Anglesey Land Holdings

Prosperity Parc, Anglesey

Geotechnical Interpretation and Design Parameters Report

Table of Revisions

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11th September 2024	-	First Issue
7th November 2024	-	Second Issue

Job Title	Geotechnical Interpretation and Design Parameters Report		
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Contents

Executive Summary

The report, prepared for Anglesey Land Holdings, focuses on the geotechnical interpretation and design parameters for the proposed development at the former Anglesey Aluminium Ltd site at Penrhos, near Holyhead. The site spans approximately 800 meters in length and 220 meters in width, located southeast of Holyhead, bordered by multiple roads, a railway line, and natural features such as Penrhos Beach and Penrhos Coastal Park.

The proposed development, named Project Prosperity Parc, is expected to provide up to 238,000 square meters of employment space, featuring facilities like data centres, technology-based infrastructure, research and development centres, and office spaces. Additionally, the development will incorporate landscaping, habitat enhancement, drainage systems, and infrastructure for vehicle, cycling, and pedestrian access. The primary aim of this report is to summarize historical soil investigation results, derive key geotechnical design parameters, and evaluate the performance of typical, traditional shallow foundations to support the planning application and overall feasibility of the proposed development.

The site underwent several rounds of investigations from 2010 to 2015, leading to a collection of various geotechnical and geo-environmental reports. The findings from these investigations provide a solid understanding of the site's general ground conditions and geotechnical characteristics, establishing its suitability for redevelopment. Although the previous data may not comprehensively cover every part of the site, it offers substantial evidence to guide an efficient strategy for developing new buildings. Any potential gaps in the existing data will be addressed through further investigations before the start of construction, ensuring thorough assessment and planning.

Based on a careful analysis of the site, two distinct ground models have been established to support planning objectives: shallow bedrock and deep bedrock. The information gathered from previous studies has been instrumental in defining appropriate design parameters for the development.

Bearing capacity and settlement analyses were conducted using foundation dimensions typical for industrial buildings, specifically 2m x 2m and 4m x 4m, with a foundation depth of 1.5 meters. Standard loading conditions were assumed, with 150 kPa applied to the foundations and an additional 50 kPa slab loading. The analyses demonstrated that the site is suitable for the intended redevelopment and that shallow pad foundations and ground-bearing floor slabs can adequately support the proposed structures.

Furthermore, the Geotechnical Remediation Strategy indicated no significant obstacles that would hinder the redevelopment of the site. The combination of historical data analysis, geotechnical evaluations, and remediation strategies provides confidence in the feasibility of constructing new industrial facilities on the site. With suitable design considerations and further investigations as required, the project is expected to proceed successfully, meeting the needs of Anglesey Land Holdings while ensuring structural stability and environmental compliance.

1. Introduction

1.1 Site Location

The proposed development will be constructed within the bounds of the former Anglesey Aluminium Ltd's Penrhos works near Holyhead, at National Grid reference 226700 381200. The site, measuring approximately 800 meters by 220 meters.

The Anglesey Aluminium Metals site at Holyhead is southeast of the town, lying between the A5 London Road to the north and the A55 to the south, with a light industrial and retail park to the west. The site extends from the entrance road in the west, across the existing smelter facilities, and towards the southeast. It is surrounded by security fencing and set apart from neighbours by a margin of forest. To the south, the site is bordered by a railway track and the A55, with agricultural land and marshy areas beyond. To the west lies a retail park and the A5153 road. To the north is the A5 London Road, with Penrhos Beach situated 100 meters further north. The southeastern boundary is marked by the Alpoco Aluminum Powder Plant, while the Penrhos Coastal Park and Beddmanarch Bay lie to the northeast (Figure 1).

Figure 1: Site Location Plan

The northern part of the application site is included within the boundary of the Anglesey Area of Outstanding Natural Beauty (AONB). Additionally, the coastline and inland sea to the east and southeast are classified as a Site of Special Scientific Interest (SSSI). The nearest residential properties are located at Penrhos Farm within Penrhos Coastal Park, approximately 200 meters northeast of the site. The site's designation for redevelopment to create new economic investment and jobs, plus it's part inclusion within the AONB presents a range of issues to be considered and balanced to find a sustainable re-use of this key brownfield site.

1.2 Proposed Development

The proposed development is in outline for up to 238,000 sq.m. of new employment development which will be provided in a range of building types and sizes in due course, but at this stage a detailed layout for the site is not fixed. The proposed development will include data centres and technology related uses, including research and development and office space. An Illustrative Masterplan forms part of the application, included below as Figure 2 to demonstrate how the site might be developed in due course.

- Area: 66.20 hectares (163.58 acres)
- Components: The development will feature on-plot and other landscaping and planting, habitat enhancement and creation, drainage, and other infrastructure, including vehicular, cycle, and pedestrian access.
- Usage: Up to 238,000 sqm for Class B1 (business, and research and development) and B8 (data centre) uses, as well as unique uses such as battery energy storage.
- Building Heights: Maximum building heights will be up to 21 meters excluding point features.
- Finished Floor Levels: These will be similar to existing ground levels, approximately 5 to 10 meters above ordnance datum (AOD).

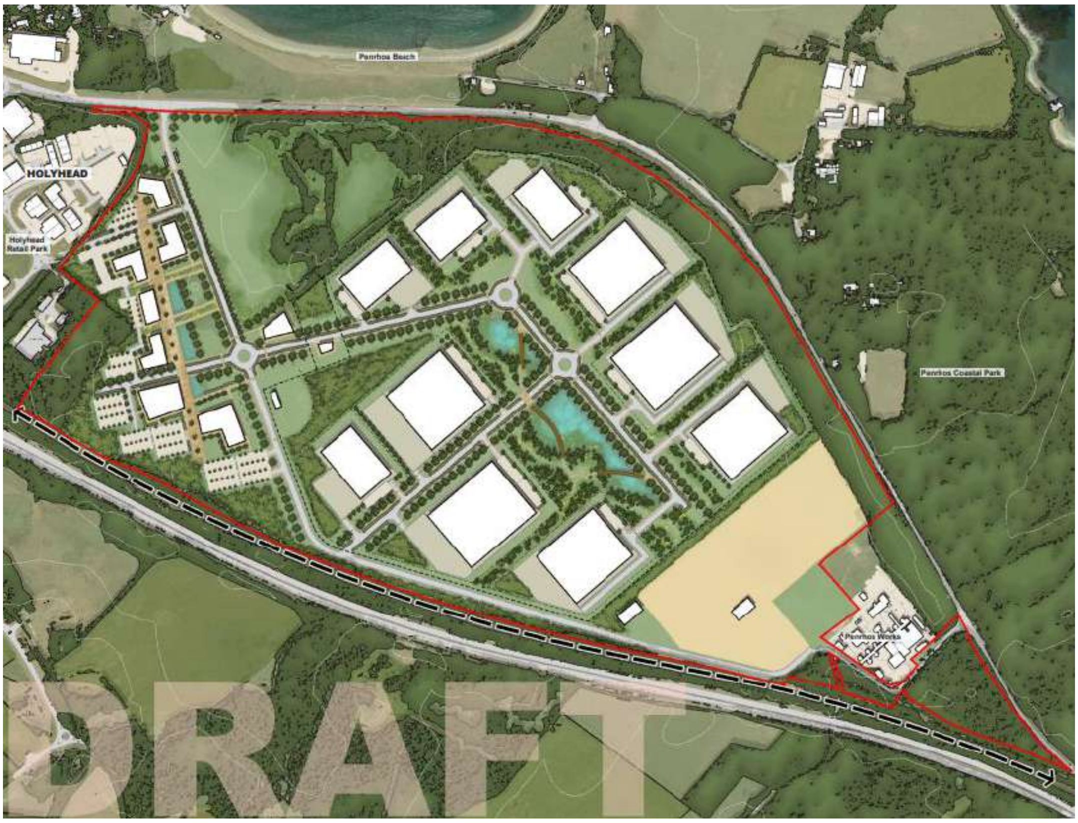

Figure 2: Illustrative Masterplan

1.3 Objectives

The objectives of this report are to:

- Summarize historical soil investigation results and from it derive geotechnical design parameters;
- Use derived design parameters to evaluate the performance of typical size pad foundations;
- Analyze the results to support the planning application and feasibility of the proposed development.

Appendix B of this report (Historical Review and Summary Report) summarises the various past studies in some detail. For information on contamination assessments, readers should refer to the Geo-Environmental Summary Report prepared by Hydrogeo Ltd on behalf of HBGS Ltd.

2. Geotechnical Investigation Results

2.1 Available Information

Multiple geotechnical and geoenvironmental reports related to the considered site are listed below. Among them, the first four reports provide the majority of relevant geotechnical information.

1. 2010 07 Anglesey Aluminium REP, Ground Investigation Report, Mott MacDonald.
2. 2010 07 Report on a Ground Investigation at Anglesey Aluminium REP, Holyhead (Volume One), Soil Engineering.
3. 2010 07 Anglesey Aluminium REP, Geotechnical Design Report, Mott MacDonald.
4. 2012 05 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase A, Golder Associates Ltd.
5. 2008 01 Phase II Environmental Site Investigation of Anglesey Aluminium Metal Ltd, Penrhos Works, Holyhead, Anglesey, Golder Associates Ltd.
6. 2012 04 Anglesey Aluminium Metal Ltd, North Road Benchmark Investigation, Golder Associates Ltd.
7. 2013 02 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase B, Golder Associates Ltd.
8. 2013 06 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Garage Area, Golder Associates.
9. 2014 01 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase C, Golder Associates.
10. 2015 12 Anglesey Aluminium Metal Ltd, Factual Report on Further Intrusive, Investigation of Pitch Tanks and Anode Bake Area, Golder Associates Ltd.

2.2 Results of In Situ Tests

The historical investigation's in-situ tests were limited to SPT (Standard Penetration Test) tests, along with several in-situ CBR (California Bearing Ratio) and soakaway tests. An attempt was made to conduct a shear vane test within a soft silt layer. However, the silt was too stiff to properly seat the test equipment, leading to the abandonment of the test. No other suitable materials were found for this test.

SPT tests

The results from the SPT tests conducted during the investigation at Anglesey Aluminium REP are shown in Figure 3. Some tests recorded minimal penetration, leading to extremely high SPT values (exceeding 150) when linearly interpolated. Therefore, a threshold of 150 counts was set to plot the SPT profile.

