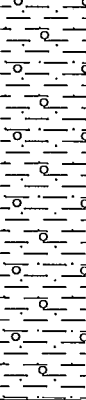
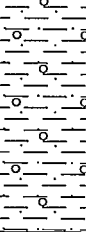
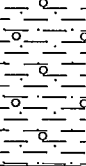
CLIENT Murray Partnership
ENGINEER

BOREHOLE DEPTH (m) 6.20

CASING DEPTH (m) 6.20

BORED BY F McEvoy

PROCESSED BY C.Killaly

Depth (m)	Description	Legend	Elevation	Depth (m)	Samples			Field Test Results	Standpipe Details
					Ref. Number	Sample Type	Depth (m)		
0	MADE GROUND comprised of concrete			0.05	2126	B	1.00-1.50	N = 13 (2, 2, 3, 3, 3, 4)	
	MADE GROUND comprised of hardcore fill material			0.20					
	MADE GROUND comprised of firm grey brown sandy gravelly CLAY with red brick fill			0.50					
1	Firm grey brown sandy gravelly CLAY with occasional subrounded and subangular cobbles and boulders								
2					2127	B	2.00-2.50	N = 15 (2, 3, 3, 4, 4, 4)	
	Medium dense brown fine SAND			2.50	2128	B	3.00-3.50	N = 38 (5, 6, 7, 9, 10, 12)	
3	Stiff to very stiff black sandy very gravelly CLAY with occasional subrounded, subangular and angular cobbles and boulders			2.60					
4									
5					2130	B	5.00-5.50	N = 40 (5, 6, 8, 9, 9, 14)	
6								N = 80/150 mm (9, 14, 30, 50)	
	Obstruction			6.15					
	End of Borehole at 6.20 m			6.20					
7									
8									
9									

HARD STRATA BORING/CHISELLING

From (m)	To (m)	Time (h)	Comments
0	0.2	0.75	
6	6.2	2	

WATER STRIKE DETAILS

Water Strike	Casing Depth	Sealed At	Rise To	Time (hh:mm)	Comments
2.50	2.50	2.70	2.10		Moderate
6.20	6.20		2.10		

GROUNDWATER DETAILS

INSTALLATION DETAILS

Date	Tip Depth	RZ Top	RZ Base	Type	Date	Hole Depth	Casing Depth	Depth to Water	Comments
16/04/2006	6.20	1.00	6.20	50mm SP	19-04-06	6.20	0.00	6.00	End of Borehole

REMARKS

IGSL BH LOG 11697.GPJ IGSL GDT 24/4/06

Appendix II – Window Sample Records

GROUND INVESTIGATION
FOR A DEVELOPMENT
AT QUARRY ROAD
CABRA
FOR
MURRAY PARTNERSHIP

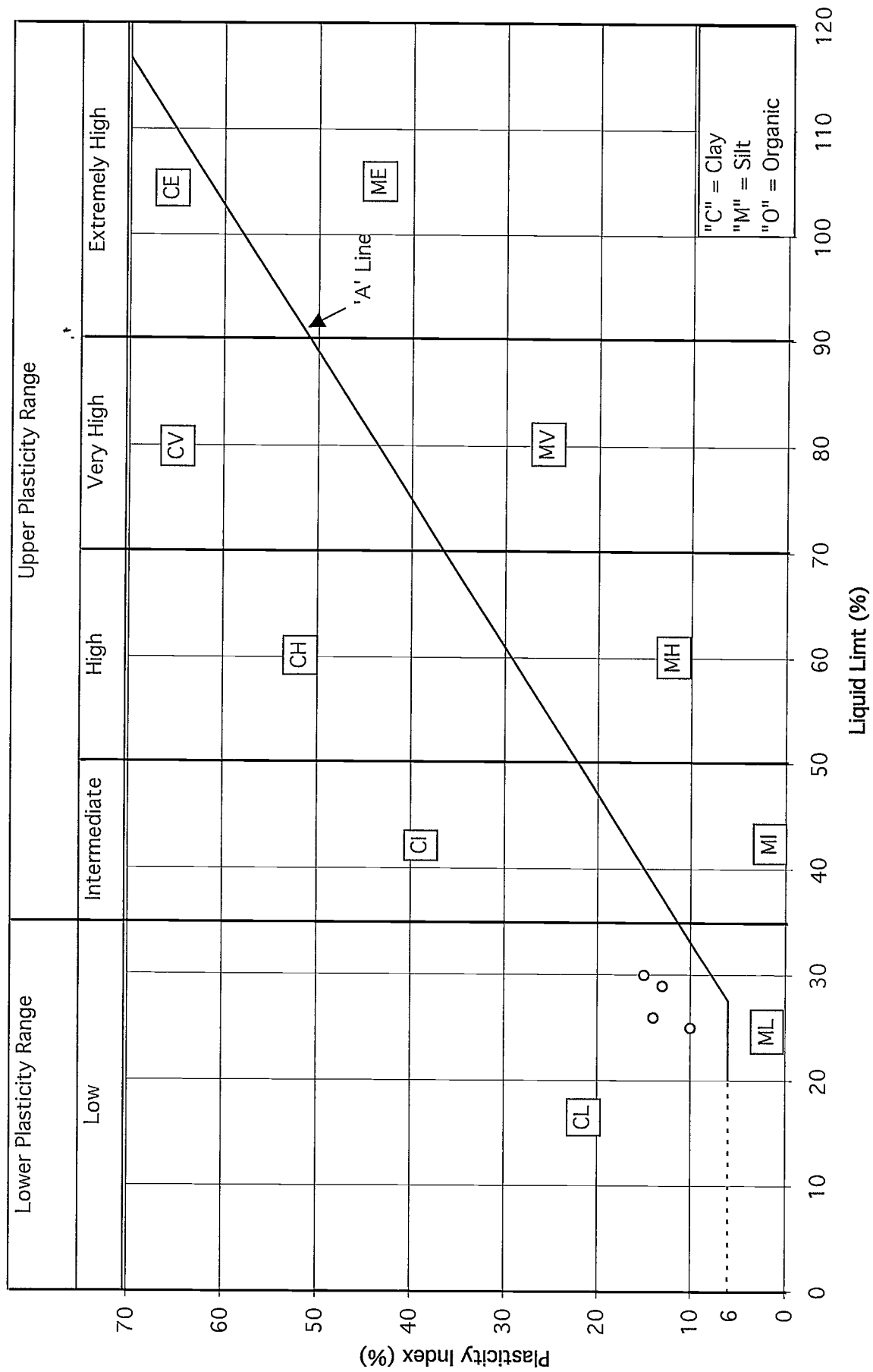
WINDOW SAMPLE SUMMARY DETAILS

Location	Depth	Sub Samples
WS1	0 – 1.80m	Yes
WS2	0 – 2.00m	Yes
WS3	0 – 2.80m	Yes
WS4	0 – 2.80m	Yes
WS5	0 – 3.00m	Yes
WS6	0 – 1.00m	Yes

DP's 4, 5 and 6 through warehouse floor requiring concrete breaking

**Appendix III – Laboratory Test Records
(Geotechnical)**

11697		CLASSIFICATION TEST RESULTS				TO BS 1377:1990:PART2:CL 4 & 5				IGSL	
Contract: QUARRY ROAD CABRA											
BH No.	Depth (M)	Ref No.	Description	Passing 425um %	Test Code	Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)	Classification	Water Content %	
1	3.00	2120	Grey black sandy gravelly CLAY		A	25	15	10	C L	9.34	
2	2.00	2123	Brown sandy gravelly CLAY		A	29	16	13	C L	13.56	
3	3.00	2133	Black sandy gravelly CLAY		A	26	12	14	C L	8.82	
4	1.00	2126	Brown sandy gravelly CLAY		A	30	15	15	C L	14.05	
Code:				A -Air dried, -425 micron specimen				N - Natural specimen after removal of coarse material			



Report No.	11697	PARTICLE SIZE ANALYSIS		I.G.S.I
CONTRACT: QUARRY ROAD LOCATION No. BH 1 SAMPLE No. 2119 DEPTH: 2.00 TEST METHOD: Wet Sieve and Hydrometer DESCRIPTION: Grey brown sandy gravelly CLAY				
particle size	% passing			
75	100.0	COBBLES		
63	100.0	GRAVEL		
50	100.0			
37.5	100.0			
28	91.3			
20	83.5			
14	78.7	SAND		
10	75.0			
6.3	67.8			
5	65.1			
3.35	60.3			
2	54.9	SILT/CLAY		
1.18	51.7			
0.6	47.8			
0.425	44.6			
0.3	43.1			
0.15	35.2			
0.063	31.2			
0.04	28.4			
0.03	26.3			
0.02	23.9			
0.013	19.8			
0.009	15.9			
0.005	12.5			
0.002	8.6			

The graph plots % passing (0.0 to 100.0) against sieve size in millimeters on a logarithmic scale (0.0001 to 100). The curve starts at 100% passing for 75mm and decreases to 8.6% passing at 0.002mm. Key classification boundaries are marked: COBBLES (>75mm), GRAVEL (75mm to 0.425mm), SAND (0.425mm to 0.075mm), and SILT/CLAY (<0.075mm).

Sieve Size (mm)	% Passing
75	100.0
63	100.0
50	100.0
37.5	100.0
28	91.3
20	83.5
14	78.7
10	75.0
6.3	67.8
5	65.1
3.35	60.3
2	54.9
1.18	51.7
0.6	47.8
0.425	44.6
0.3	43.1
0.15	35.2
0.063	31.2
0.04	28.4
0.03	26.3
0.02	23.9
0.013	19.8
0.009	15.9
0.005	12.5
0.002	8.6

Report No.	11697		PARTICLE SIZE ANALYSIS I.G.S.L																																																																														
CONTRACT: QUARRY ROAD LOCATION No. BH 4 SAMPLE No. 2128 DEPTH: 3.00 TEST METHOD: Wet Sieve and Hydrometer DESCRIPTION: Black sandy gravelly CLAY <table border="1" style="margin-top: 10px; width: 100%; font-size: small;"> <caption>Graph Data Points (Approximate)</caption> <thead> <tr> <th>Particle Size (mm)</th> <th>% Passing</th> </tr> </thead> <tbody> <tr><td>75</td><td>100.0</td></tr> <tr><td>63</td><td>100.0</td></tr> <tr><td>50</td><td>100.0</td></tr> <tr><td>37.5</td><td>96.7</td></tr> <tr><td>28</td><td>92.3</td></tr> <tr><td>20</td><td>87.6</td></tr> <tr><td>14</td><td>79.0</td></tr> <tr><td>10</td><td>74.2</td></tr> <tr><td>6.3</td><td>66.9</td></tr> <tr><td>5</td><td>63.3</td></tr> <tr><td>3.35</td><td>59.1</td></tr> <tr><td>2</td><td>51.7</td></tr> <tr><td>1.18</td><td>48.0</td></tr> <tr><td>0.6</td><td>44.2</td></tr> <tr><td>0.425</td><td>42.3</td></tr> <tr><td>0.3</td><td>38.7</td></tr> <tr><td>0.15</td><td>33.9</td></tr> <tr><td>0.063</td><td>30.4</td></tr> <tr><td>0.04</td><td>24.7</td></tr> <tr><td>0.03</td><td>22.5</td></tr> <tr><td>0.02</td><td>21.2</td></tr> <tr><td>0.013</td><td>17.3</td></tr> <tr><td>0.009</td><td>15</td></tr> <tr><td>0.005</td><td>11.1</td></tr> <tr><td>0.002</td><td>7.4</td></tr> </tbody> </table>				Particle Size (mm)	% Passing	75	100.0	63	100.0	50	100.0	37.5	96.7	28	92.3	20	87.6	14	79.0	10	74.2	6.3	66.9	5	63.3	3.35	59.1	2	51.7	1.18	48.0	0.6	44.2	0.425	42.3	0.3	38.7	0.15	33.9	0.063	30.4	0.04	24.7	0.03	22.5	0.02	21.2	0.013	17.3	0.009	15	0.005	11.1	0.002	7.4																										
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Report No. 11697		SULPHATE CONTENT & pH				IGSL
Contract:						
QUARRY ROAD						
Location BH	Depth (m)	Reference No.	Description	WATER	SOIL	pH
				Parts per 100,000	Percentage Sulphates	
1	2.00	2119	CLAY		0.06	7.5
3	2.00	2132	CLAY		0.09	7.8
Note: SO4 = SO3 x 1.2						

**Appendix IV – Laboratory Test Records
(Environmental)**

Table Of Results

Ref Number: 06-B02246/01

Sample Type: SOIL

Client: IGSL Ltd

Location: QUARRY RD CABRA DUBLIN

Date of Receipt: 4/12/06

Client Contact: John Clancy

(of first sample)

Client Ref: 11697

ALcontrol Reference	Sample Identity	Other ID	Detection Method Method Detection Limit UKAS Accredited										
			CV AA <0.0005mg/kg	GC <10ug/kg	GC <10ug/kg	GC <10ug/kg	GC <10ug/kg	GC <10ug/kg	GC <10ug/kg	GC <10ug/kg	GC <10ug/kg	GCMS <1.6mg/kg	GCMS <1ug/kg
06-B02246-S0004	WS 1 - A	UNKNOWN											
06-B02246-S0005	WS 1 - B	UNKNOWN											
06-B02246-S0006	WS 2 - C	UNKNOWN											
06-B02246-S0007	WS 3 - E	UNKNOWN											
06-B02246-S0008	WS 3 - F	UNKNOWN											
06-B02246-S0009	WS 3 - G	UNKNOWN											
06-B02246-S0010	WS 4 - H	UNKNOWN											
06-B02246-S0011	WS 4 - I	UNKNOWN											
06-B02246-S0012	WS 4 - J	UNKNOWN											
06-B02246-S0013	WS 5 - K	UNKNOWN											
06-B02246-S0014	WS 5 - L	UNKNOWN											
06-B02246-S0015	WS 5 - M	UNKNOWN											
06-B02246-S0016	WS 6 - N	UNKNOWN											
06-B02246-S0017	WS 2 - D	UNKNOWN											
			Dissolved Mercury Low CEN 10:1 Leachate	mg/kg									
			Petrol Range Organics C9	ug/kg									
			Petrol Range Organics C10-12	ug/kg									
			Benzene	ug/kg									
			Toluene	ug/kg									
			Ethylbenzene	ug/kg									
			Total Xylene	ug/kg									
			Mineral Oil by GC	mg/kg									
			Total 6 PAHs	mg/kg									
			PCB Congener 28	ug/kg									
			PCB Congener 52	ug/kg									
			PCB Congener 101	ug/kg									
			PCB Congener 118	ug/kg									
			PCB Congener 153	ug/kg									
			PCB Congener 138	ug/kg									

Notes : METHOD DETECTION LIMITS ARE NOT ALWAYS ACHIEVABLE DUE TO VARIOUS CIRCUMSTANCES BEYOND OUR CONTROL.

NDP = NO DETERMINATION POSSIBLE

THE DATA ON THIS PRELIMINARY REPORT IS NOT VALIDATED AND MAY BE SUBJECT TO CHANGE.