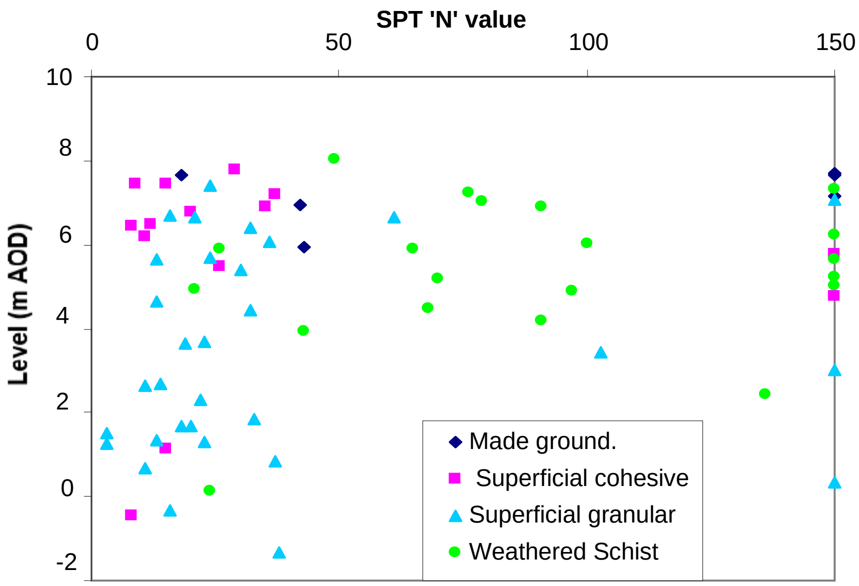

Figure 3: SPT profile through made ground and drift deposits

eIn situ CBR

A total of around 150 CBR (California Bearing Ratio) tests were planned for trial pits along the proposed roadways and hardstandings. Among these, only 4 tests were actually conducted within the superficial granular material at depths between 0.55 and 0.75 meters. The CBR values from these tests ranged from 9.2% to 16%, with an average value of 11.5%. This indicates that the material tested is not suitable for pavement construction, and therefore, capping material will be necessary or other ground improvement measures.

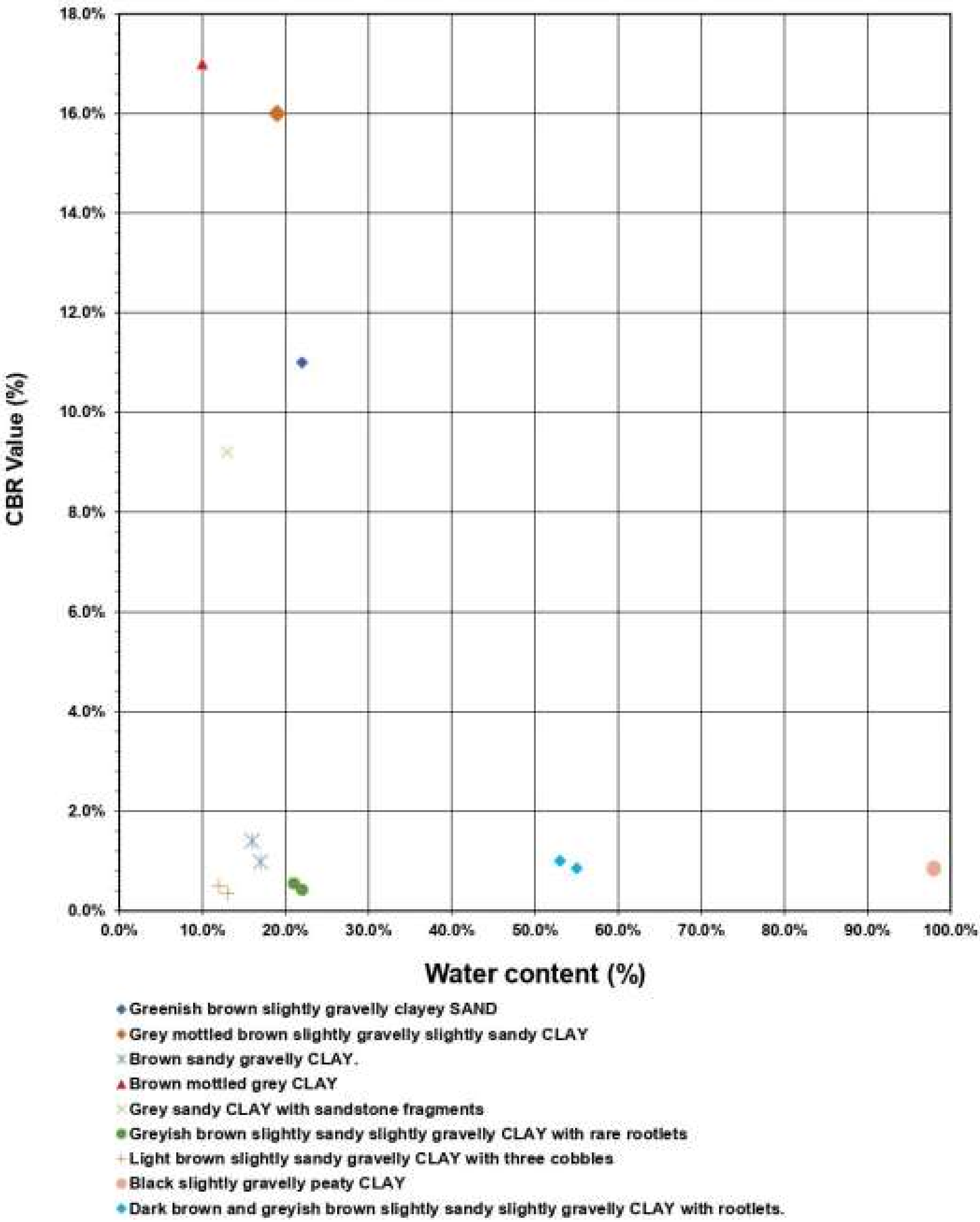

Figure 4: Results of Filed CBR tests

Soakaway test

Five soakaway tests were conducted across the site. Three tests were performed within the made ground at locations 15, 141, and 252, while two tests were carried out within granular superficial deposits at locations 53 and 183. All tests indicated very limited soil infiltration, making it impossible to determine the permeability from these tests.

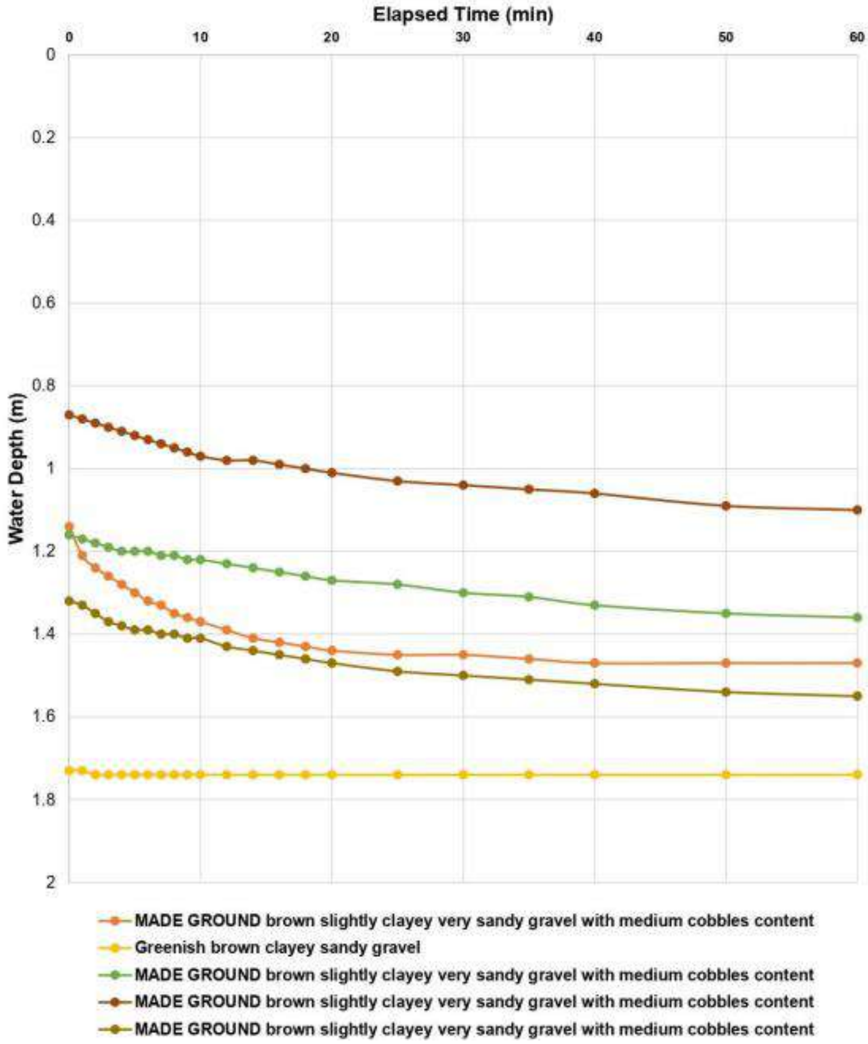

Figure 5: Results of soakaway tests

2.3 Results of Laboratory Tests

The laboratory tests conducted include both soil and rock testing. The soil tests primarily focus on physical parameters, while the rock tests emphasize strength parameters.

Soil Testing

Particle Size Distribution

The geotechnical testing along the existing conveyor route and in the reclamation yard areas revealed a diverse composition of superficial materials. These materials varied significantly, consisting of both cohesive and granular deposits. This section summarizes the cohesive and granular classification testing for these areas.

Table 1: Classification summary of cohesive superficial material

Location (bh)	**Depth** (m)	**Clay** (%)	**Silt** (%)	**Sand** (%)	**Gravel** (%)	**Cobbles** (%)
2 Conveyor route	1.0	16	37	35	12	0
6 Conveyor route	0.8	7	18	28	47	0
11 Conveyor route	1.5	12	37	33	18	0
11 Conveyor route	2.5	15	33	30	23	0
Average conveyor route		**12.5**	**31.3**	**31.5**	**25**	**0**
253 Reclamation yard	1.0	4	24	30	38	4
253 Reclamation yard	2.8	8	31	29	32	0
257 Reclamation yard	3.8	5	77	18	0	0
Average reclamation yard		**5.7**	**44**	**25.7**	**23**	**1.3**

Table 2: Classification summary of granular superficial material

Location (bh)	**Depth (m)**	**Clay (%)**	**Silt (%)**	**Sand (%)**	**Gravel (%)**	**Cobbles (%)**
2 Conveyor route	2.8	0	1	8	55	36
3 Conveyor route	2.2	5	8	29	59	0
4 Conveyor route	1.7	0	14	20	64	2
5 Conveyor route	1.8	4	13	34	45	5
7 Conveyor route	1.0	0	9	90	1	0
8 Conveyor route	1.0	5	11	85	0	0
9 Conveyor route	0.5	5	15	26	44	11

Location (bh)	Depth (m)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	Cobbles (%)
Average Conveyor route		**2.7**	**10.1**	**41.2**	**38.3**	**7.7**
23 Reclamation yard	1.0	2	19	21	52	5
23 Reclamation yard	2.0	0	8	35	57	0
253 Reclamation yard	4.8	2	21	29	45	2
253 Reclamation yard	5.8	0	5	8	42	45
257 Reclamation yard	6.8	0	4	19	77	0
Average Reclamation yard		**0.8**	**11.4**	**22.4**	**54.6**	**10.4**
177 Wooded area	1.0	3	17	33	47	0
177 Wooded area	3.0	2	21	25	51	1
Average wooded area		**2.5**	**19**	**29**	**49**	**0.5**

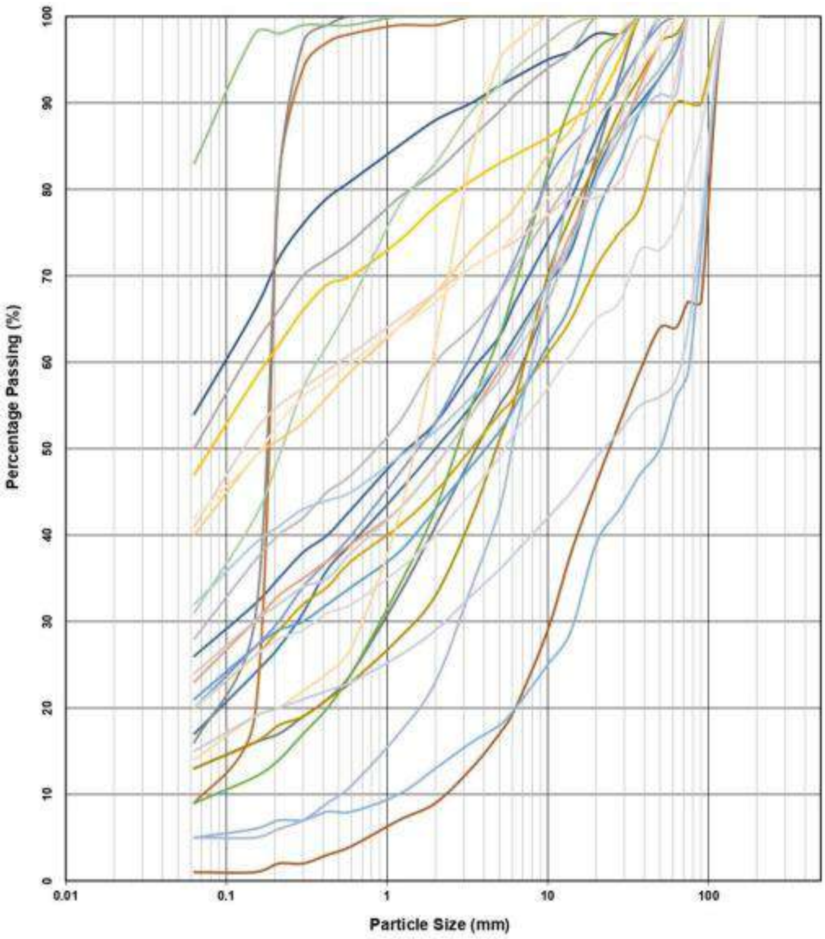

Figure 6: Particle Size Distribution Curves

Moisture Content & Atterberg Limits

Four plasticity tests were conducted along the conveyor route. The natural moisture contents were generally much higher than the plastic limits, indicating that the material is normally consolidated. The results of these tests are presented in Table 3.

Table 3: Plasticity results along conveyor route

Moisture content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Classification	
113	56	50	6	MH	High plasticity Silt
632	678	385	293	O	Organic Peat
90	128	66	62	ME	Extremely high plasticity Silt
28	44	30	14	MI	Intermediate plasticity Silt

Rock Testing

Point Load Index Test

A total of 163 paired diametral and axial point load tests were scheduled. However, due to the nature of the rock, several tests were invalidated as breaks did not occur between the points of load application. Valid result pairs (axial and diametral) were obtained for 62 samples, while a single diametral result was obtained for 95 samples.

The invalidated tests were caused by the rock fracturing along planes of weakness, generally oriented between 0 and 10 degrees, but occasionally at greater angles. All point load results were normalized for a 50mm diameter specimen (Is50). Figure 7 illustrates the scatter of diametral point loads against depth for the entire site. The normalized point load results show a seemingly random scatter with no clear relationship to depth.

Most results fall between 0.1 and 0.8 MN/m², with an average Is50 of 0.8 MN/m² when considering the entire range of results.

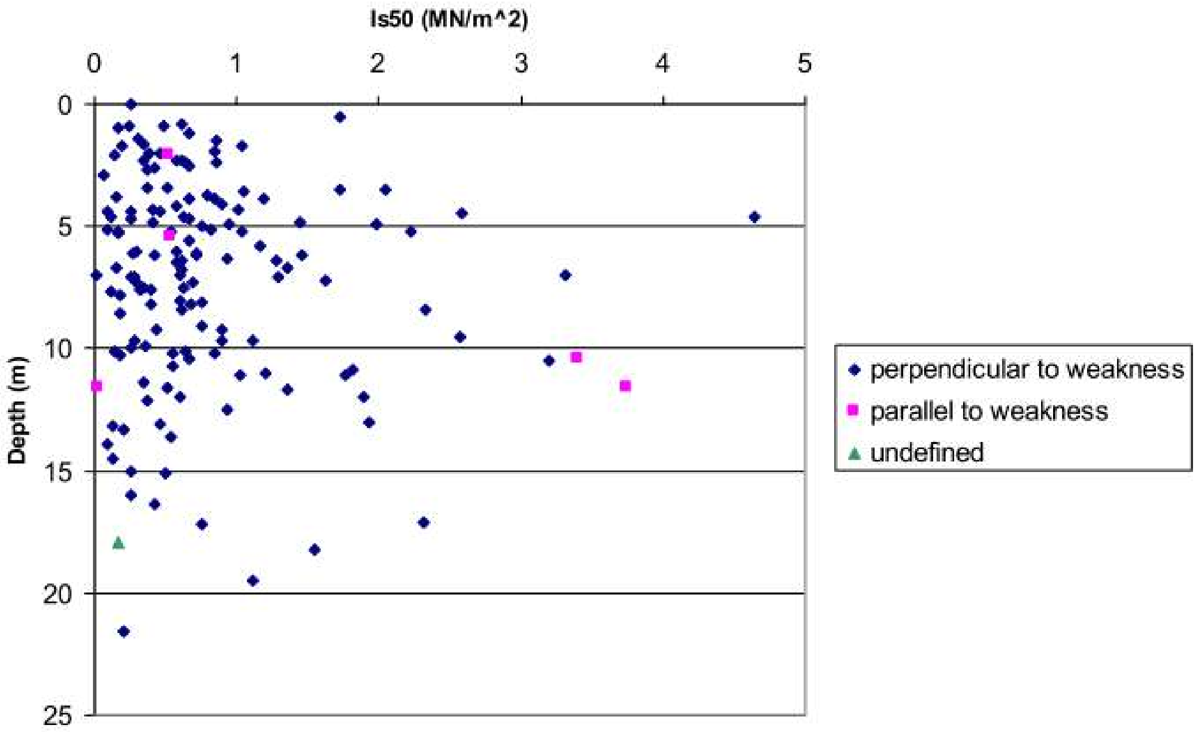

Figure 7: Results of Point Load Tests on Rock Specimens

Unconfined Compression Tests

A total of 32 UCS (Unconfined Compression Strength) tests were conducted on samples from 26 boreholes. Of these, 29 UCS tests included measurements of Young's modulus and Poisson's ratio. Figure 8 shows the scatter of UCS values with depth for all tests. Most UCS values range between 10 and 35 MN/m², with an average of 30 MN/m². Outliers include a low of 1.72 MN/m² and a high of 112 MN/m².

Young's modulus and Poisson's ratio were measured from 29 UCS specimens, with the results illustrated in Figure 9 & Figure 10.

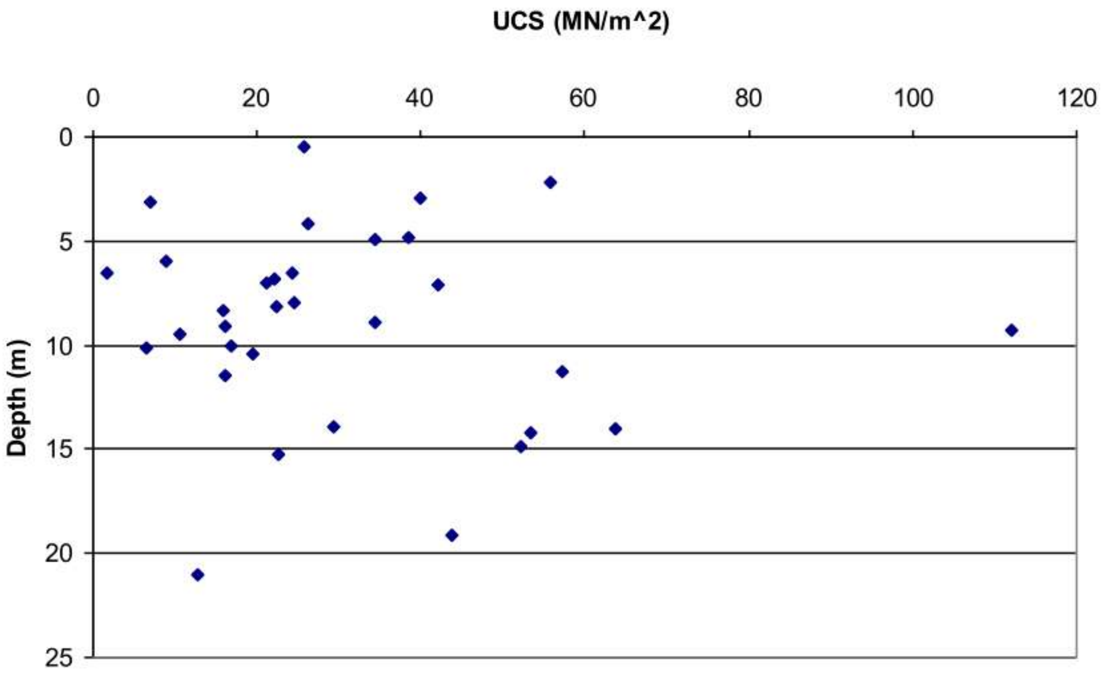

Figure 8: Unconfined Compression Strength of Rock Specimens

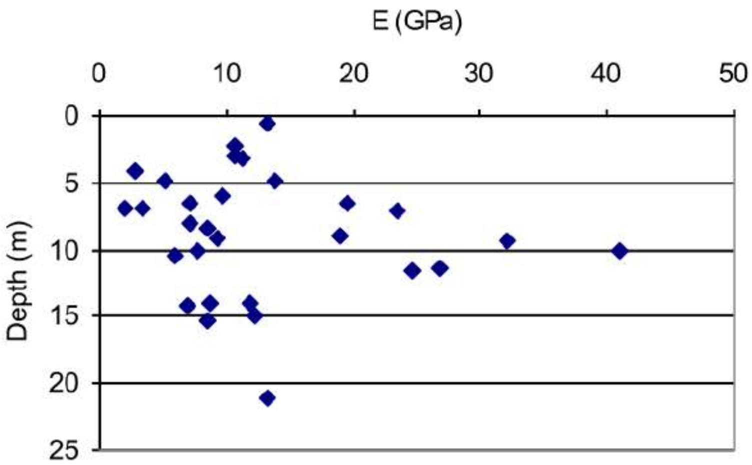

Figure 9: Young's Modulus of Rock Specimens

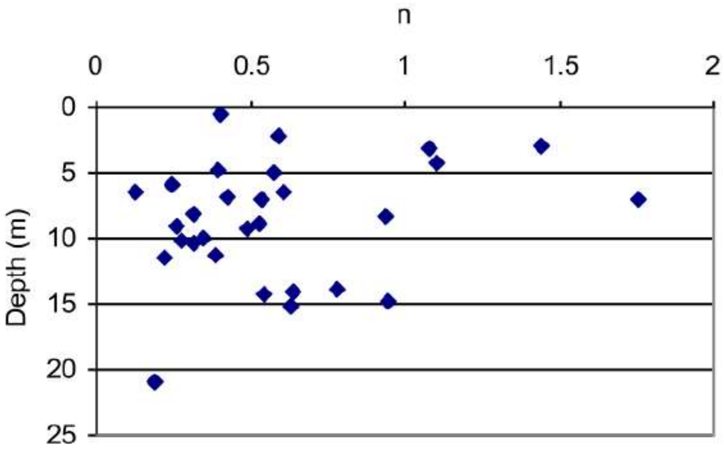

Figure 10: Poisson's Ratio of Rock Specimens

2.4 Summary

The summary of geotechnical tests, including both in-situ and laboratory tests, for each stratum is presented in Table 4 and Table 5 below.