Checked By : Ann-Marie Rutledge

Table Of Results

Ref Number: 06-B02246/01

Sample Type: SOIL

Client: IGSL Ltd

Location: QUARRY RD CABRA DUBLIN

Date of Receipt: 4/12/06

Client Contact: Jphn Clancy

(of first sample)

Client Ref: 11697

ALcontrol Reference	Sample Identity	Detection Method Method Detection Limit UKAS Accredited	GC/MS	GC/MS	HPLC	ICP MS	ICP MS	ICP MS	ICP MS	ICP MS	ICP MS	ICP MS	ICP MS
			<1ug/kg	<1ug/kg	<0.1mg/kg	<0.01mg/kg	<0.01mg/kg	<0.01mg/kg	<0.01mg/kg	<0.01mg/kg	<0.01mg/kg	<0.01mg/kg	<0.01mg/kg
06-B02246-S0004	WS 1 - A	UNKNOWN	<1	<1	<0.1	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
06-B02246-S0005	WS 1 - B	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0006	WS 2 - C	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0007	WS 3 - E	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0008	WS 3 - F	UNKNOWN	<1	<1	<0.1	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
06-B02246-S0009	WS 3 - G	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0010	WS 4 - H	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0011	WS 4 - I	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0012	WS 4 - J	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0013	WS 5 - K	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0014	WS 5 - L	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
06-B02246-S0015	WS 5 - M	UNKNOWN	<1	<1	<0.1	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
06-B02246-S0016	WS 6 - N	UNKNOWN	<1	<1	<0.1	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
06-B02246-S0017	WS 2 - D	UNKNOWN	-	-	-	-	-	-	-	-	-	-	-
Other ID			mg/kg										
Dissolved Zinc Low CEN 10:1 Leach			mg/kg										
Dissolved Selenium Low CEN 10:1 Leach			mg/kg										
Dissolved Nickel Low CEN 10:1 Leach			mg/kg										
Dissolved Molybdenum Low CEN 10:1 Leach			mg/kg										
Dissolved Lead Low CEN 10:1 Leach			mg/kg										
Dissolved Copper Low CEN 10:1 Leach			mg/kg										
Dissolved Chromium Low CEN 10:1 Leach			mg/kg										
Dissolved Cadmium Low CEN 10:1 Leach			mg/kg										
Dissolved Barium Low CEN 10:1 Leach			mg/kg										
Dissolved Arsenic Low CEN 10:1 Leach			mg/kg										
Dissolved Antimony Low CEN 10:1 Leach			mg/kg										
Total Phenols in CEN 10:1 Leachate			mg/kg										
Natural Moisture Content			%										
PCB Total of 7 Congeners			ug/kg										
PCB Congener 180			ug/kg										

Notes : METHOD DETECTION LIMITS ARE NOT ALWAYS ACHIEVABLE DUE TO VARIOUS CIRCUMSTANCES BEYOND OUR CONTROL.

NDP = NO DETERMINATION POSSIBLE

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Checked By : Ann-Marie Rutledge

ALcontrol Laboratories Ireland

Table Of Results

☒ Interim
☐ Validated

Ref Number: 06-B02246/01

Sample Type: SOIL

Client: IGSL Ltd

Location: QUARRY RD CABRA DUBLIN

Date of Receipt: 4/12/06
(of first sample)

Client Contact: John Clancy

Client Ref: 11697

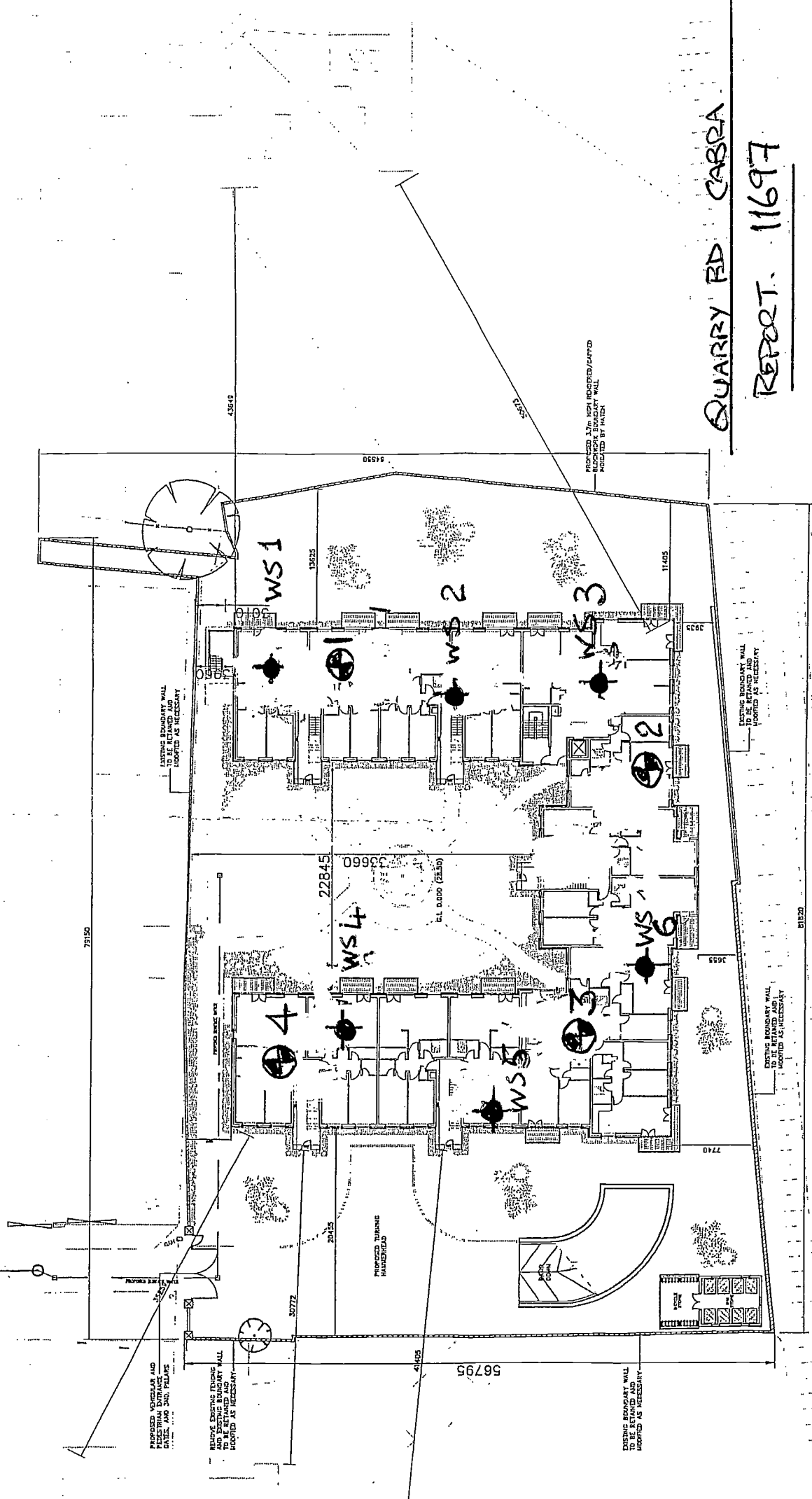
Detection Method Method Detection Limit UKAS Accredited									
IR	KONE	KONE	KONE	LECO	METER				
<20mg/kg	<10mg/kg	<1mg/kg	<30mg/kg	<0.01%	<50mg/kg				
✓	✓	✓	✓	✓					
Dissolved Organic Carbon in CEN 10:1 Leachate						mg/kg			
Chloride in CEN 10:1 Leachate						mg/kg			
Fluoride in CEN 10:1 Leachate						mg/kg			
Sulphate in CEN 10:1 Leachate						mg/kg			
Total Organic Carbon**						%			
Total Dissolved Solids CEN 10:1						mg/kg			

Notes : METHOD DETECTION LIMITS ARE NOT ALWAYS ACHIEVABLE DUE TO VARIOUS CIRCUMSTANCES BEYOND OUR CONTROL. NDP = NO DETERMINATION POSSIBLE
THE DATA ON THIS PRELIMINARY REPORT IS NOT VALIDATED AND MAY BE SUBJECT TO CHANGE.

Checked By : Ann-Marie Rutledge

Appendix V – Site Plan

- ⊙ -- DENOTES BORE HOLE
- ⊙ -- DENOTES WINDOW SAMPLE



QUARRY RD CASRA

REPORT. 11697

XX

X

XX

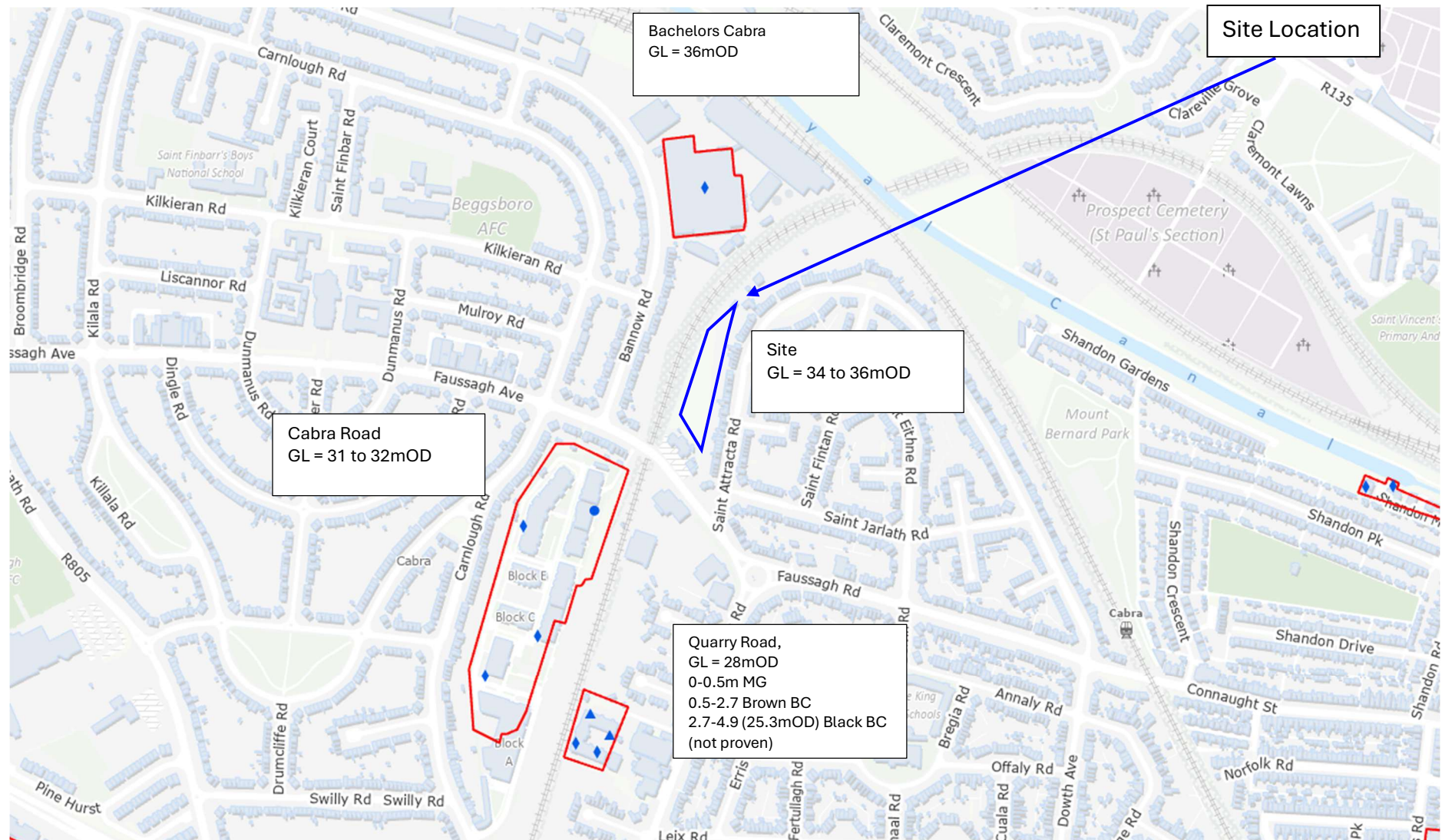
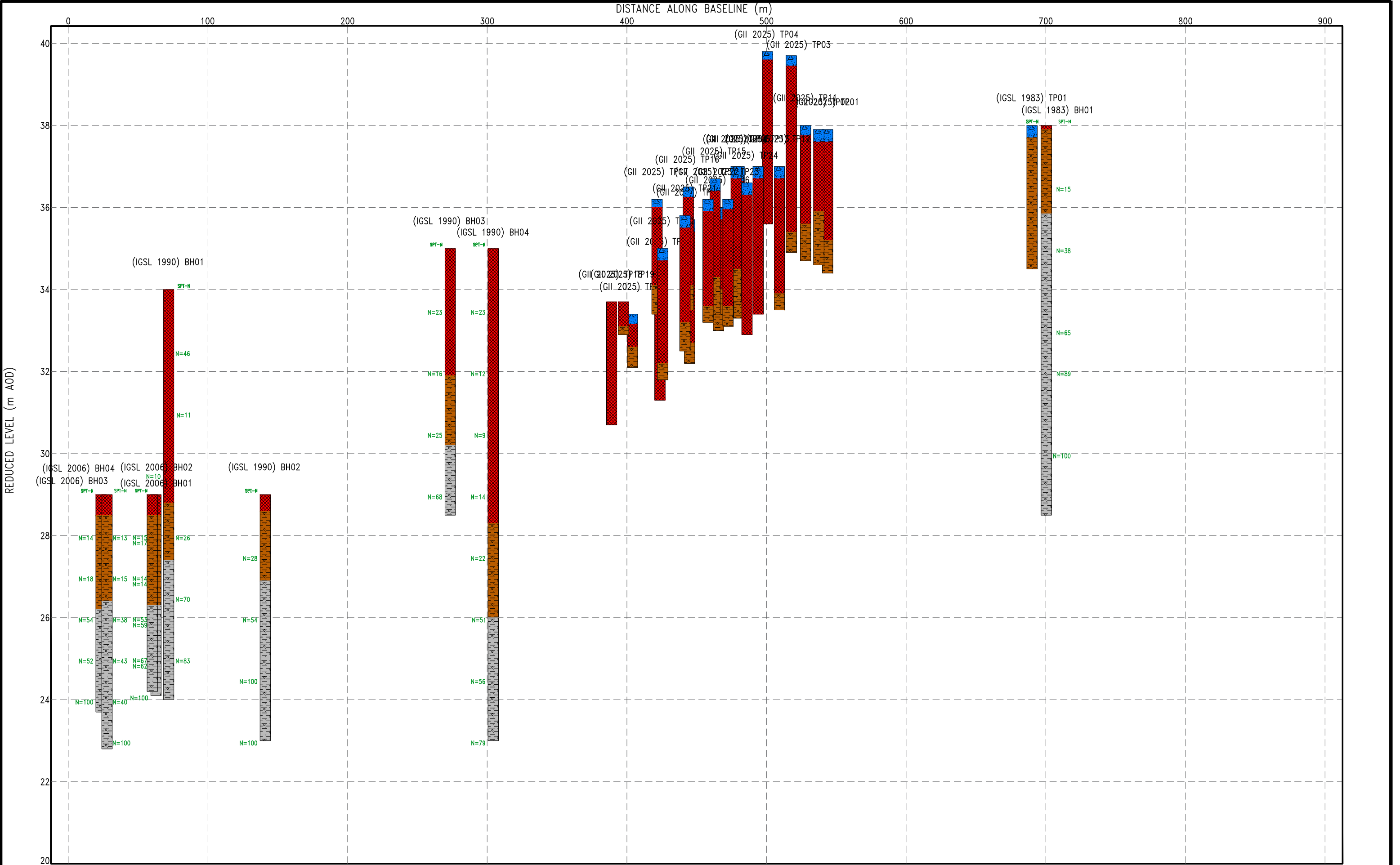


Figure 1: GSI – SI Location Map

APPENDIX C
SUBSURFACE PROFILES



APPENDIX D
AGL CALCULATIONS

Oatlands House
Bar pull-out shear resistance calculations

Designed in accordance with BS8006-2:2011

Vertical Slope height	6.70	m
Elevation of slope crest	34.74	mOD
Surcharge	10	kN/m2
Water Elevation	31	mOD
wd	3.50	m

Water Table Depth from top of slope

SOIL PARAMETERS

Made Ground Properties- Effective Stress

γ	20	kN/m ³	Bulk Unit Weight
c	50	kN/m ²	Effective Cohesion
ϕ_k	30.0	°	Slope angle of shearing resistance
k_r	0.9		BS8006-2 pg58 & 59 k_r is a factor relating the average radial effective stress around the nail to the vertical and has a value typically in the range 0.55 to 0.9
λ_r	1.00		BS8006-2 pg58 & 59 The interface factor λ_r should be taken to be between 0.7 and 1.0 with the lower value relating to smooth interfaces and the higher value relating to rough interfaces.

Brown Boulder Clay - Effective Stress

γ	21	kN/m ³	Bulk Unit Weight
c	75	kN/m ²	Effective Cohesion
ϕ_k	32.0	°	Angle of effective shearing resistance

threadbar coarse pitch (MN/m2)	5000
Effect ext Diam. (m)	0.0277
corroded perimeter (m)	8.70E-02
Force (kN)	435.11

Black Boulder Clay

γ	23	kN/m ³	Bulk Unit Weight
c	350	kN/m ²	Effective Cohesion
ϕ_k	38.0	°	Angle of effective shearing resistance

Not Used - Effective Stress

γ		kN/m ³	Bulk Unit Weight
c		kN/m ²	Effective Cohesion
ϕ_k		°	Angle of effective shearing resistance

1

5.47

5.28

Soil Nail Hole & Wall Details

d_{hole}	0.1	m	Diameter of Soil Nail hole in GT
d_{hole}		m	Diameter of Soil Nail hole in London Clay
Soil Nail Angle	20	°	Inclination to horizontal δ (deg)
S_v	1.25	m	Vertical Spacing of soil nails
S_h	1.5	m	Horizontal Spacing of soil nails
γ_k	1.5		Partial factor on bond stress for effective stress and total stress calcs: Table 5 or 6 of BS8006-2:2011

Black Boulder Clay

α	0.75		Adhesion Factor
c _u	250	kPa	Undrained shear strength of soil (<i>Ignored for granular till and gravel</i>)
Bond Stress	100	kPa	Min Grout/Tendon Bond Stress

Brown Boulder Clay

α	0.75		Adhesion Factor
c _u	75	kPa	Undrained shear strength of soil (<i>Ignored for granular till and gravel</i>)
Bond Stress	56.3	kPa	Min Grout/Tendon Bond Stress

Undrained Properties of Made Ground

α	0.75		Adhesion Factor
c _u	50	kPa	Undrained shear strength
Bond Stress	37.5	kPa	Min Grout/Tendon Bond Stress

TD40/16	f_y (kN)	95% f_y (kN)
	520	494

Ignored Length m

SLOPE/W INPUT

Nail ID	Depth to top of Nail from top of slope, z (m)	Height of top of nail from slope toe (m)	Level of nail head (mOD)	Total Length of Soil Nail (m)	Length of nail in MG (m)	Length of nail in Brown (m)	Length of nail in Black (m)		Depth to 3/4 point of nail in soil (m)	Porewater Pressure (kN/m2)	Effective Vertical Stress, σ'_v (kN/m2)	Characteristic Bond Stress (kPa)	Minimum Characteristic Bond Stress (kPa)	Design Bond Skin Friction (F/A) (kPa)	Design Bond Skin Friction (F/A) (kPa/m)	Design Pullout Resistance of nail (full length) (kN)
													max of (min grout/tendon bond stress or $\sigma'_v \times \tan(\phi_k)$)	Characteristic Bond stress / γ_k	per m (horz spacing)	Design Bond stress x Nail surface area
1	0.50	6.20	34.24	8.0	8.0	3.0			2.55	0.0	61.0	31.7	37.5	25.0	16.7	55.0
2	1.75	4.95	32.99	8.0	5.0	3.0			3.80	3.0	83.1	44.5	44.5	29.7	19.8	65.2
3	3.00	3.70	31.74	8.0	1.0	7.0			6.05	15.2	95.8	53.4	56.3	37.5	29.0	82.5
4	4.25	2.45	30.49	8.0	0.0	7.0	1.0		6.30	27.5	108.6	63.0	63.0	42.0	28.0	92.3
5	5.50	1.20	29.24	8.0	0.0	3.5	4.5		7.55	39.8	121.3	77.8	77.8	51.9	34.6	114.1



26-135

Dywidag, Geostrong and MaccaferriSelf drill hollow Steel bar properties
for the short term and long term conditions

Maccaferri

	R32-280
Nominal Thread Diameter (mm)	32
Effective External Diameter (mm)	29.34
Internal Diameter [a] (mm)	18.4
Cross Section Area [b] (mm ²)	429
Ultimate Strength (kN)	280
Yield Strength (kN)	220
Steel Grade [c] Yld (N/mm ²)	540
Average Section Modulus (cm ³)	2.21
Moment of Inertia (cm ⁴)	3.3
Thread (Left-Hand)	
Weight (kg/m)	
Cross Section Area [b] (mm ²)	410

a) Internal Øs based on nominal values

b) Cross section area based on average values

c) Steel Grades from Characteristic Strengths, in accordance with EN 10083-1

Sacrificial Steel Thickness (mm) Table 3 of BS 8006-2:2011 for Galvanised Steel	1.4
Effective External Diameter (mm)	26.54
Reduced Cross Section Area (mm ²)	287.3

60 years

Steel yield strength factor, γ_s	1.15
Design yield strength, accounting for sacrificial steel thickness and γ_s (kN)	134.9

Circular hollow sections and tubes of uniform thickness	
Shear capacity factor (pg 51 of EC3 Part 1 Section 6.2.6 (2 & 3 (g)))	0.37
Shear resistance of bar (kN)	49.6

Design Shear resistance of bar (kN) i.e. 50% of Shear Capacity	
--	--

Design of facing calculated in accordance with CIRIA C367 Soil nailing – best practice guidance and and FHWA (GEC 7-2003) modified to incorporate BS 5400

REINFORCED CONCRETE FACING

ϕ'	30	Deg	Characteristic angle of Shearing Resistance for facing design
γ	20	kN/m3	Bulk Weight of soil
γ^w	9.81	kN/m3	Bulk Weight of water
H	5.52	m	Depth to nail head
Water Head at nail head	1	m	Height of water above lowest nail
Ka	0.33		Active Earth Pressure Coefficient $(1-\sin \phi')/(1+\sin \phi')$
qp	0	kPa	Permanent surcharge (unfactored)
qv	10	kPa	Variable surcharge (unfactored)
ratio of surcharge vertical stress at bottom of slope relative to top	1		
Sh	1.6	m	Horizontal Nail Spacing (i.e., at pile locations)
Sv	1.25	m	Vertical Nail Spacing
qp	0	kPa	Permanent surcharge (unfactored) Adjusted for stress at bottom relative to top of slope
qv	10	kPa	Variable surcharge (unfactored) Adjusted for stress at bottom relative to top of slope
p	36.9	kPa	Pressure on facing=local earth pressure, Full active earth pressure
p	18.4	kPa	Pressure on facing=local earth pressure, calculated using 50% of active earth pressure
Water Pressures	9.8	kPa	Water pressures behind facing
Additional applied earth pressures	0.0	kPa	Characteristic additional earth pressure on facing
	28.2	kPa	Unfactored nail head tensile pressure
t _f	56.5	kN	Unfactored nail head tensile force
T _{o,ULT}	84.7	kN	Factored nail head tensile force (Global FoS = 1.5 for critical structures)
	46.7	kPa	Total characteristic earth pressures behind wall (using full active earth pressure & water pressure)
w	58.3	kN/m	Total characteristic earth pressures behind wall x vertical nail spacing
M _k	18.7	kNm/m	Characteristic Bending Moment; $wl^2/8$; l = sh (i.e. horizontal nail spacingl)

Check on Wire Mesh Capacity ONLY			
Mesh Type	A393		
Bar Diam	10	mm	
Bar area	7.9E-05	mm2	
Yield stress of steel	500	N/mm2	
Partial factor on yield strength of steel , γ_s	1.15		
Partial factor on yield strength of concrete , γ_c	1.5		
Shear Capacity factor	0.67	Reduction factor applied to shear resistance of bar	
Min No. of wires over bearing plate	8	No.	
punching resistance	182.1	kN	Ignores contribution of shotcrete
OK if punching resistance > T _{o,ULT}	PASS		Therefore requires shotcrete to resist punching shear

Limiting Reinforcement Ratios

shotcrete thickness (h)	150	mm	
b	1000	mm	Facing unit width (1m)
d	75	mm	Assumes mesh in middle of shotcrete
0.15%bd	112.5	mm^2/m	Minimum reinforcement ratio (per metre width)
4%bd	3000	mm^2/m	Maximum reinforcement ratio (per metre width)

Design Reinforcement ratio

a _n = a _m	393	mm^2/m	Ok - Reinforcement area within BS 5400:Part 4:1990 limits
---------------------------------	-----	--------	---

Reinforcement Details

Wire Size	10	mm	
d	75	mm	
min cover	65	mm	Ok - Reinforcement cover exceeds BS 5400:Part 4:1990 minimum values

Flexural Capacity

f _y	500	N/mm^2	Yield strength of steel
f _{cu}	40	N/mm^2	Cube strength of C40/50 concrete, cautiously taken as 40 MPa
Material Factor for concrete =	1.5		from BS 5400:Part 4:1990
A _s	393	mm^2/m	
z	69.6	mm	lever arm = $(1-(1.1A_s f_y/f_{cu} bd))d$
m _{vm} = m _{vn}	11.9	kNm/m	0.87f _y a _{mz} - square grid

Facing Flexural Resistance (R_f)

C _f	1		(CIRIA Eqn A2.9 & A2.10)
n	237.96	kN/m^2	Flexure pressure factor from Table 4.2 of FHWA
R _{ff}	475.9	kN	for β values see Table 3.15 of BS8110-1:1997
			>To therefore OK for flexural resistance

Punching Shear Capacity (R_{fp})

ξ _{ss}	1.5		(CIRIA Eqn A2.12)
100A _y /bd	0.524		
v _c	0.59		from Table 8 of BS 5400-4:1990
L _{BP}	300	mm	Bearing plate width
R _{fp}	119.4	kN	>To therefore OK for punching shear

Upper Cantilever

D _{cant}	0.5	m	
Ka	0.33		
mc	0.14	kNm/m	
m _{c,ult}	0.23	kNm/m	<M _m therefore Ok

Lower Cantilever

D _{cant}	0.5	m	
Ka	0.33		
mc	0.73	kNm/m	
m _{c,ult}	1.21	kNm/m	<M _m therefore Ok

TABLE 4.2 FACING PRESSURE FACTORS RECOMMENDED VALUES FOR DESIGN				
Nominal Facing Thickness (mm)	Temporary Facings		Permanent Facings	
	Flexure Pressure Factor C _f	Shear Pressure Factor C _s	Flexure Pressure Factor C _f	Shear Pressure Factor C _s
100	2.0	2.5	1.0	1.0
150	1.5	2.0	1.0	1.0
200	1.0	1.0	1.0	1.0