Table 4: Summary of Geotechnical Test Results for Superficial Strata

Strata	NMC %	SPT	LL	PL	PI	CBR %	k m/s
Superficial granular	-	11 - 36	-	-	-	9.2 - 16	1e-6
Superficial cohesive	28 - 113	8 - 37	44 - 128	30 - 66	6 - 62	-	-
Weathered Schist	-	50 - 100	-	-	-	-	-

Table 5: Summary of Geotechnical Test Results for Intact Rock

Strata	Point Load (MPa)	UCS MPa	E_{intact} GPa	n	γ_b kg/m^3	NMC %	RQD %
Schist	0.1 – 0.8	10 - 35	1 - 15	0.3 – 0.6	2620 - 2770	0.1 – 0.7	15 - 30

3. Design Parameters

In the process of designing the foundations for the new buildings, two fundamental aspects demand meticulous attention: the assessment of bearing capacity and settlement estimation. Hence, our emphasis is directed towards the primary physical and mechanical parameters that significantly influence these two pivotal considerations. These parameters encompass:

- Physical parameters: Unit weight γ;
- Deformation parameters: Constrained modulus M;
- Shear strength parameters: Internal friction angle φ', Undrained shear strength s_u.

3.1 Soil Characteristics

Strength Parameters

SPT results were converted to strengths using relevant empirical relationships, with typical strengths provided for each individual stratum.

Cohesive Superficial Material

The undrained shear strength of cohesive strata was derived using Stroud's empirical relationship (Stroud, 1974 after Ciria 143, 1995)[1], which relates the plasticity index, SPT N-Value, and undrained shear strength.

SPT counts for the cohesive superficial material generally ranged from 8 to 37, with occasional very high counts likely due to inclusions such as cobbles and boulders, which should be discounted.

A conservative strength relationship of (Cu = 4N) is applicable for this stratum based on the highest PI of 62%, resulting in a shear strength range of 32-185 kN/m², equating to soft to stiff (BS 8004:1986)[2]. Considering the spread of in-situ testing results, a design undrained shear strength of 50 kN/m² is deemed appropriate.

Granular Superficial Material

The strength of granular strata was calculated using the empirical relationship from BS 8002:1994[3]:

$$\varphi_{max} = 30+A+B+C$$

where A, B, and C are parameters based on angularity, grading, and SPT 'N' counts, respectively.

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[1] CIRIA report 143. Clayton, 1995. The Standard Penetration Test (SPT): Methods and Use
[2] BS 8004 Code of Practice for Foundations, 1986
[3] BS 8002 Code of Practice for Earth Retaining Structures, 1994

SPT counts for the granular superficial material generally ranged from 11 to 36, with more outliers over 150 and a few low blow counts (N = 3) associated with the surface material of the conveyor route. The over 150 counts are likely due to cobbles and boulders, while the low blow counts are due to the waterlogged tidal nature of the conveyor route.

A φ_{max} value of 34-42 degrees was calculated for most of the reclamation yard area and conveyor route, although φ_{max} for the sands at the center of the conveyor route was significantly lower at 30 degrees.

Weathered Schist

SPT counts for the weathered schist were generally higher than those of the superficial material, ranging from 50 to 100. Some outliers included blow counts as low as 21, but these were outnumbered by counts exceeding 150.

A φ_{max} value of 40-43 degrees was calculated for most of the weathered schists encountered during the site investigation.

Deformation Modulus

Densities of the superficial deposits were derived from literature (Tomlinson, 2001)[4]. Poisson ratio were also derived from the literature (Tomlinson, 2001).

Undrained cohesive stiffness parameters of the cohesive deposits were determined using geotechnical data from the GIR and the published relationship (CIRIA 143):

Eu = (1.0 – 1.2)N (MPa)

where N is the number of blow counts. A value of 1.0 was used within the above equation which relates to a worst case plasticity of 62%.

Drained cohesive stiffness parameters were derived using the relationship (CIRIA 143):

E’ = 0.73 Eu

Granular Young’s moduli were derived using the lower mean E’/N relationships with penetration resistance published Burland and Burbidge (CIRIA 143) and reproduced within Table 6.

Table 6: Young’s modulus derived from Burland and Burbidge’s Ic values

Penetration Resistance (blows/300mm)	E’/N (MPa) at		
	Mean	Lower limit	Upper limit
4	1.6 - 2.4	0.4 – 0.6	3.5 – 5.3
10	2.2 – 3.4	0.7 – 1.1	4.6 – 7.0

[4] Tomlinson, M.J. 2001. Foundation Design and Construction, 7th edition. Pearson Education Ltd, Harlow

30	3.7 – 5.6	1.5 – 2.2	6.6 – 10.0
60	4.6 – 7.0	2.3 – 3.5	8.9 - 13.5

3.2 Rock Mass Characteristics

The characteristics of the rock mass were determined using the Rock Quality Designation (RQD). Figure 11 illustrates the variation of RQD with depth. This figure indicates that there is little correlation between RQD and depth. An average RQD of 22% was calculated, with values generally ranging from 15% to 30%.

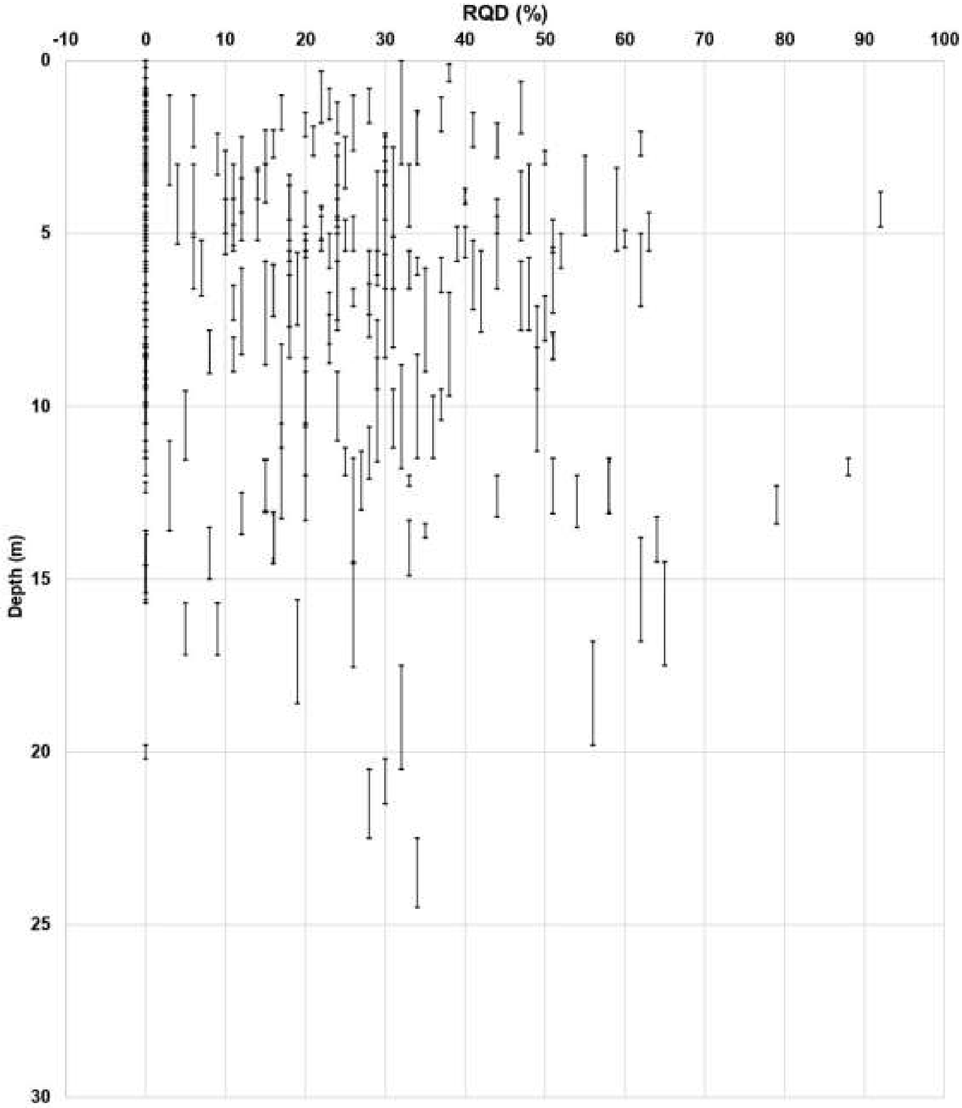


Figure 11: Variation of RQD with depth

Strength Parameters

Kulhawy and Goodman (1980 and 1987, as cited in Tomlinson 2001) have demonstrated that the cohesion c and friction angle (φ) parameters can be related to RQD. They proposed the relationships detailed in Table 5.5, where q_c is the ultimate bearing capacity of the rock mass and q_{uc} is the unconfined compressive strength of an intact specimen.

Table 7: Relationship between RQD and rock mass parameters

RQD%	Rock Mass Properties		
	q_c	c	φ (°)
0 - 70	$0.33q_{uc}$	$0.1q_{uc}$	30
70 - 100	$0.33q_{uc} – 0.80q_{uc}$	$0.1q_{uc}$	30 – 60

Most of the rock mass encountered during the site investigation had an RQD between 0% and 70%. Consequently, rock mass properties were calculated using the UCS values obtained from laboratory and field tests, as summarized in Section 5.3.2.7. These properties are detailed in Table 5.6.

Deformation Modulus

The deformation modulus of the rock mass Em is calculated using the expression:

$$E_m = j \cdot E$$

where j is a mass factor related to RQD, and E is the Young's modulus of the intact rock.

Based on the range of RQD aforementioned, a mass factor j value of 0.2 was obtained (Tomlinson, 2001).

3.3 Summary

For convenience, the following is a summary table of all design parameters for each stratum, including both soils and rock.

Table 8: Summary of Design Parameters

Strata		γ_b kg/m^3	c MPa	Ø °	E_m GPa	n
Superficial deposits	cohesive	2000[5]	c_u = 0.050	0	E_u = 0.012 E' = 0.009	0.1 (drained)
	granular	2000[6]	-	34 – 42	E' = 0.031	0.2
Weathered rock		2660	-	40 - 43	E' = 0.185	0.2

[5] Assumed from literature
[6] Assumed from literature

Strata	**γ_b kg/m^3**	**c MPa**	**Ø °**	**E_m GPa**	**n**
Rock mass	2660 + 4z	0.5 – 2.5	30	0.2 + 0.1z	0.4

4. Design Concept

The design concept has a number of key features to it; these are discussed within this chapter.

4.1 Ground models

Two ground models are presented - one for the northwestern side and one for the southeastern side.
- The northwestern side ground model has up to 8.0m of superficial deposits and weathered rock overlying intact bedrock.
- The southeastern side ground model has up to 4.3m of superficial deposits over bedrock.

The finished floor level (FFL) is not available at this moment, but it is expected to be similar to the natural ground level (NGL). Therefore, for the purposes of this analysis, the FFL is assumed to be equal to the NGL

The graphical representation of the ground model is depicted in the figure below.