APPENDIX E

SLOPE/W RESULTS

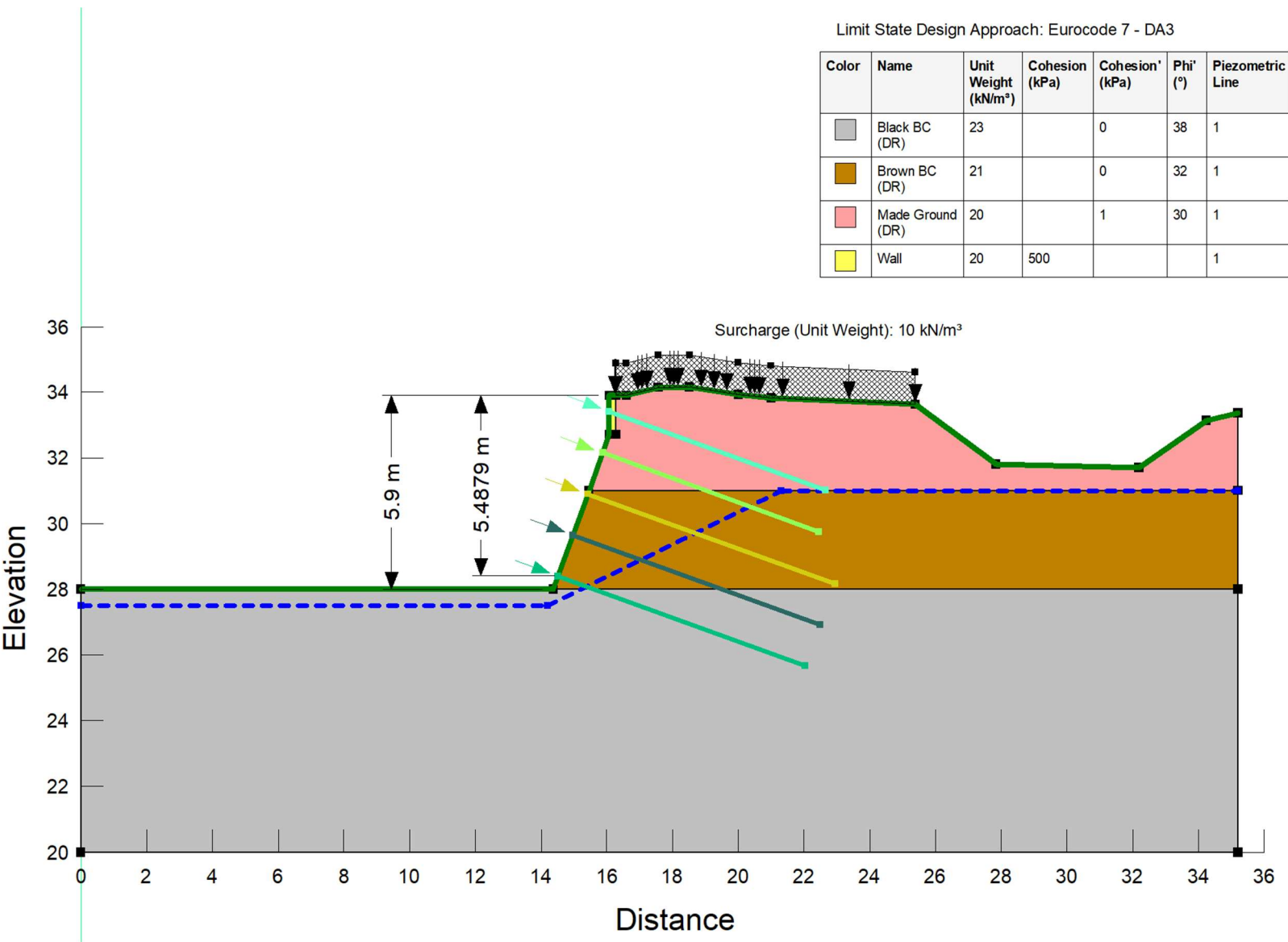


Figure 1: SlopeW Model - Section 1 – Initial Condition (DS-1) - Drained

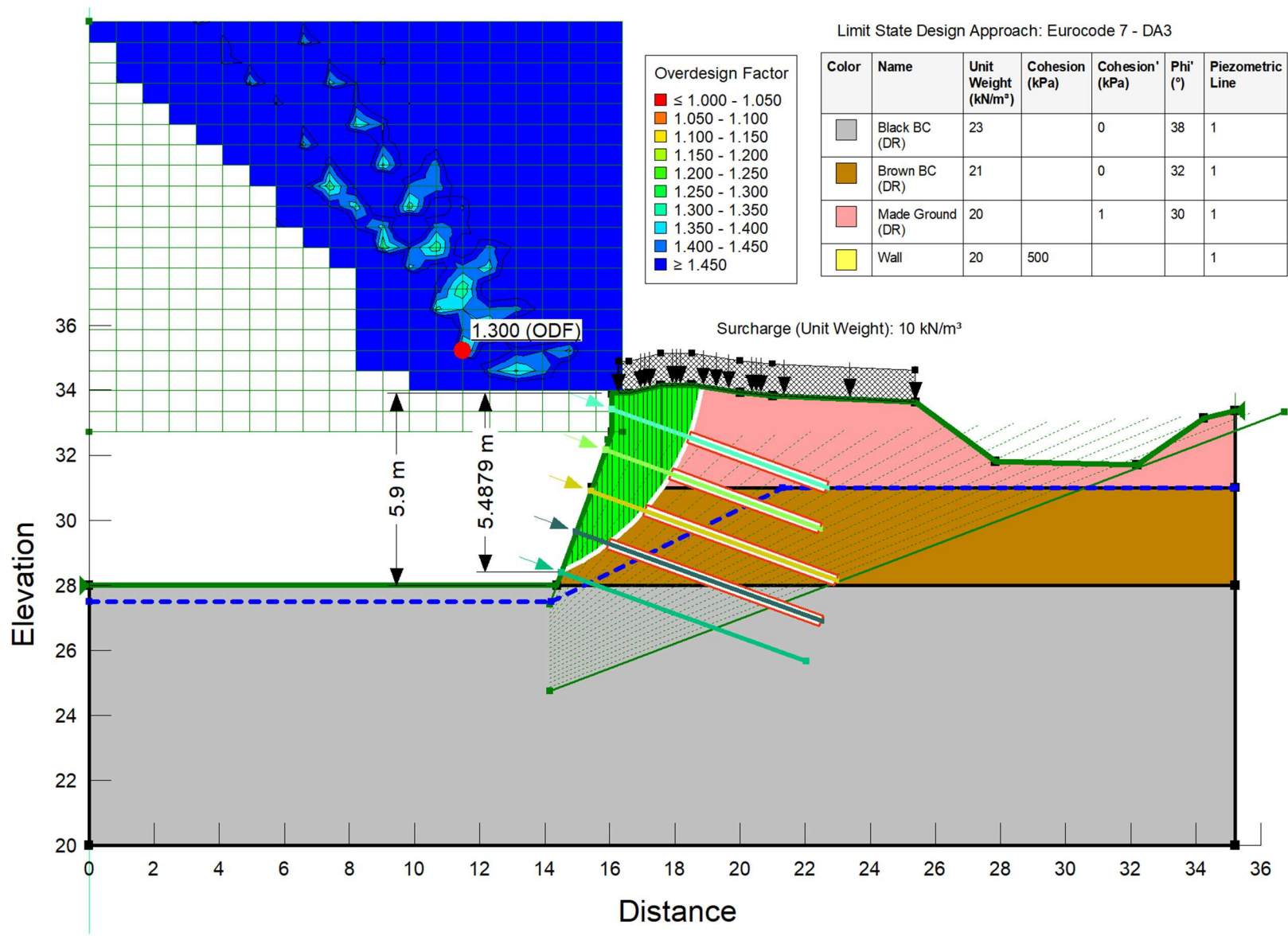


Figure 2: SlopeW Results – Section 1 – Initial Condition (DS-1) - Drained

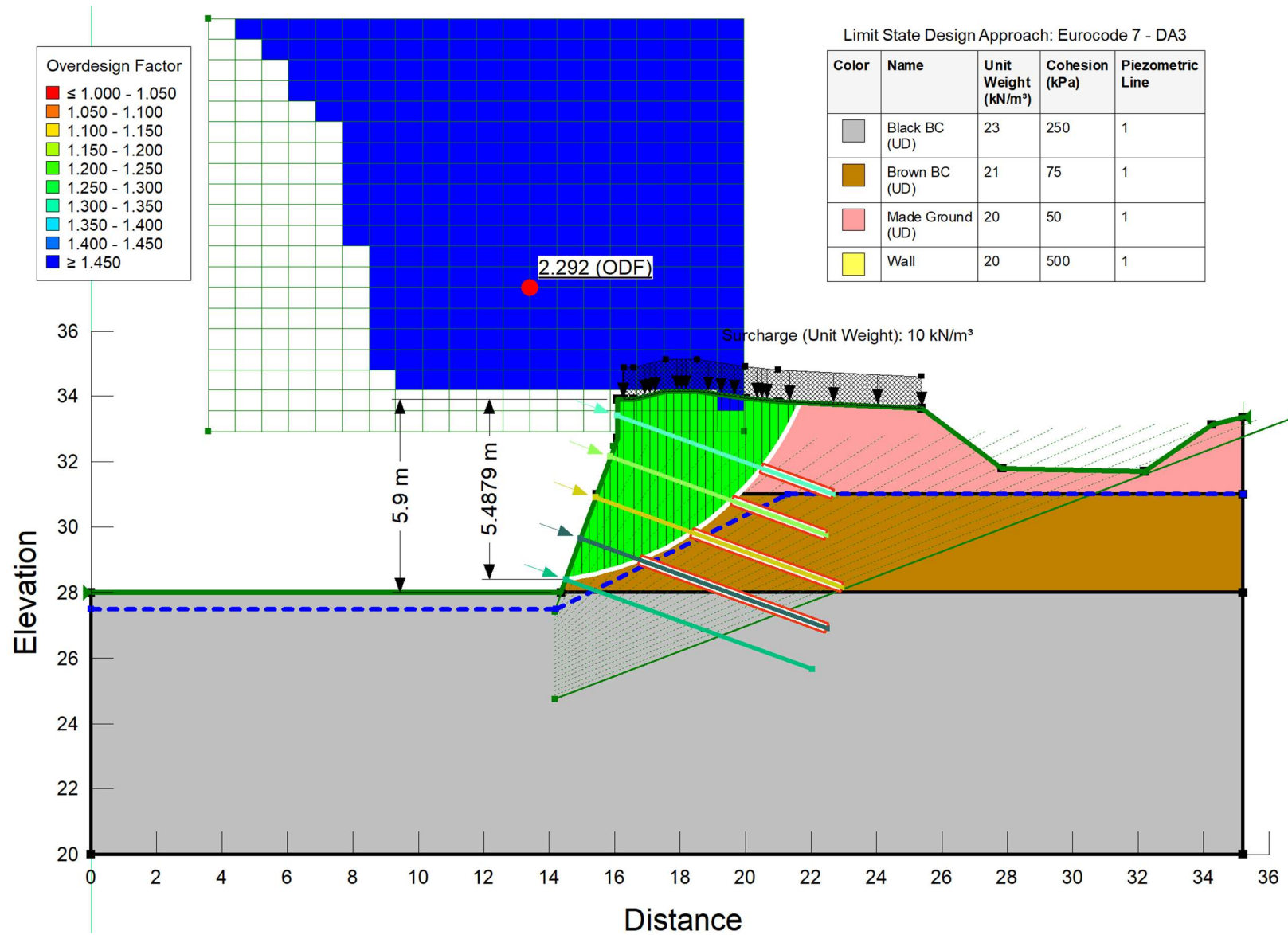


Figure 3: SlopeW Results – Section 1 – Initial Condition (DS-1) - Undrained

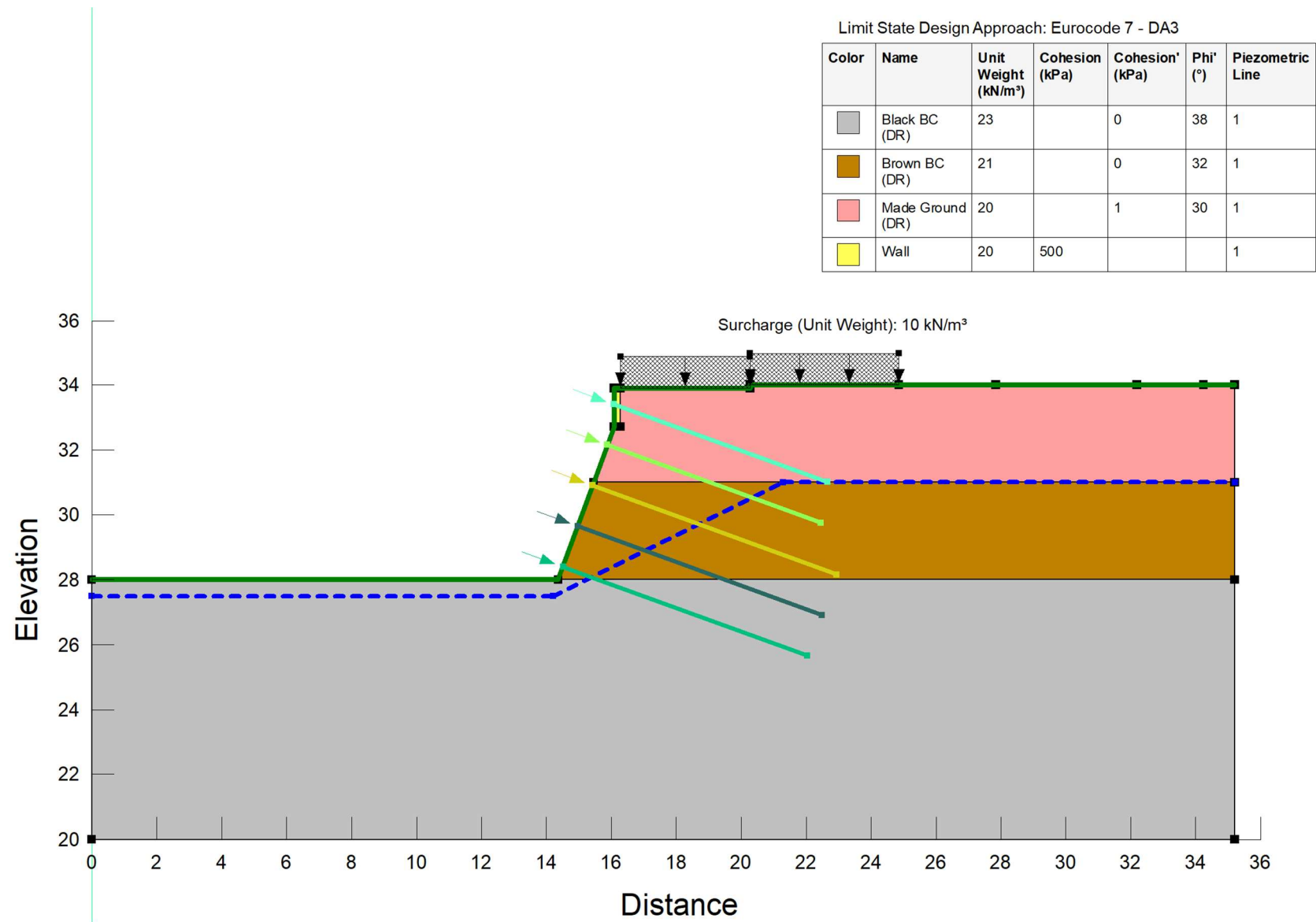


Figure 4: SlopeW Model – Section 1 – Permanent Condition (DS-2) – Drained