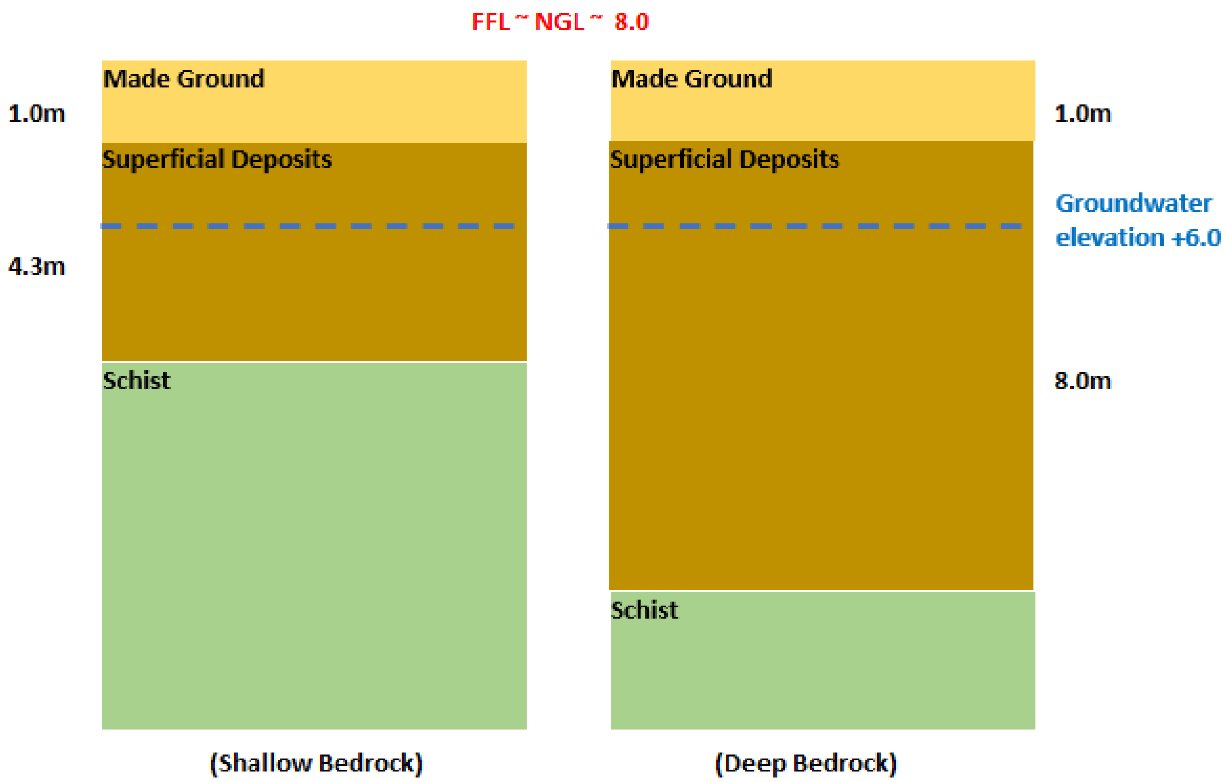


Figure 12: Ground model

In this figure, the superficial deposits can be either the superficial cohesive deposits or superficial granular deposits. The made ground layer in natural ground will be excavated and replaced by the Engineered Fill material in the final ground condition.

4.2 Proposed Soil improvement works

The soil improvement strategy with regard to the building areas consist of several key steps as follows:

Demolition of Existing Structures: The main goal of this phase is to dismantle the current structures at the former Anglesey Aluminium plant site in Holyhead safely and systematically. This step is essential to prepare the site for a variety of mixed-use industrial developments, ensuring that the area is clear for subsequent ground improvement and construction tasks. The Demolition phase is well underway, with the main Pot Line and other major former buildings now demolished.

Material Crushing and Processing: This phase focuses on transforming the debris from the demolished structures into reusable materials for ground improvement and construction. This approach is vital for minimizing waste, reducing the necessity for new materials, and promoting sustainable site redevelopment.

Earthworks: The earthworks phase aims to prepare the site for construction by establishing a stable foundation. This involves adjusting the terrain through excavation, filling, grading, and compaction to achieve optimal ground conditions.

Soil Improvement: This can include methods such as Lime Modification/Stabilization, which enhances the soil's engineering properties by increasing its strength and stability, improving its load-bearing capacity, and reducing settlement. Alternatively, Rolling Dynamic Compaction (RDC) is employed to densify loose soil layers, especially those consisting of made ground, to depths greater than 2 meters - where traditional methods may be less effective. This is crucial for establishing a dependable foundation for construction and ensuring the long-term stability of the ground.

4.3 Building Foundation

Selection of typical foundations

- In the absence of detailed foundation design information, assumptions are made based on typical values for bearing capacity and settlement analysis. Given that the size of the foundation directly impacts both bearing capacity and settlement, two foundation dimensions are selected for analysis: 2m x 2m and 4m x 4m.
- The foundation depth is assumed to be 1 meter, which means it remains above the superficial deposits. Consequently, the parameters of the made ground layer have minimal impact on the analysis results. Additionally, in practice, the made ground layer will be excavated and replaced by the Engineered Fill material.

Design loading

- The applied load intensity on the foundation base stands at 150kPa.
- The load distribution on the slab is envisioned as uniformly distributed, with an intensity of 50kPa.

Design criteria

The design criteria for this project are:

- ULS bearing capacity of pad foundation > 150kPa;
- Post construction settlement of pad foundation < 50mm;
- Differential settlement of ground bearing slab < 1:500.

5. Design Analysis and Discussion

5.1 Analysis Results

The analysis was carried out using two typical soil stratifications presented in section 4.1. The outcomes are summarized in Table 9. The detailed calculations are presented in Appendix B.

Table 9: Summary of analysis results

Strata	Foundation size	Bearing capacity	Settlement
Deep Bedrock Cohesive Deposits	2m x 2m	Rd = 225kPa > σ = 174kPa Satisfactory	41.2 mm < 50mm Satisfactory
	4m x 4m	Rd = 225kPa > σ = 174kPa Satisfactory	58.9 mm > 50mm Not satisfactory
Deep Bedrock Granular Deposits	2m x 2m	Rd = 583kPa > σ = 174kPa Satisfactory	12 mm < 50mm Satisfactory
	4m x 4m	Rd = 693kPa > σ = 174kPa Satisfactory	17.3 mm < 50mm Satisfactory
Shallow Bedrock Cohesive Deposits	2m x 2m	Rd = 225kPa > σ = 174kPa Satisfactory	32 mm < 50mm Satisfactory
	4m x 4m	Rd = 225kPa > σ = 174kPa Satisfactory	44.4 mm < 50mm Satisfactory
Deep Bedrock Granular Deposits	2m x 2m	Rd = 583kPa > σ = 174kPa Satisfactory	9.6 mm < 50mm Satisfactory
	4m x 4m	Rd = 7026kPa > σ = 174kPa Satisfactory	13.6 mm < 50mm Satisfactory

5.2 Discussion

Here are the observations from the analysis results:

- The bearing capacity is satisfied in all analysed cases.
- The settlement criteria are also met, except in the case of the deep bedrock strata. For the 4m x 4m foundation on a cohesive superficial deposit, the settlement slightly exceeds the criteria. It should be noted that this value represents total settlement, with a portion occurring during construction. Therefore, the structure may experience a final settlement within acceptable limits. If further reduction in settlement is required, intervention measures noted in the geotechnical remediation strategy should be employed.

5.3 Conclusion

The analysis of bearing capacity and settlement for typical foundation sizes across the site's strata indicates satisfactory performance. Therefore, there is nothing to preclude the use of shallow foundations for the development site.

6. Summary and Conclusions

Project Prosperity Parc involves the proposed development of several buildings. The site underwent various investigations between 2010 and 2015. Multiple geotechnical and geoenvironmental reports from these investigations were collected and summarized – Appendix B Historical Review and Summary Report summarises the various past studies in some detail. The body of evidence from the various assessments and reports undertaken across much of the site provides a good understanding of the general characteristics of the site with regard to ground conditions and geoetechnical attributes, and confirms the suitability of the site for redevelopment. Although not comprehensive in its coverage, that evidence is directly informing a strategy for how to efficiently deliver new buildings, and where required any gaps or deficiencies in the available data will be addressed before construction begins.

After careful analysis, the site has been divided into two distinct ground models to aid planning objectives: shallow bedrock and deep bedrock. The data collected from historical reports helped derive the design parameters.

Bearing capacity and settlement analyses were performed using typical industrial building foundation dimensions of 2m x 2m and 4m x 4m, with a foundation depth of 1.5m. The loadings used were typical, with 150 kPa on the foundation and 50 kPa slab loading.

Based on the analysis conducted in this report, along with the Geotechnical Remediation Strategy, there are no identified obstacles to the site's re-development. Furthermore, there are no issues preventing the use of shallow pad foundations or ground-bearing floor slabs for the proposed development.

7. List of Appendices

The reports and/or documents that form the various appendices to this document are:

Appendix A – Foundation Analysis

Appendix B – HBGS FAAP Historical Review and Summary Report

Appendix C – HBGS FAAP Geotechnical Ground Model and Design Parameters

Appendix D – Technical Note – Soakaways

Appendices

Appendix A – Foundation Analysis

Spread footing verification

Input data

Project

Task : Holyhead
Part : Deep Bedrock - Cohesive Deposits - Foundation 2m x 2m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)									
Permanent design situation									
		Combination 1				Combination 2			
		Unfavourable		Favourable		Unfavourable		Favourable	
Permanent actions :	γ_G =	1.35	[–]	1.00	[–]	1.00	[–]	1.00	[–]

Partial factors for soil parameters (M)					
Permanent design situation					
		Combination 1		Combination 2	
Partial factor on internal friction :	γ_ϕ =	1.00	[–]	1.25	[–]
Partial factor on effective cohesion :	γ_c =	1.00	[–]	1.25	[–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00	[–]	1.40	[–]
Partial factor on unconfined strength :	γ_v =	1.00	[–]	1.40	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Superficial Cohesive Deposits		0.00	50.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Cohesive Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	0.00 °
Cohesion of soil :	c_{ef}	=	50.00 kPa
Oedometric modulus :	E_{oed}	=	9.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

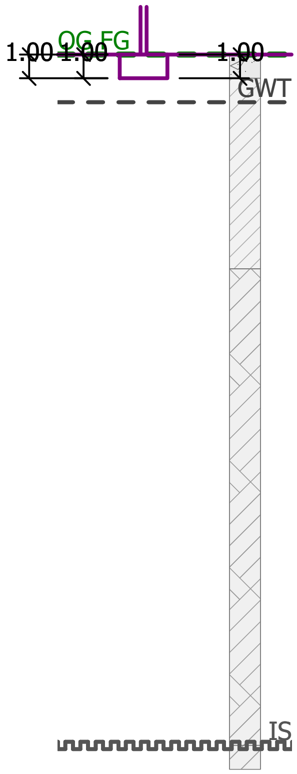
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation	Stage - analysis : 1 - 0



Geometry of structure

Foundation type: centric spread footing

Spread footing length	x	=	2.00	m
Spread footing width	y	=	2.00	m
Column width in the direction of x	c_x	=	0.25	m
Column width in the direction of y	c_y	=	0.25	m
Spread footing volume		=	4.00	m^3