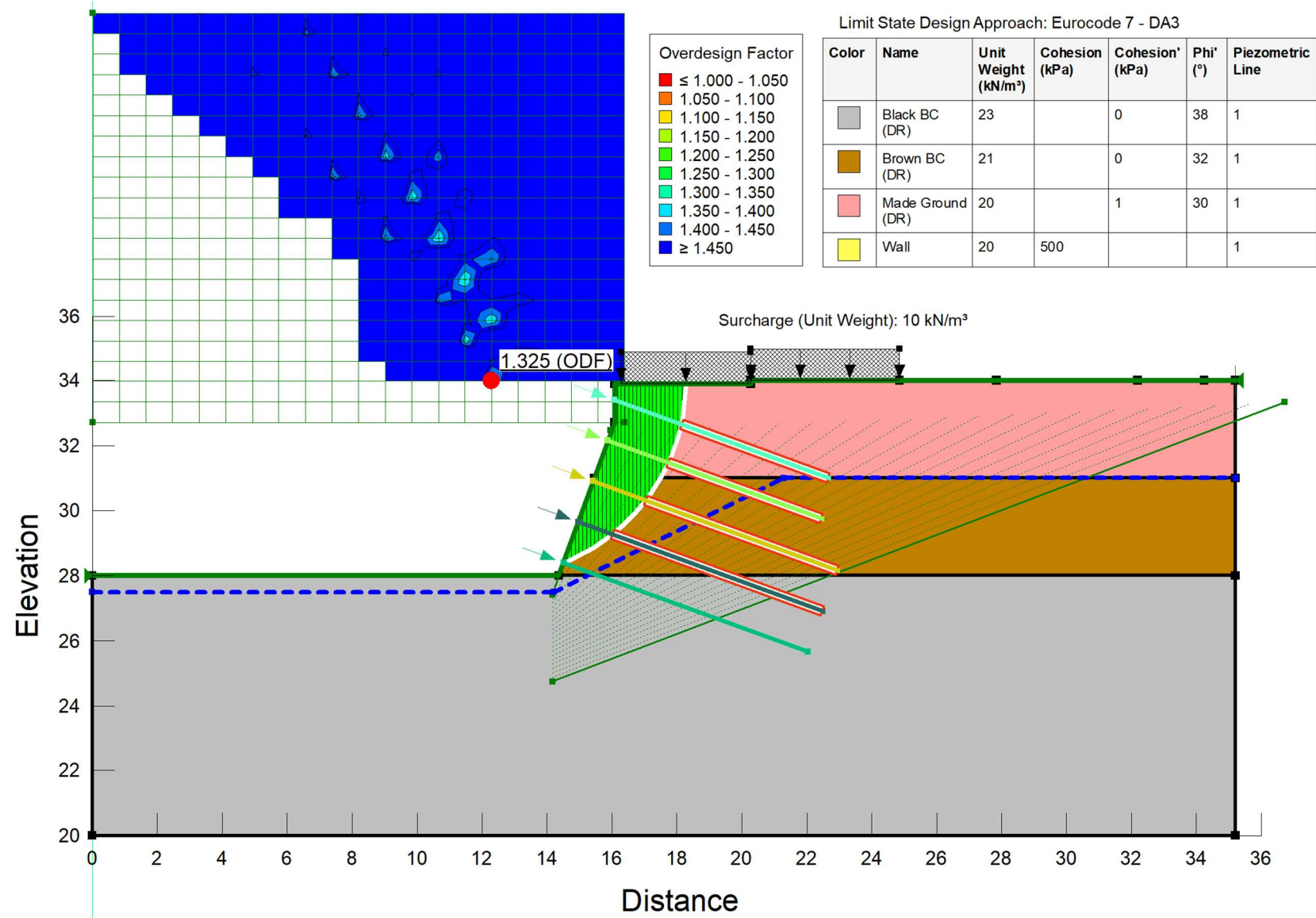


Figure 5: SlopeW Results – Section 1 – Permanent Condition (DS-2) – Drained

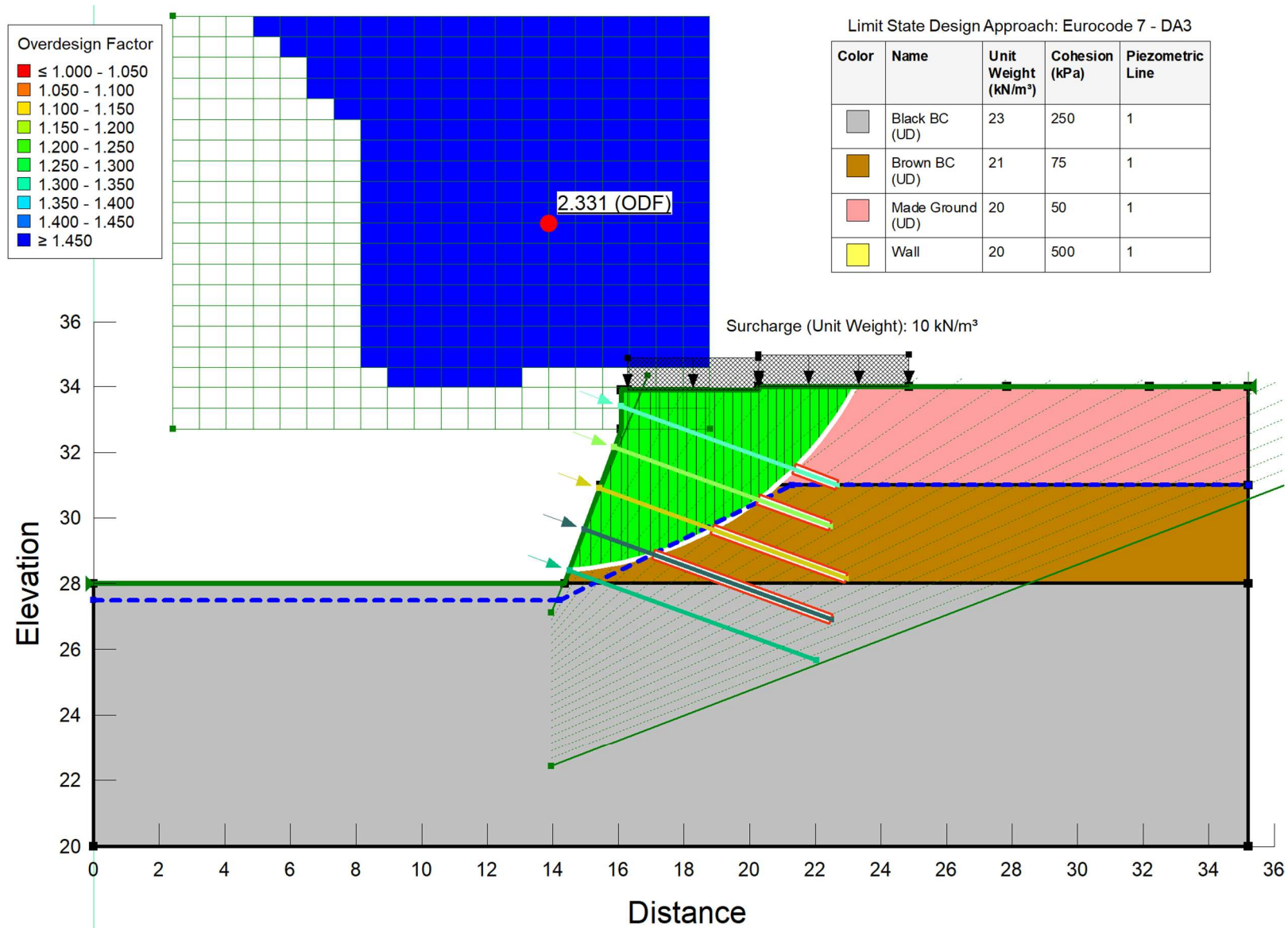


Figure 6: SlopeW Results – Section 1 – Permanent Condition (DS-2) – Undrained

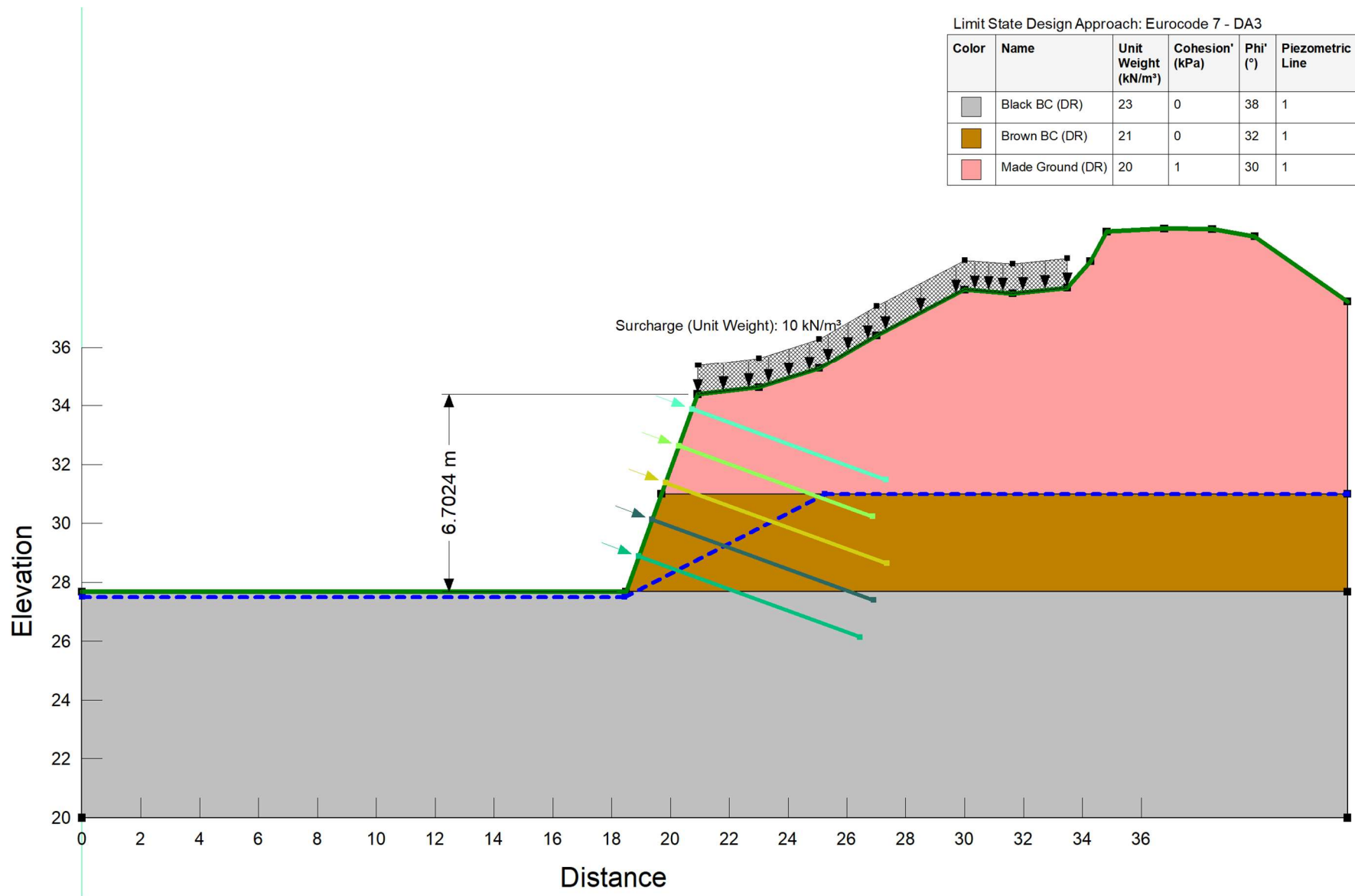


Figure 7: SlopeW Model – Section 2 – Initial Condition (DS-3) – Drained

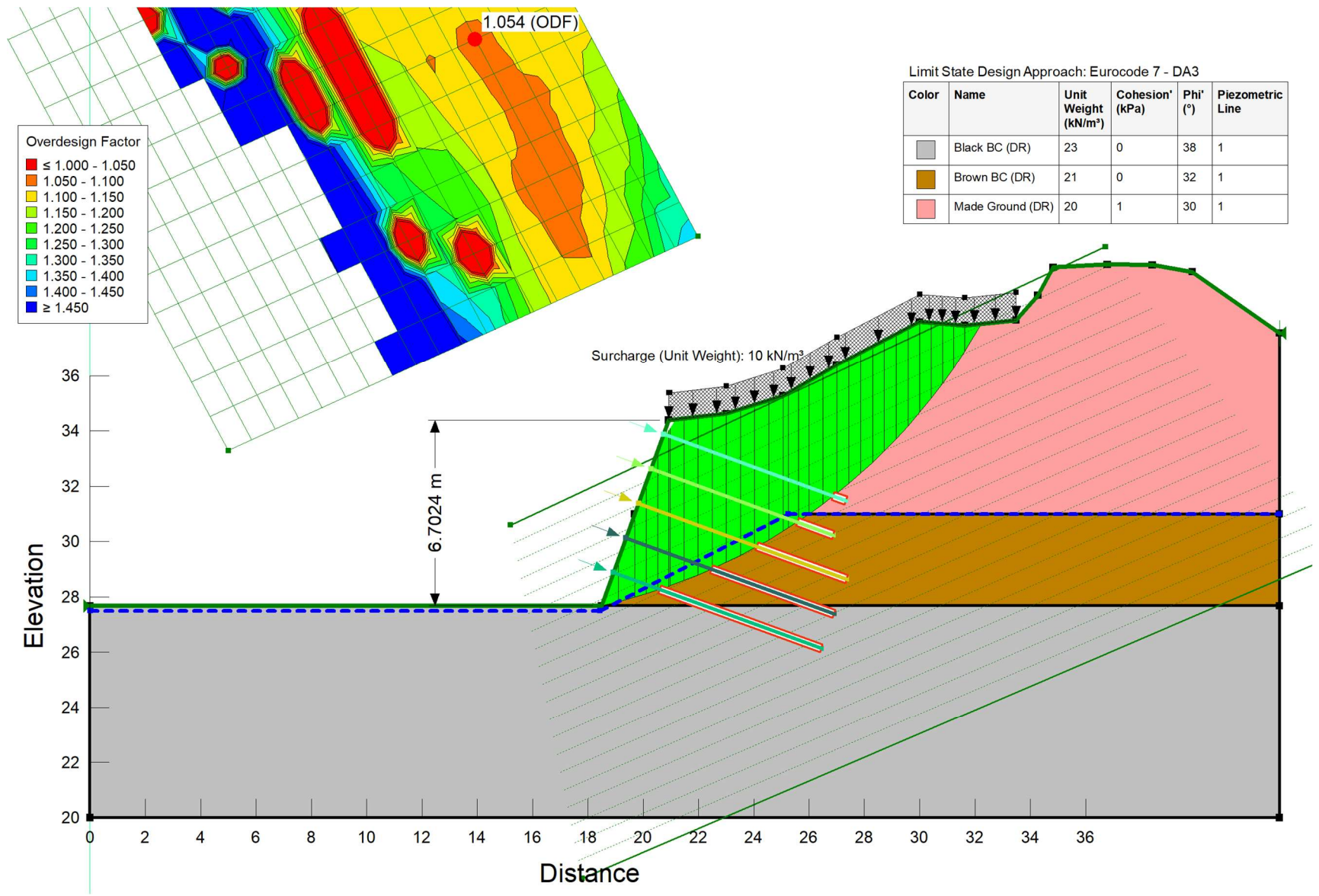


Figure 8: SlopeW Results – Section 2 – Initial Condition (DS-3) – Drained

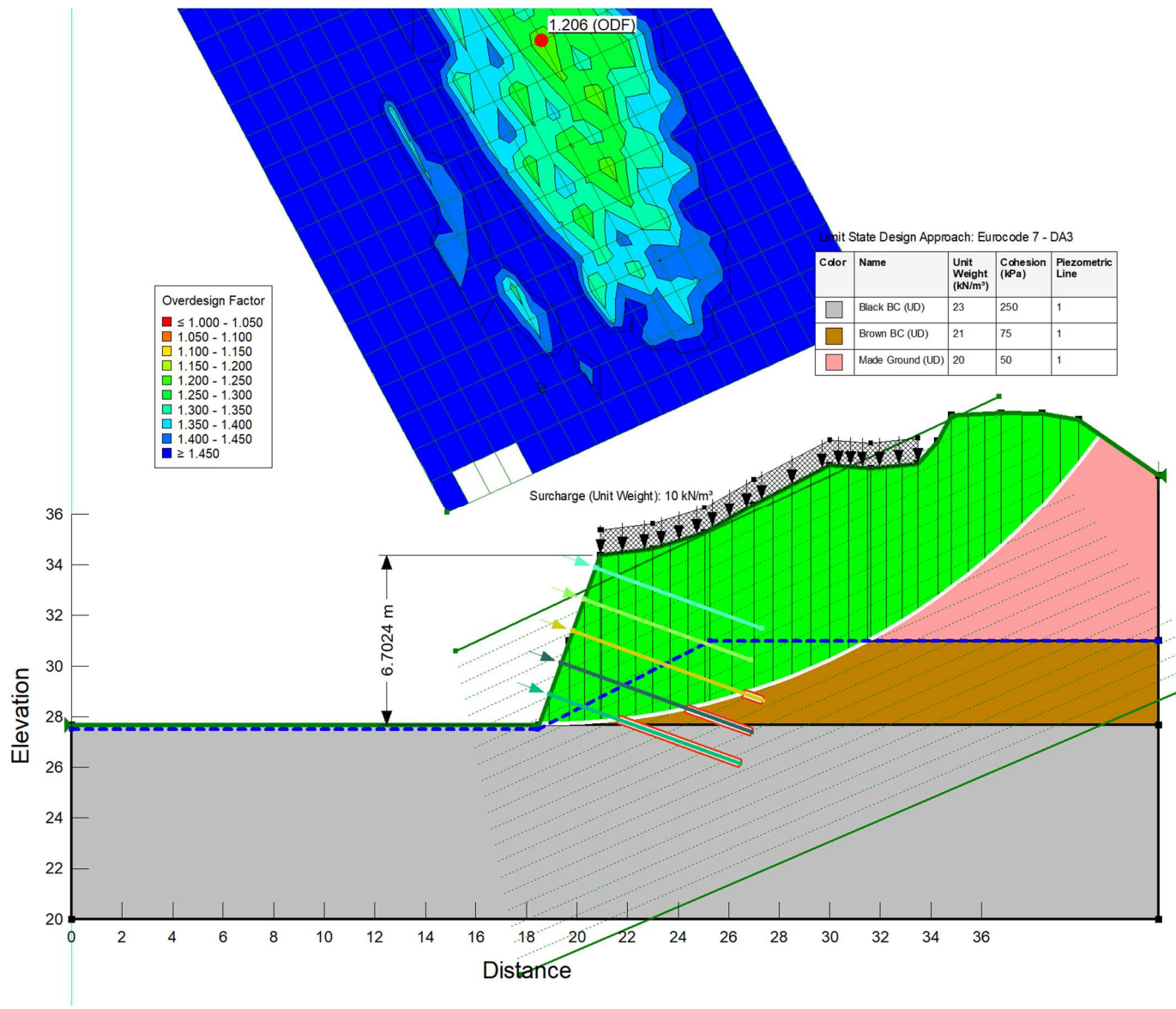


Figure 9: SlopeW Results – Section 2 – Initial Condition (DS-3) – Undrained

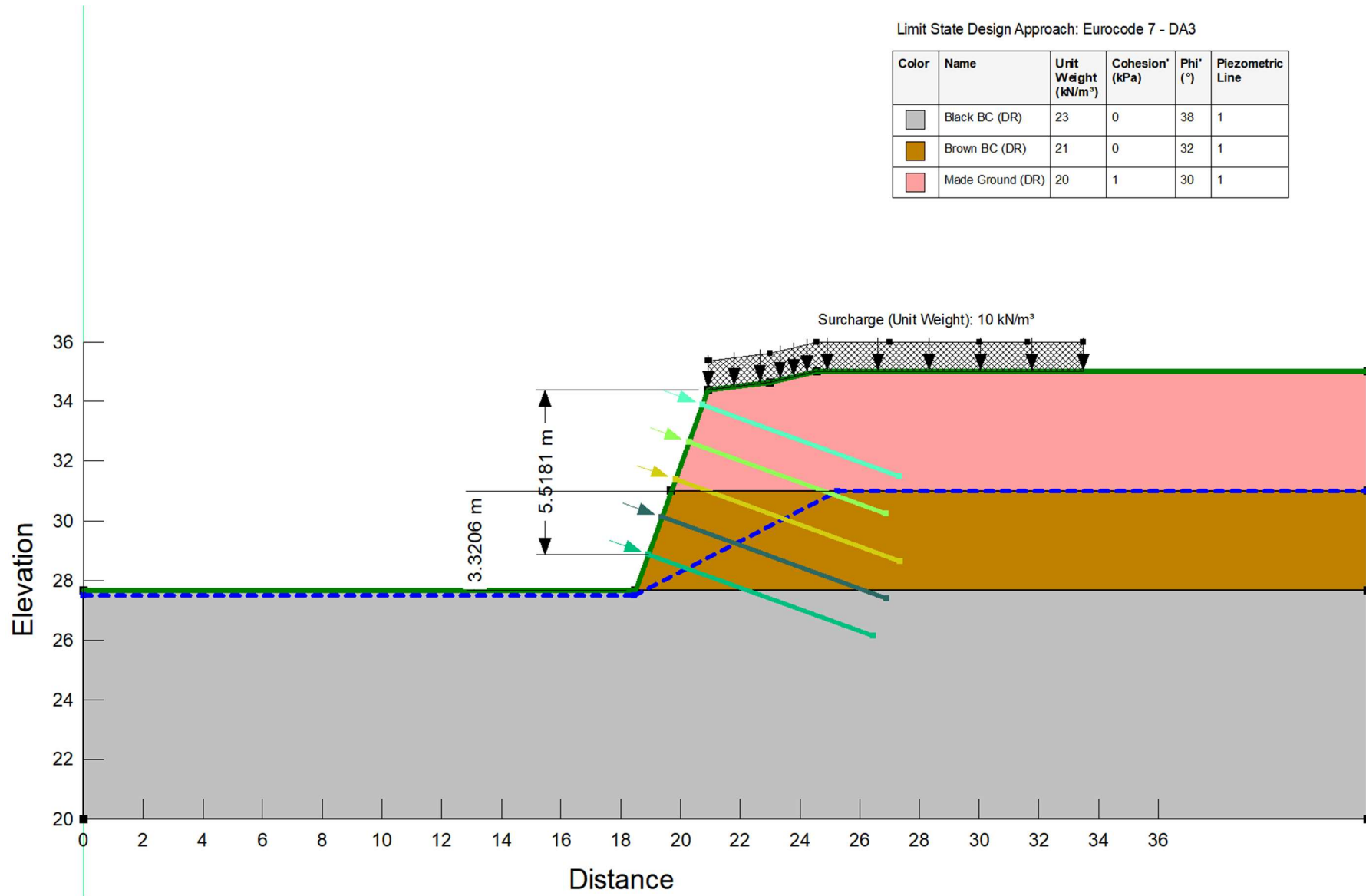


Figure 10: SlopeW Model – Section 2 – Permanent Condition (DS-4) – Drained

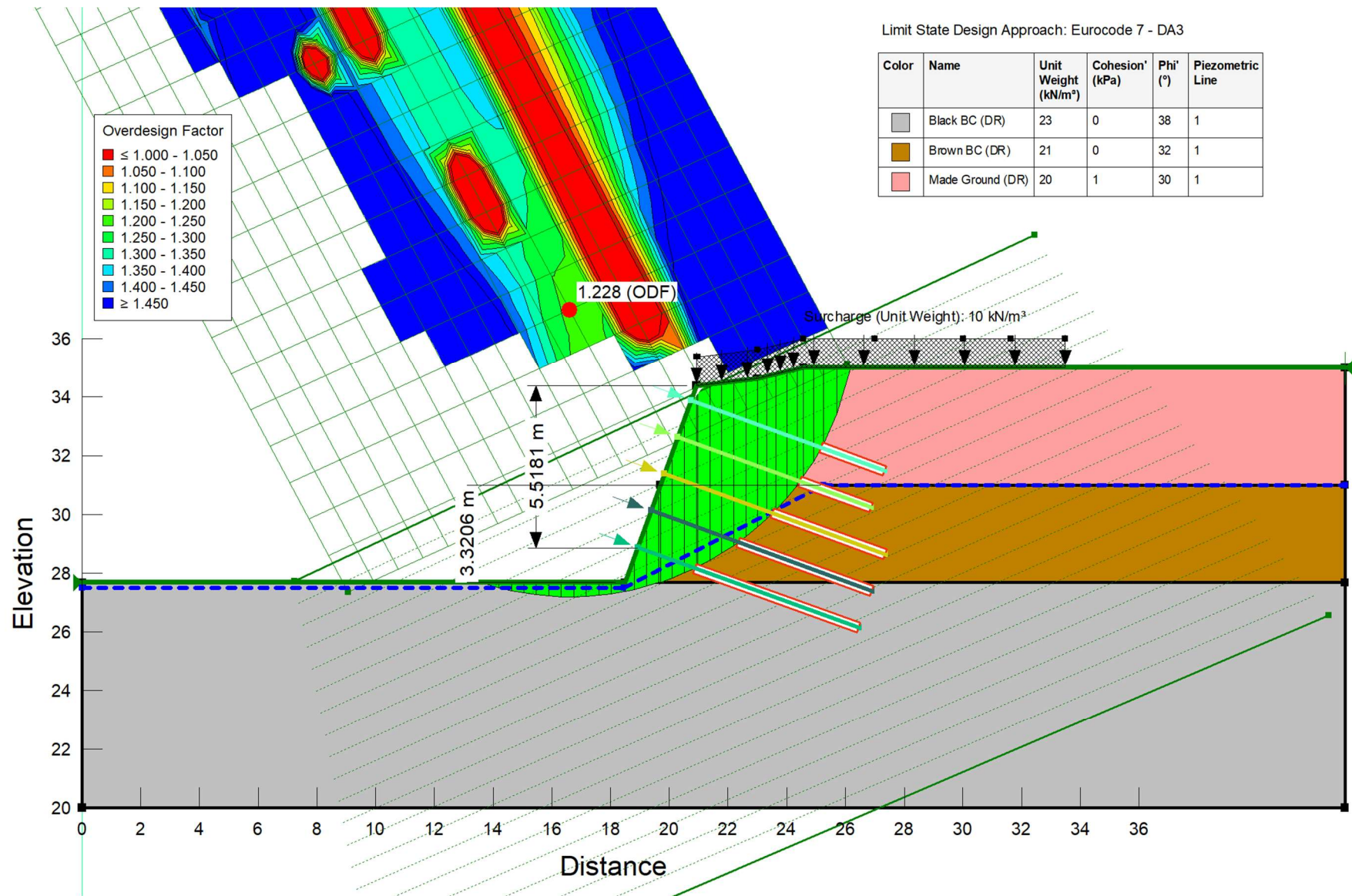


Figure 11: SlopeW Results – Section 2 – Permanent Condition (DS-4) – Drained

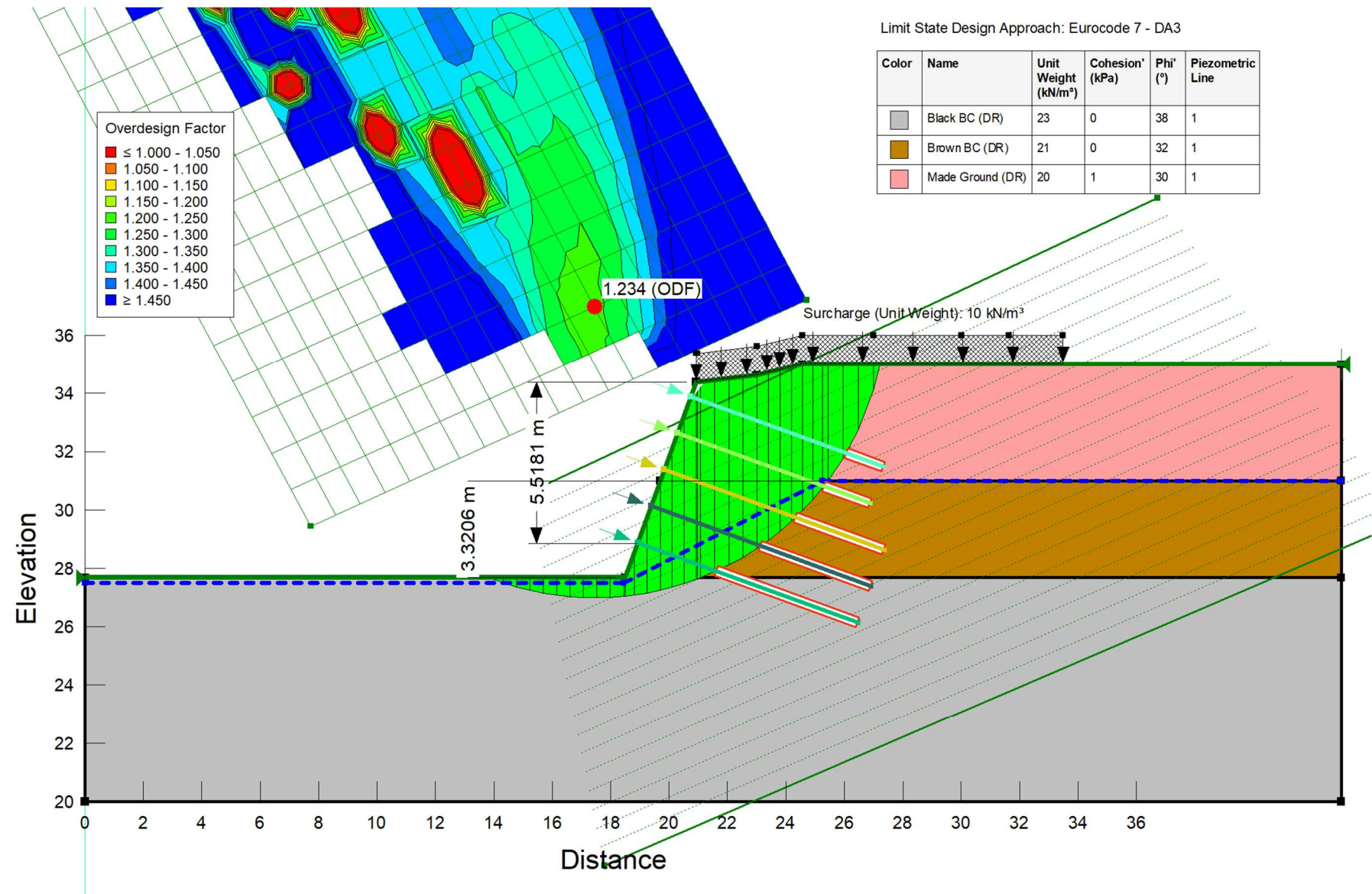
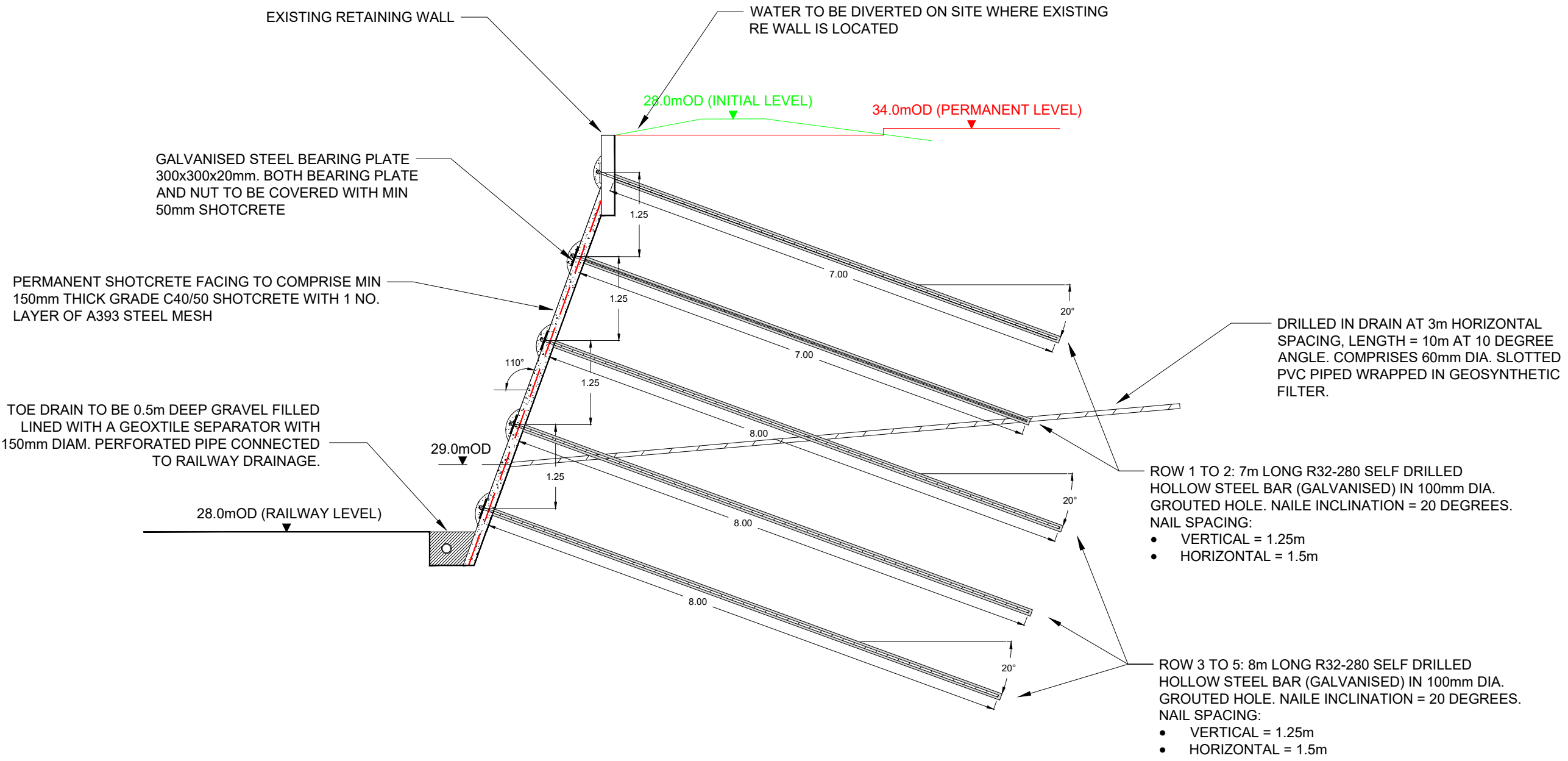