Material of structure

Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

Concrete : C 20/25

Cylinder compressive strength	f_{ck}	=	20.00	MPa
Tensile strength	f_{ctm}	=	2.20	MPa
Elasticity modulus	E_{cm}	=	30000.00	MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

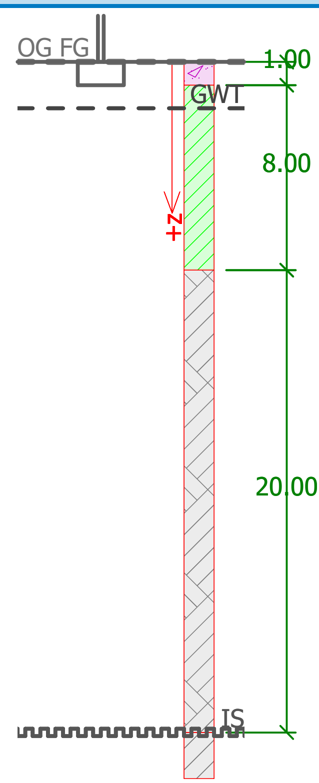
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	8.00	Superficial Cohesive Deposits	
3	20.00	Schist	
4	-	Schist	

Name : Profile and assignment	Stage - analysis : 1 - 0



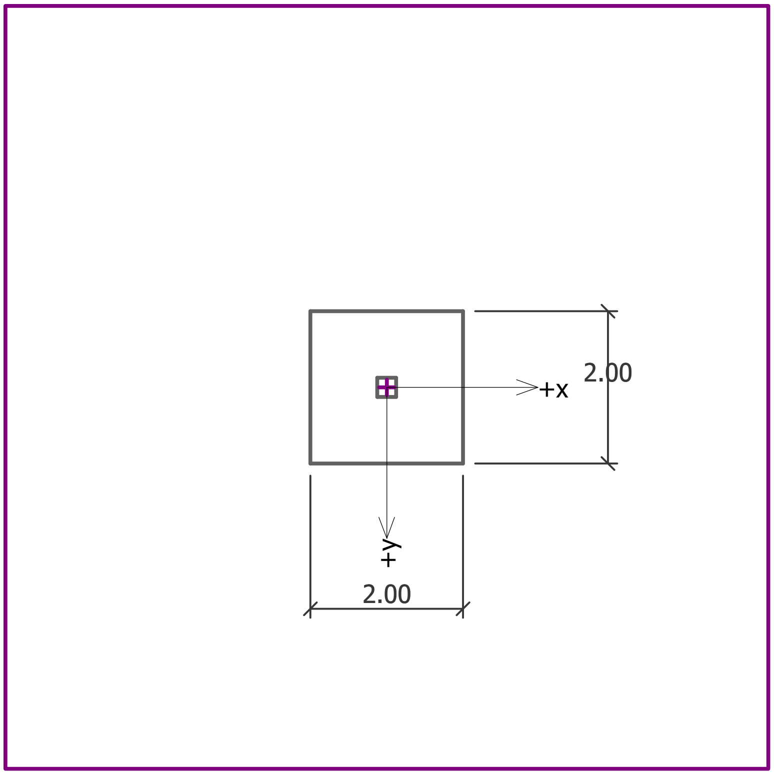
Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		ULS	Design	600.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge	Stage - analysis : 1 - 0



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 1.42 m
Length of slip surface l_{sp} = 3.00 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

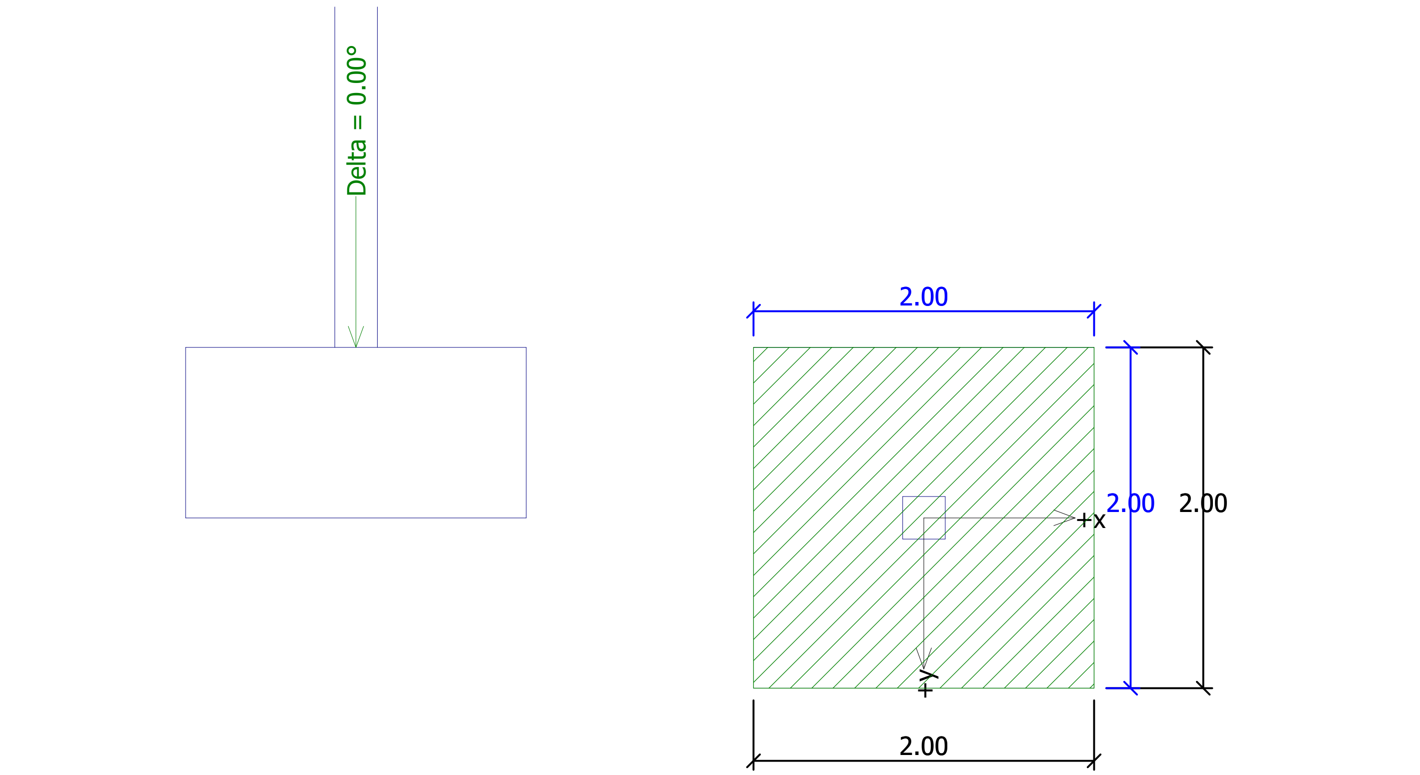
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 210.14 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap.	Stage - analysis : 1 - 1



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 17.3 mm
Settlement of mid point of edge x - 2 = 17.3 mm
Settlement of mid point of edge y - 1 = 17.3 mm
Settlement of mid point of edge y - 2 = 17.3 mm
Settlement of foundation center point = 26.9 mm
Settlement of characteristic point = 19.4 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 8.80 MPa
Foundation in the longitudinal direction is rigid (k=426.14)
Foundation in the direction of width is rigid (k=426.14)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

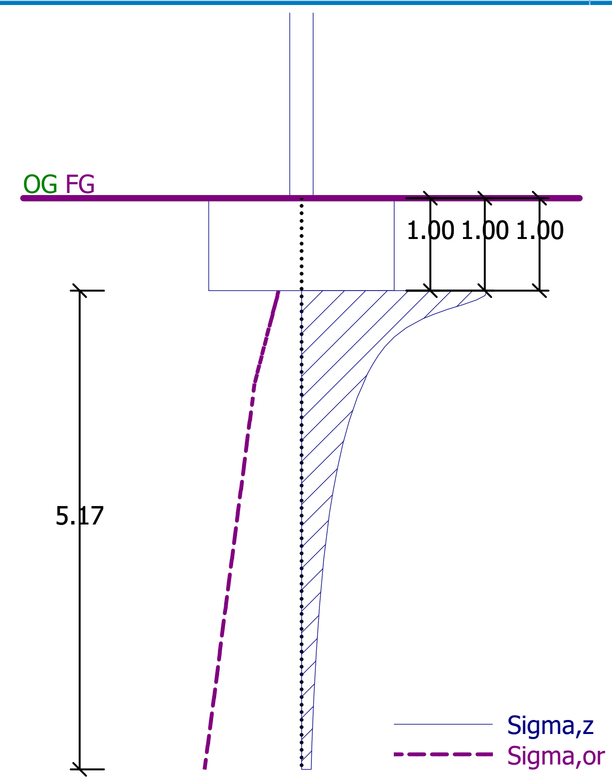
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 19.4 mm
Depth of influence zone = 5.17 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement	Stage - analysis : 1 - 1



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Cohesive Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing $G = 94.25$ kN
Computed weight of overburden $Z = 0.00$ kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface $z_{sp} = 1.42$ m
Length of slip surface $l_{sp} = 3.00$ m

Design bearing capacity of found.soil $R_d = 224.66$ kPa
Extreme contact stress $\sigma = 173.56$ kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length $e_x = 0.000<0.333$
Max. eccentricity in direction of base width $e_y = 0.000<0.333$
Max. overall eccentricity $e_t = 0.000<0.333$

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 210.14 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 39.1 mm
Settlement of mid point of edge x - 2 = 39.1 mm
Settlement of mid point of edge y - 1 = 39.1 mm
Settlement of mid point of edge y - 2 = 39.1 mm
Settlement of foundation center point = 48.6 mm
Settlement of characteristic point = 41.2 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 15.19 MPa
Foundation in the longitudinal direction is rigid (k=246.92)
Foundation in the direction of width is rigid (k=246.92)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 41.2 mm
Depth of influence zone = 9.42 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Cohesive Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 1.42 m
Length of slip surface l_{sp} = 3.00 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 210.14 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 52.4 mm
Settlement of mid point of edge x - 2 = 52.4 mm
Settlement of mid point of edge y - 1 = 52.4 mm
Settlement of mid point of edge y - 2 = 52.4 mm
Settlement of foundation center point = 61.9 mm
Settlement of characteristic point = 54.6 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 23.91 MPa
Foundation in the longitudinal direction is rigid (k=156.85)
Foundation in the direction of width is rigid (k=156.85)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 54.6 mm
Depth of influence zone = 11.15 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 600.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil = 9.38 kN
Force transmitted by shear strength of SRC = 590.62 kN
Considered column perimeter u_0 = 1.00 m
Shear resistance at the column perimeter $v_{Ed,max}$ = 0.62 MPa
Resistance at the column perimeter $v_{Rd,max}$ = 2.94 MPa

Critical section without shear reinforcement

Force transmitted into found. soil = 187.50 kN
Force transmitted by shear strength of SRC = 412.50 kN
Distance of section from the column = 0.48 m
Section perimeter u_{cr} = 3.99 m
Shear stress at section v_{Ed} = 0.11 MPa
Shear resistance of section without shear reinforcement $v_{Rd,c}$ = 1.10 MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Spread footing verification