Figure 12: SlopeW Results – Section 2 – Permanent Condition (DS-4) – Undrained

APPENDIX F
CONSTRUCTION DRAWINGS



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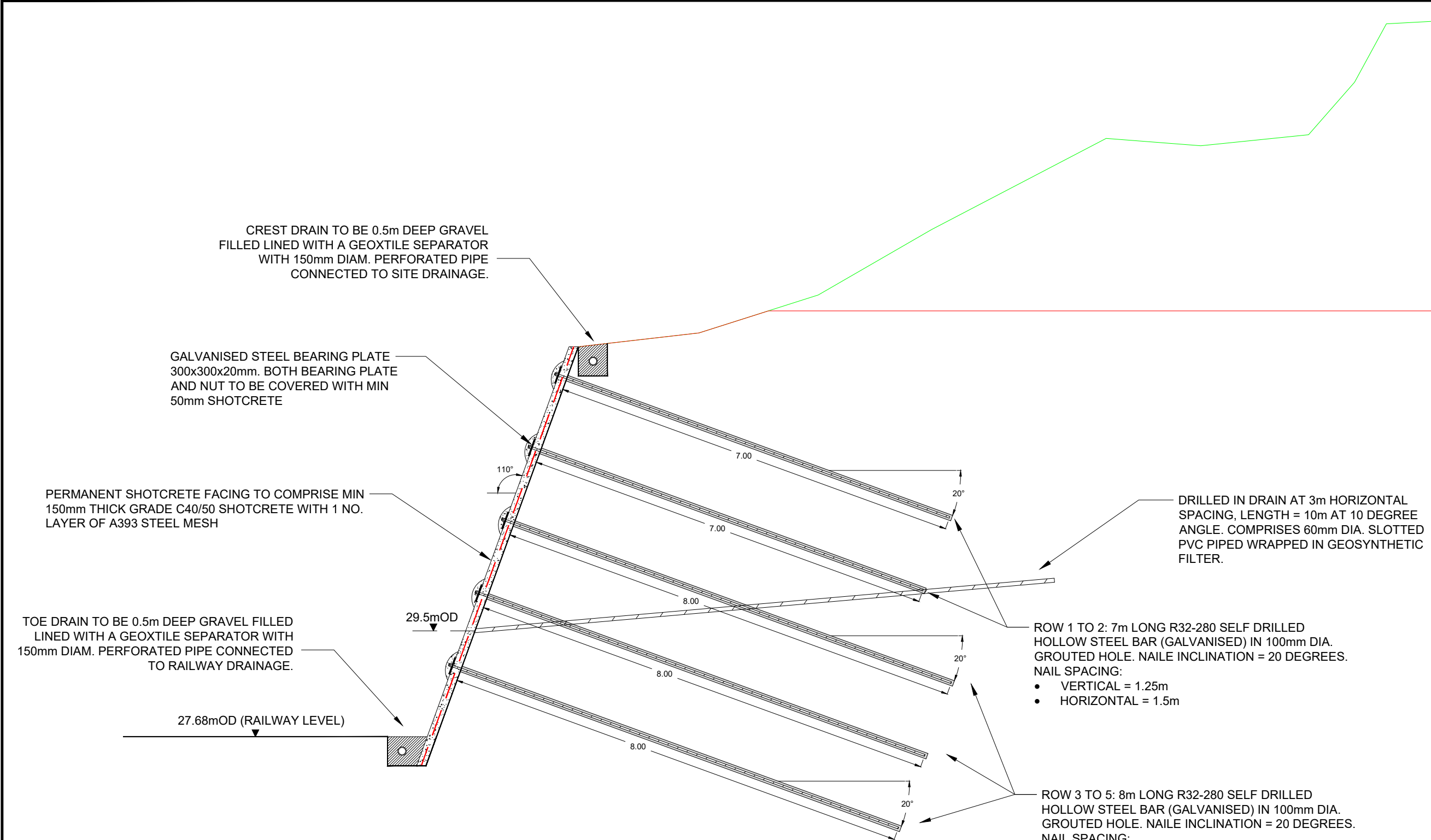
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CLIENT: LOHAN & DONNELLY

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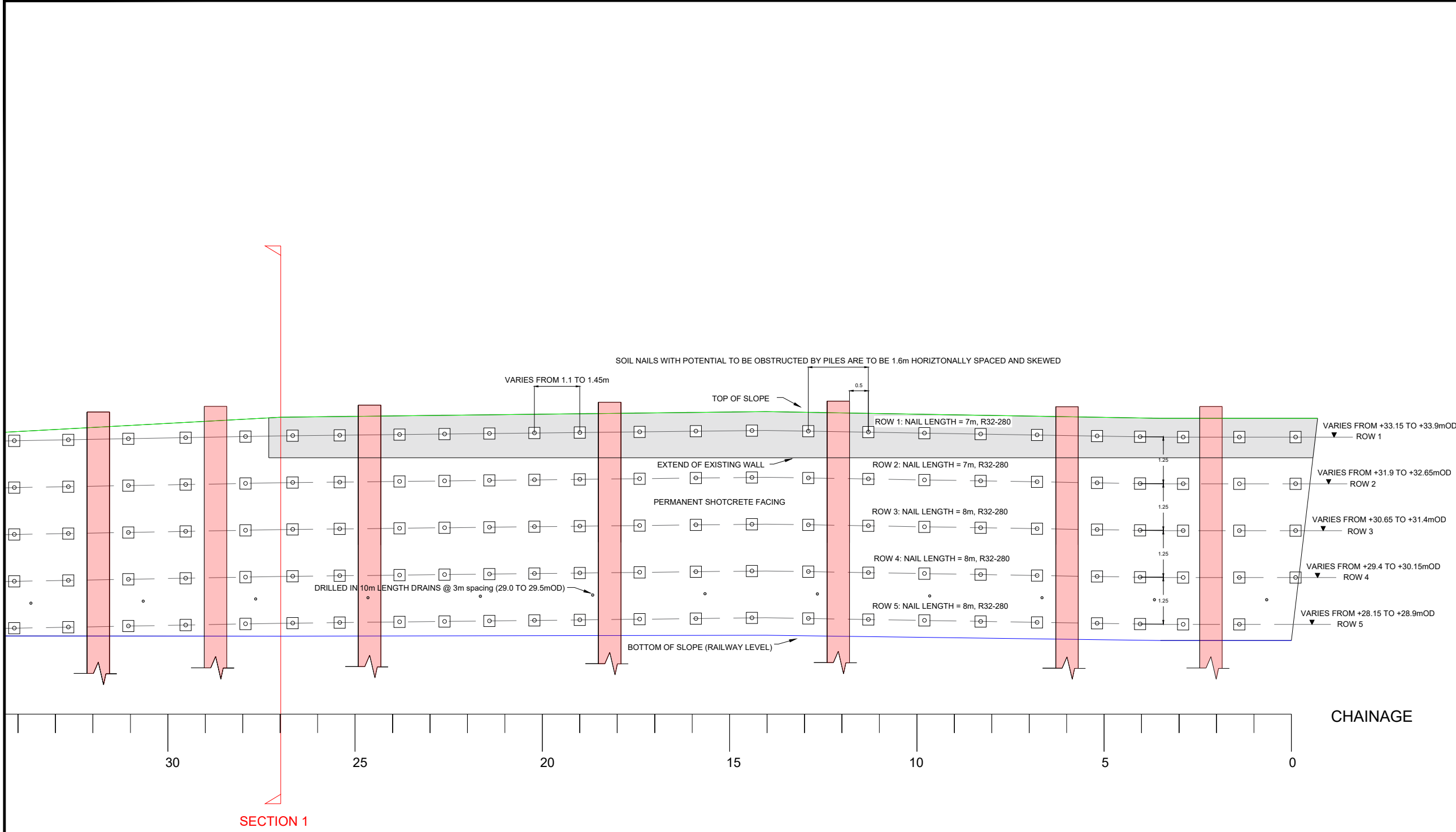
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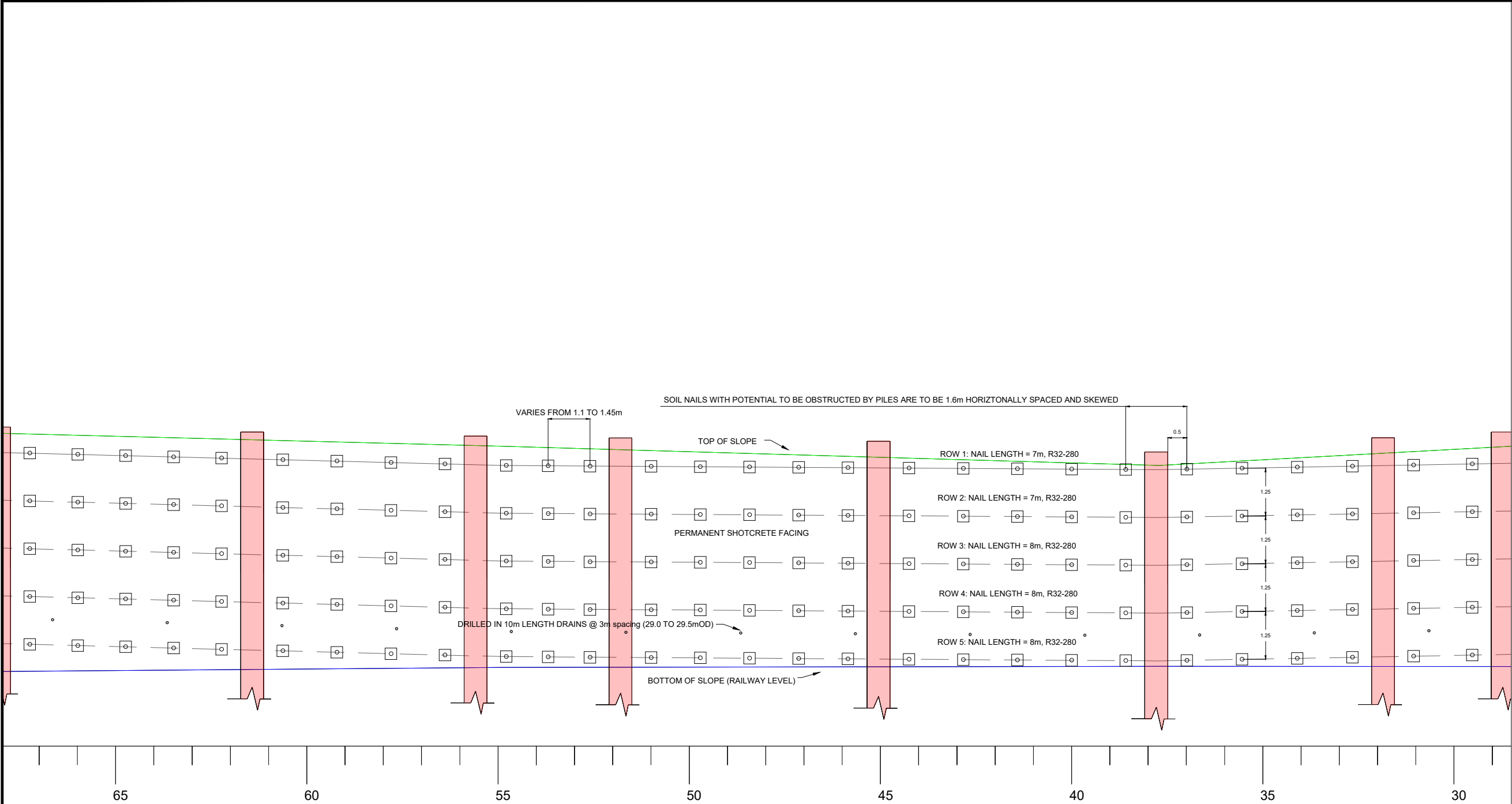
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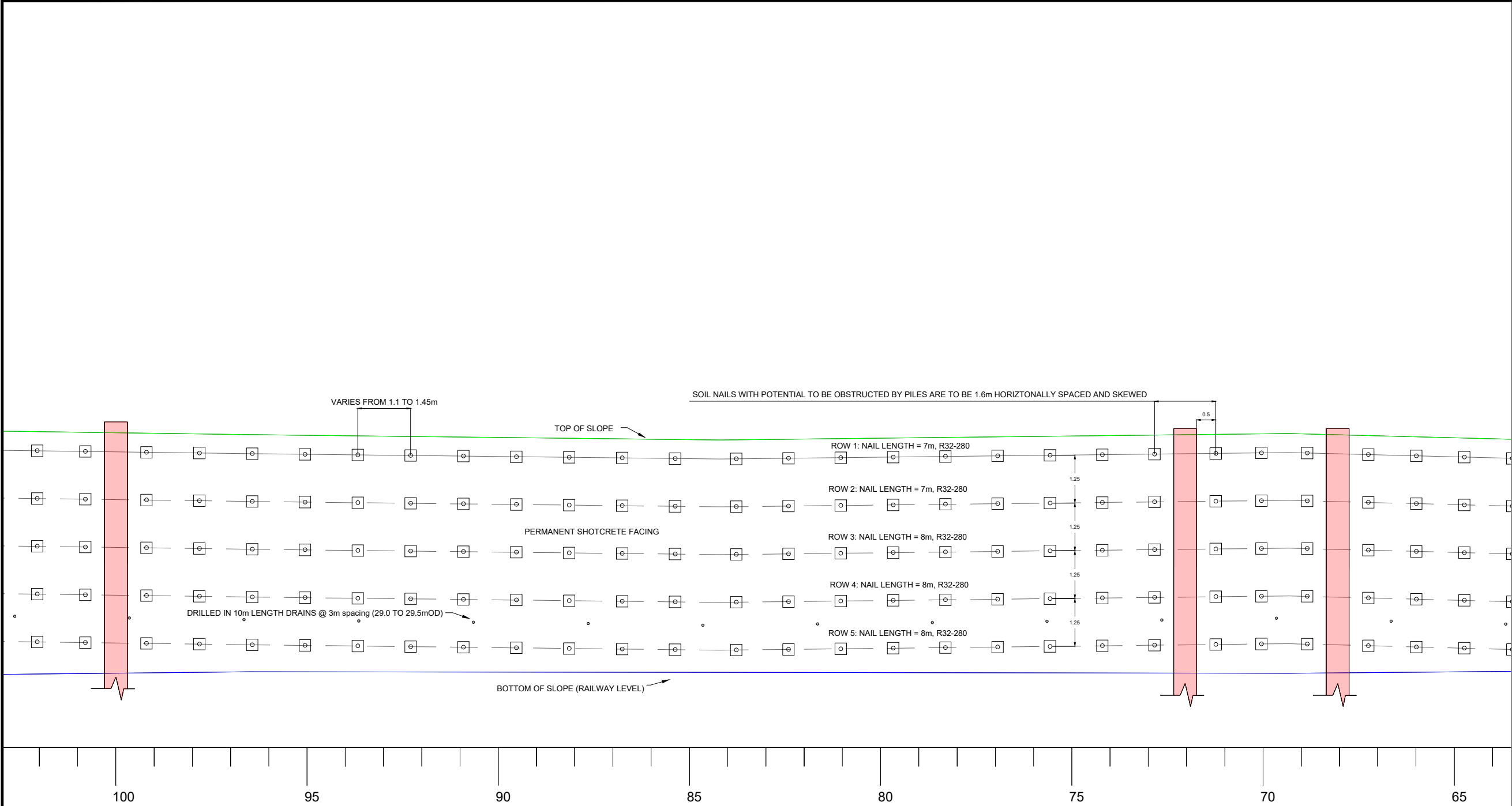
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CLIENT:
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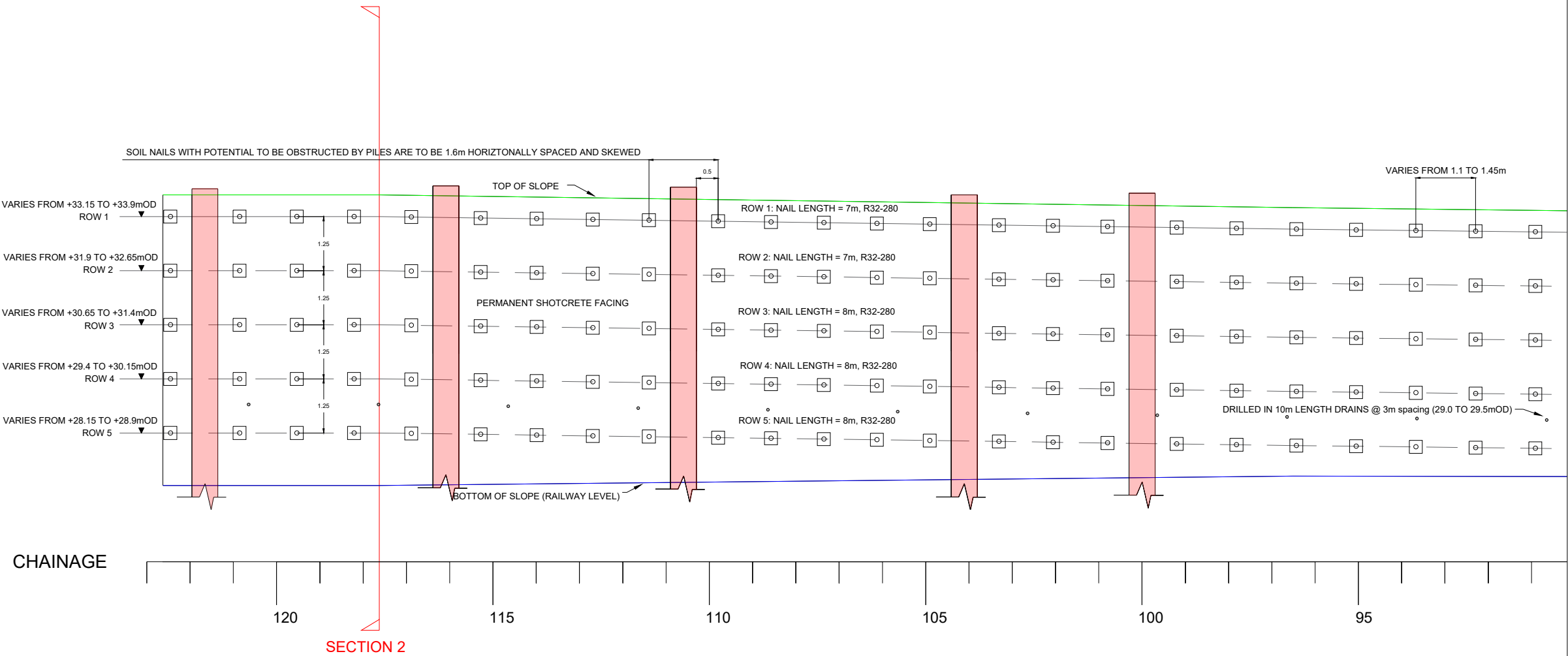
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ELEVATION DRAWING - SHEET 4 OF 4