Input data

Project

Task : Holyhead
Part : Deep Bedrock - Cohesive Deposits - Foundation 4m x 4m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)									
Permanent design situation									
		Combination 1				Combination 2			
		Unfavourable		Favourable		Unfavourable		Favourable	
Permanent actions :	γ_G =	1.35	[–]	1.00	[–]	1.00	[–]	1.00	[–]

Partial factors for soil parameters (M)					
Permanent design situation					
		Combination 1		Combination 2	
Partial factor on internal friction :	γ_ϕ =	1.00	[–]	1.25	[–]
Partial factor on effective cohesion :	γ_c =	1.00	[–]	1.25	[–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00	[–]	1.40	[–]
Partial factor on unconfined strength :	γ_v =	1.00	[–]	1.40	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Superficial Cohesive Deposits		0.00	50.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Cohesive Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	0.00 °
Cohesion of soil :	c_{ef}	=	50.00 kPa
Oedometric modulus :	E_{oed}	=	9.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

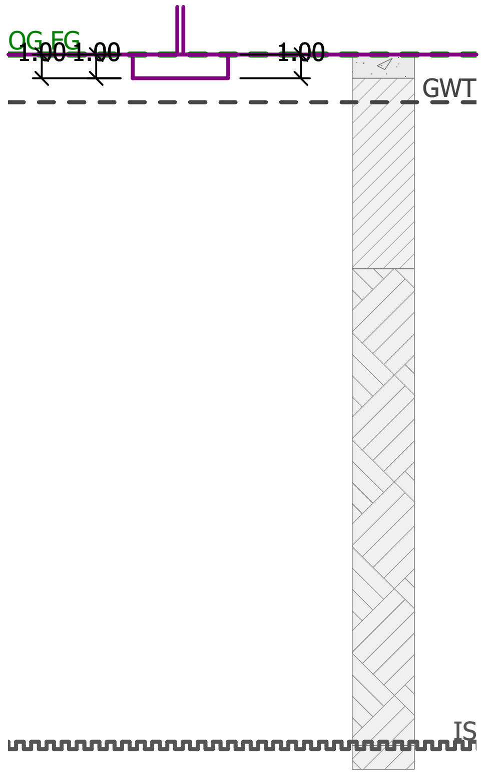
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation	Stage - analysis : 1 - 0



Geometry of structure

Foundation type: centric spread footing

Spread footing length	x	=	4.00	m
Spread footing width	y	=	4.00	m
Column width in the direction of x	c_x	=	0.25	m
Column width in the direction of y	c_y	=	0.25	m
Spread footing volume		=	16.00	m^3

Material of structure

Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

Concrete : C 20/25

Cylinder compressive strength	f_{ck}	=	20.00	MPa
Tensile strength	f_{ctm}	=	2.20	MPa
Elasticity modulus	E_{cm}	=	30000.00	MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

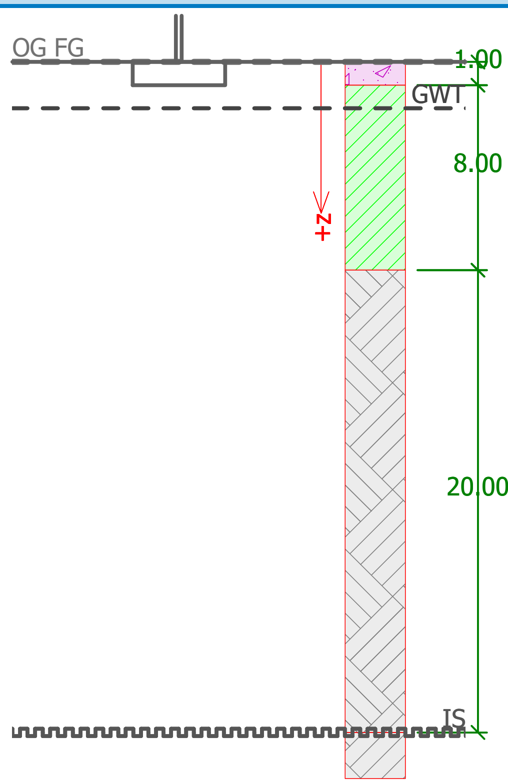
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	8.00	Superficial Cohesive Deposits	
3	20.00	Schist	
4	-	Schist	

Name : Profile and assignment	Stage - analysis : 1 - 0



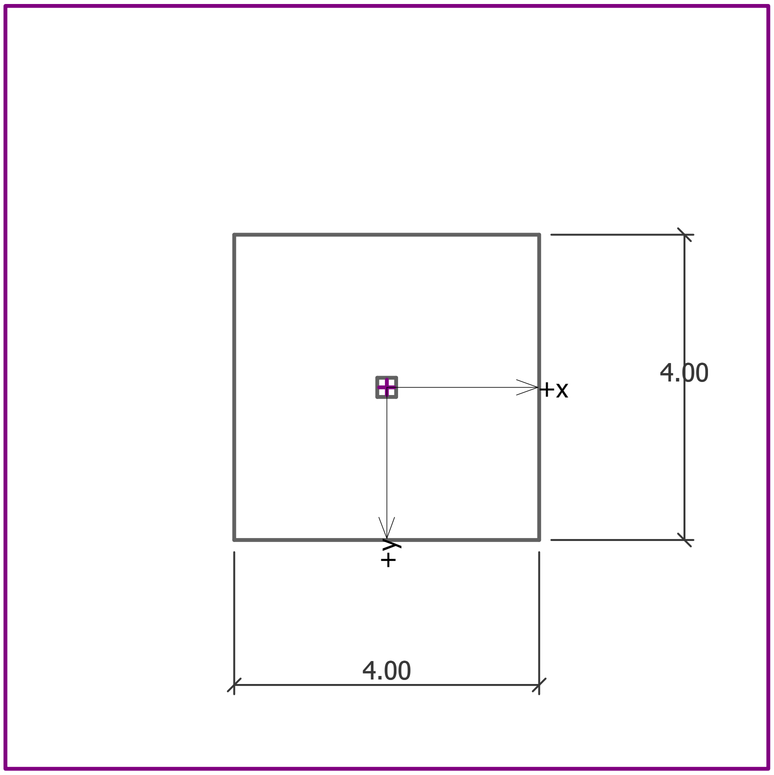
Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge	Stage - analysis : 1 - 0



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 2.83 m
Length of slip surface l_{sp} = 6.01 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

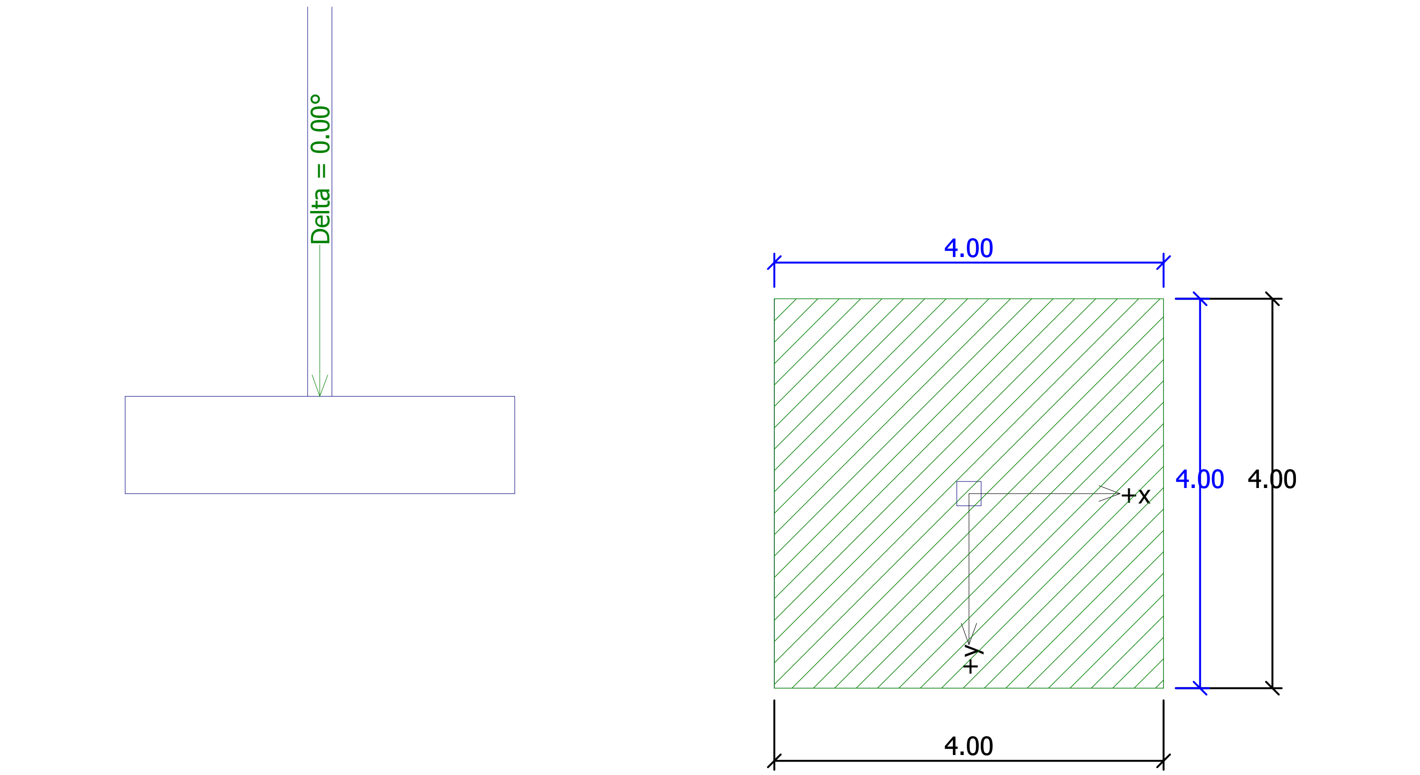
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 820.28 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap. | **Stage - analysis : 1 - 1**



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 34.3 mm
Settlement of mid point of edge x - 2 = 34.3 mm
Settlement of mid point of edge y - 1 = 34.3 mm
Settlement of mid point of edge y - 2 = 34.3 mm
Settlement of foundation center point = 54.6 mm
Settlement of characteristic point = 38.8 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 13.86 MPa
Foundation in the longitudinal direction is rigid (k=33.81)
Foundation in the direction of width is rigid (k=33.81)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