SCALE: DNS (A3) DRAWN: MP

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PLAN EXTENT OF SOIL NAILING WORK

Section I (S2)

Section H

Section G

Section F

Section E

Section D

Section C (S1)

Section B

Section A

0		MP	DRG	DRG

REV	DESCRIPTION	DRN	CHKD	APPR	DATE

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CLIENT: LOHAN & DONNELLY

DRAWING TITLE: PLAN DRAWING

SCALE: DNS (A3) DRAWN: MP

DATE: 17/04/26	JOB NO: 26-135	DRAWING NO: 26-135-D07	REV: 0
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APPENDIX G

DESIGNER'S RISK ASSESSMENT

26-135-RR01-Rev 0 Designer's Risk Assessment for Soil Nail Reinforced Slope at Faussagh Avenue, Cabra

DESIGNERS RISK ASSESSMENT – FOR SOIL NAIL REINFORCED SLOPE AT RAILWAY							
	Hazard	Cause	Potential Impact	Initial Risk	Mitigation steps taken in design	Construction Control	Residual Risk
1	Strike of permanent CFA bearing pile by soil nail causing damage to building	1. Incorrect set out. 2. Excessive deviation of soil nail during installation 3. CFA piles not correctly located	1. Damage to building 2. Cessation of works.	High	1. A sacrificial “dummy” piles is to be installed on the railway side of each bearing pile that is within the plan extent of the soil nailing. This ensures that the piles supporting the building can't be damaged by the soil nailing works. 2. The as-built location of the CFA piles is to be recorded and clearly marked on the slope face. 3. The soil nails are to be set out to allow for minimum 0.5m offset from the CFA piles.	1.The mitigation measures as given in the AGL GDR 26-135-R01 are to be implemented	Low
2	Collapse of existing retaining wall at crest of slope on to the railway	1.Soil nailing works destabilises retaining wall 2.Wall in poor condition 3.Stockpiling of material behind wall 4.Excessive plant loading on wall	1. Death or injury to site personnel working at the base of the slope. 2. Loss and cessation of works.	Medium	1.The existing retaining wall is to be stabilised by a row of soil nails to ensure that a future collapse on to the railway cannot occur. 2.The shotcrete facing may also extend over the wall depending on its structural condition.	1.The Contractor shall follow the Construction Sequence set out in the GDR, in particular the limits on excavation of unsupported slope faces below the soil nails. 2.A temporary works design is required for construction of access track and working platform on the slope and for the excavation of slip material. 3.No materials or plant are to be stored at the top of the slope during construction. 4.Daily visual inspections of the slope and ground surface	Low

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						shall be carried out (i.e., observe of evidence of tension cracks or other signs of slope movement), especially following periods of heavy rainfall. 5.The slope should be observed during drilling for any signs of instability due to drilling the soil nails. 6.Construction works carried out during or after periods of prolonged heavy rainfall should be minimised. 7.Cut slope angles are not to exceed that given in the drawings.	
3	Collapse of the slopes during construction	1. Destabilising of slopes due to excavation, plant loading or stockpiling of materials or spoil	1. Death or injury to site personnel working at the base of the slope. 2. Loss and cessation of works. 3. Disruption of railway	High	1.A staged construction sequence has been provided to ensure temporary stability of slopes	1.The soil nails and facing are to be constructed as given in the GDR and as shown on the construction drawings. 2.If ground conditions encountered during drilling the soil nails differs to that assumed in the design then AGL is to be informed immediately.	Low
4	Collapse of slope following completion of the works	1. Ground conditions differ from those indicated by the ground investigation. 2. Ground levels are higher than those used in the design.	1.Death or injury to persons. 2.Loss and cessation of works. 3.Disruption of railway	Medium	1. The Soil Nail slope is designed in accordance with IS EN 1997-1:2005 and BS 8006-2:2011 and for a Category 2 structure 2. The ground model for the design is based on ground investigation information.	1. The soil nails and facing are to be constructed as given in the GDR and as shown on the construction drawings. 2.If ground conditions encountered during drilling the soil nails differs to that assumed in the design then	Low

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		3. Surcharge loads applied exceed those assumed in the design.			3. Moderately conservative soil parameters have been used in the design for its drained and undrained shear strength. 4. Ground levels used for the design have been obtained from the topographical survey provided to AGL to develop the ground profiles. 5. Sacrificial soil nail tests and production soil nail tests are to be carried out to verify the design 6. A minimum general surcharge load of 10kPa at the slope crest has been assumed in the design. 7. AGL shall be informed if excessive overbreak of the slope face occurs.	AGL is to be informed immediately. 3. Soil nail testing is to be carried out as set out in the GDR.	
5	Fall from height	1. Working from the top of the slope without fall protection. 2. Access by site personnel/public to the top of the slope.	1. Death or injury to site personnel. 2. Loss and cessation of works.	High	1. Access to the top of the slope shall be restricted to the general public throughout the construction works. 2. The soil nail slope shall be constructed in stages from the top to the base in accordance with the Construction Sequence set out in the GDR and on the construction drawings.	1. Access to the top of the slope shall be restricted to the general public throughout the construction works. 2. Appropriate fall protection measures are to be implemented to allow work to be carried out safely from height on the slope, if necessary (e.g., scaffolding, personal restraints). 3. Contractor shall submit a method statement detailing the construction methods proposed for the works which shall be	Low

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						approved by AGL prior to commencing the works. 4. Contractor shall carry out the works in accordance with the Method Statement and any deviations shall be approved in writing by AGL.	
6	Underground/Overground services	1. Striking overground services with plant 2. Striking underground services during soil nail installation.	1. Death or injury to site personnel 2. Loss and cessation of works.	High	1. Risk identified	1. The Contractor shall clearly identify the location of the services prior to commencing the construction works. 2. Checks for any other unidentified services shall be carried out prior to commencing construction. 3. The Contractor shall ensure that the location of the services and the requirements for the safe construction of the soil nail slope to avoid these services are communicated to all site personnel. 4. If services are identified at the location of the proposed soil nails then AGL shall be informed to relocate the soil nails.	Medium
7	Collapse of soil nail hole before grouting	1. Presence loose sand or gravel	1. Void in the ground leading to ground settlement. 2. Non-continuous grout in the soil nail	Medium	1. Self-drill hollow bars are specified, which are suitable for loose granular soils	1. Following completion of drilling to the full length of the soil nail the final length shall be recorded.	Low

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			giving low soil nail pull-out capacity and leading to long term slope instability.			2. Driller shall observe flush returns from drilling fluid for indications of excessive soil extraction during drilling the soil nails. 3. Collapsed bores are to be re-drilled and all voids filled with grout.	
8	Soil Nail Hole not fully filled with grout.	Not enough grout pumped into soil.	1. Void in the ground leading to ground settlement. 2. Non-continuous grout in the soil nail giving low soil nail pull-out capacity and leading to long term slope instability.	Medium	1. The filling of any voids below the nail bearing plate is identified in the construction sequence	1. Contractor shall ensure that the hole is fully filled with grout by allowing the grout to overflow onto the slope face. 2. Any voids below the bearing plate shall be filled with thick grout or a sand & grout mortar prior to installation of the bearing plates.	Low
9	Soil nails do not achieve the required pull-out resistance required by the design.	1. Ground conditions differ from those indicated by the ground investigation.	1. Low soil nail pull capacity and leading to long term slope instability.	Medium	1. Conservative soil parameters have been used in the design. 2. Sacrificial tests (2 No.) have been specified to be carried out in the made in advance of construction of the working nails. 3. Production tests (2%) have been specified to be carried out on working nails to verify that the required structural resistance has been achieved on the working soil nails.	1. Soil nail testing is to be carried out as specified. The results shall be provided to AGL as soon as possible to verify the design. 2. Additional soil nails may be required where the design soil nail pull-out resistance has not been achieved.	Low
10	Shallow slope failures between soil nails.	1. Insufficient support of the soil between nails.	1. Degradation of slope	Medium	1. A flexible facing comprising a shotcrete facing. 2. Where required temporary shotcrete and steel mesh is to be placed on the slope face to	1. The facing shall be installed as set out in the construction drawings. 2. Existing obstacles and vegetation are to be removed to	Low

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					stabilise the face and allow safe excavation.	ensure a regulated and even slope face prior to installing the wire mesh facing.	
11	Obstructions to drilling	1. Tree roots encountered during drilling. 2. Cobbles and boulders in glacial till	1. Shorter soil nail length resulting in insufficient bond length in soil nail leading to long term slope instability. 2. Prevention of nail installation due to surface obstruction	Medium	1. Risk identified	1. 3. Advise AGL of obstruction to soil nails that cannot be removed or encountered during nail installation. Additional soil nail/nails to be offset from construction as agreed with AGL	Low
12	Obstructions causing deflection of soil nail.	1. Soil nail deflecting due to boulder partially covering soil nail hole.	1. The design inclination of the soil nail is not achieved resulting in insufficient reinforcement of slope.	Medium	n/a	1. Contractor shall have equipment available to bore through boulder obstructions, if encountered. 2. The driller shall be aware of any deflections in the soil nail inclination during drilling.	Low
13	Excessive Overbreak on Slope Face.	1. Loose/soft ground. 2. Water seepage from slope face.	1. Excessive Overbreak would leave a gap between the slope face and the steel mesh when placed on the slope, thus, this section of the slope face would be unsupported.	Low	1. AGL have specified maximum cut slope angles for each zone of the soil nailed slope.	1. AGL shall be informed if excessive overbreak occurs on the slope face. 2. Slope cut angles shall not exceed that specified. 3. Temporary shotcrete facing to be used to stabilise the slope face where required and agreed with AGL	Low
14	Surface runoff causing erosion or collapse of the slope face	1. Surface water from surrounding area	1. Erosion of the slope face or local collapse	Medium	1. Risk identified	1. AGL to be advised of any surface water issues. Surface drains or temporary bunds may be required to divert or intercept surface water.	Low