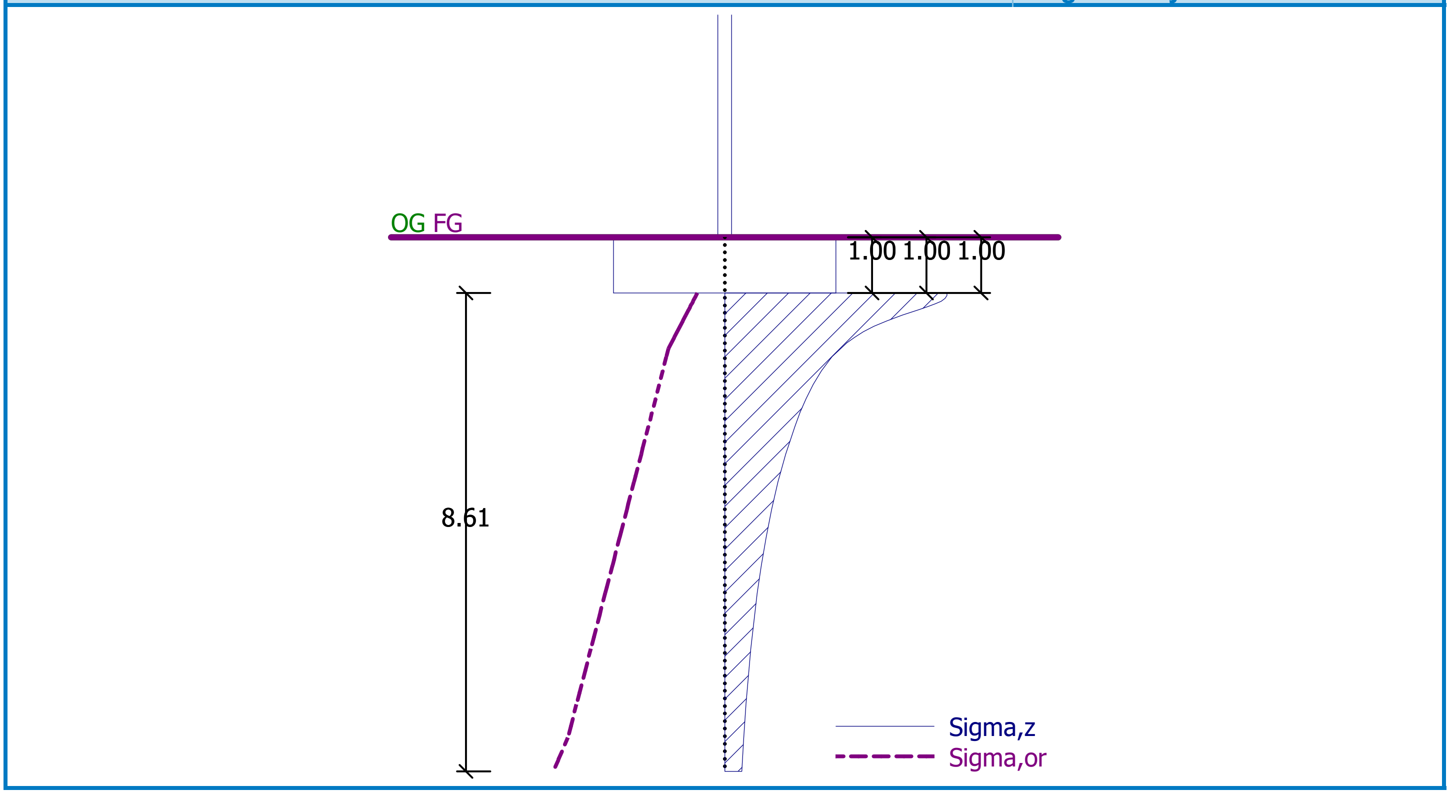
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 38.8 mm
Depth of influence zone = 8.61 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement | **Stage - analysis : 1 - 1**



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Cohesive Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing $G = 377.01$ kN
Computed weight of overburden $Z = 0.00$ kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface $z_{sp} = 2.83$ m
Length of slip surface $l_{sp} = 6.01$ m

Design bearing capacity of found.soil $R_d = 224.66$ kPa
Extreme contact stress $\sigma = 173.56$ kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length $e_x = 0.000<0.333$
Max. eccentricity in direction of base width $e_y = 0.000<0.333$
Max. overall eccentricity $e_t = 0.000<0.333$

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 820.28 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 54.3 mm
Settlement of mid point of edge x - 2 = 54.3 mm
Settlement of mid point of edge y - 1 = 54.3 mm
Settlement of mid point of edge y - 2 = 54.3 mm
Settlement of foundation center point = 74.7 mm
Settlement of characteristic point = 58.9 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 22.45 MPa
Foundation in the longitudinal direction is rigid (k=20.88)
Foundation in the direction of width is rigid (k=20.88)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 58.9 mm
Depth of influence zone = 11.22 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Cohesive Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load		Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
	new	change							
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge		Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
	new	change								
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 2.83 m
Length of slip surface l_{sp} = 6.01 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 820.28 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 67.7 mm
Settlement of mid point of edge x - 2 = 67.7 mm
Settlement of mid point of edge y - 1 = 67.7 mm
Settlement of mid point of edge y - 2 = 67.7 mm
Settlement of foundation center point = 88.0 mm
Settlement of characteristic point = 72.3 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 29.18 MPa
Foundation in the longitudinal direction is rigid (k=16.06)
Foundation in the direction of width is rigid (k=16.06)

Verification of load eccentricity

Max. excentricity in direction of base length	e_x	=	0.000<0.333
Max. eccentricity in direction of base width	e_y	=	0.000<0.333
Max. overall eccentricity	e_t	=	0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 72.3 mm
Depth of influence zone = 12.63 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter	=	16.0	mm
Number of bars	=	11	
Reinforcement cover	=	40.0	mm
Cross-section width	=	4.00	m
Cross-section depth	=	1.00	m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter	=	16.0	mm
Number of bars	=	11	
Reinforcement cover	=	40.0	mm
Cross-section width	=	4.00	m
Cross-section depth	=	1.00	m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 2400.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil		=	9.38	kN
Force transmitted by shear strength of SRC		=	2390.62	kN
Considered column perimeter	u_0	=	1.00	m
Shear resistance at the column perimeter	$v_{Ed,max}$	=	2.51	MPa
Resistance at the column perimeter	$v_{Rd,max}$	=	2.94	MPa

Critical section without shear reinforcement

Force transmitted into found. soil		=	356.61	kN
Force transmitted by shear strength of SRC		=	2043.39	kN
Distance of section from the column		=	0.71	m
Section perimeter	u_{cr}	=	5.49	m
Shear stress at section	v_{Ed}	=	0.39	MPa
Shear resistance of section without shear reinforcement	$v_{Rd,c}$	=	0.74	MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Spread footing verification

Input data

Project

Task : Holyhead
Part : Deep Bedrock - GranularDeposits - Foundation 2m x 2m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)					
Permanent design situation					
		Combination 1		Combination 2	
		Unfavourable	Favourable	Unfavourable	Favourable
Permanent actions :	γ_G =	1.35 [–]	1.00 [–]	1.00 [–]	1.00 [–]

Partial factors for soil parameters (M)			
Permanent design situation			
		Combination 1	Combination 2
Partial factor on internal friction :	γ_ϕ =	1.00 [–]	1.25 [–]
Partial factor on effective cohesion :	γ_c =	1.00 [–]	1.25 [–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00 [–]	1.40 [–]
Partial factor on unconfined strength :	γ_v =	1.00 [–]	1.40 [–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Superficial Granular Deposits		34.00	0.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Granular Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	34.00 °
Cohesion of soil :	c_{ef}	=	0.00 kPa
Oedometric modulus :	E_{oed}	=	31.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

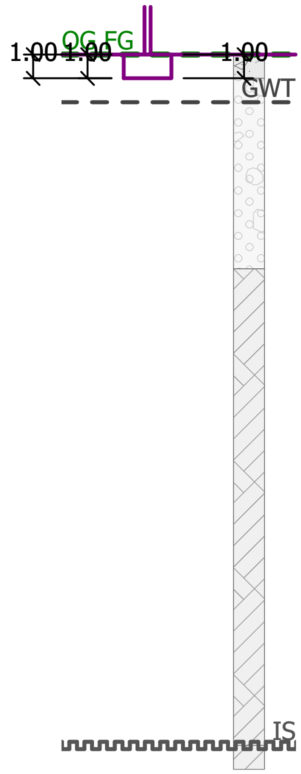
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation	Stage - analysis : 1 - 0



Geometry of structure

Foundation type: centric spread footing

Spread footing length	x	=	2.00	m
Spread footing width	y	=	2.00	m
Column width in the direction of x	c_x	=	0.25	m
Column width in the direction of y	c_y	=	0.25	m
Spread footing volume		=	4.00	m^3

Material of structure

Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

Concrete : C 20/25

Cylinder compressive strength	f_{ck}	=	20.00	MPa
Tensile strength	f_{ctm}	=	2.20	MPa
Elasticity modulus	E_{cm}	=	30000.00	MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

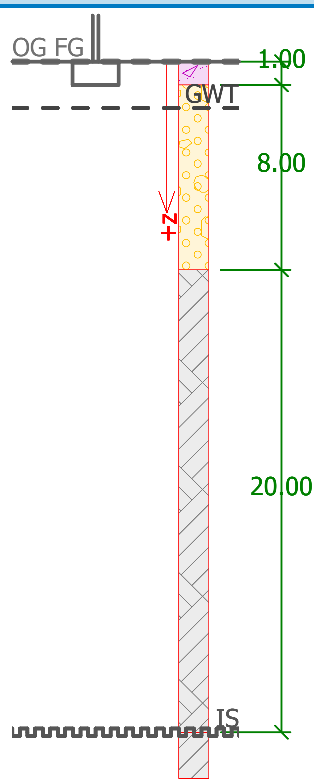
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	8.00	Superficial Granular Deposits	
3	20.00	Schist	
4	-	Schist	

Name : Profile and assignment	Stage - analysis : 1 - 0



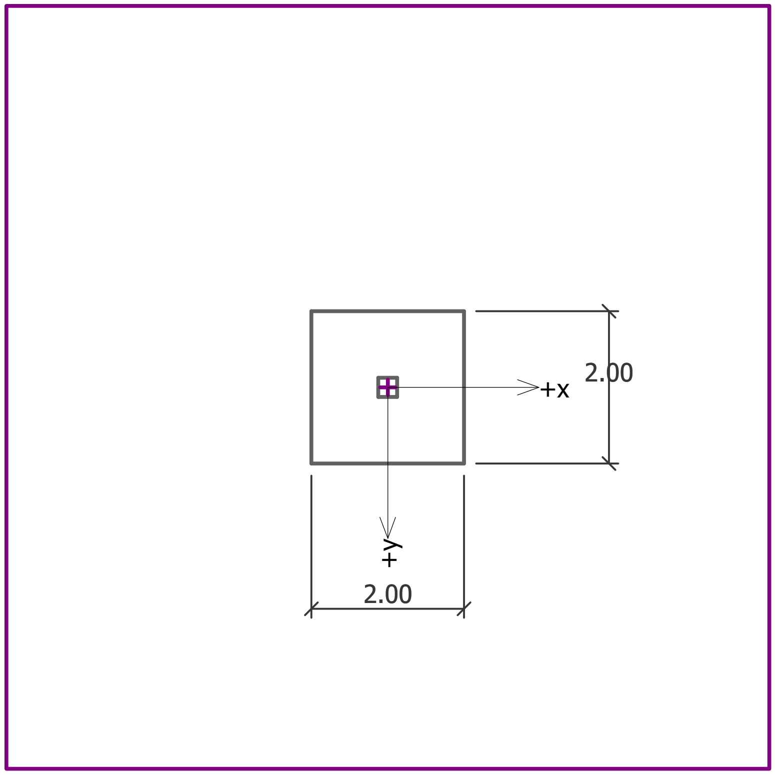
Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		ULS	Design	600.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge	Stage - analysis : 1 - 0



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1254.18	13.84	Yes
ULS	No	0.00	0.00	181.81	1254.18	14.50	Yes
SLS	Yes	0.00	0.00	173.56	582.47	29.80	Yes
SLS	No	0.00	0.00	173.56	582.47	29.80	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 3.67 m
Length of slip surface l_{sp} = 11.86 m

Design bearing capacity of found.soil R_d = 582.47 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

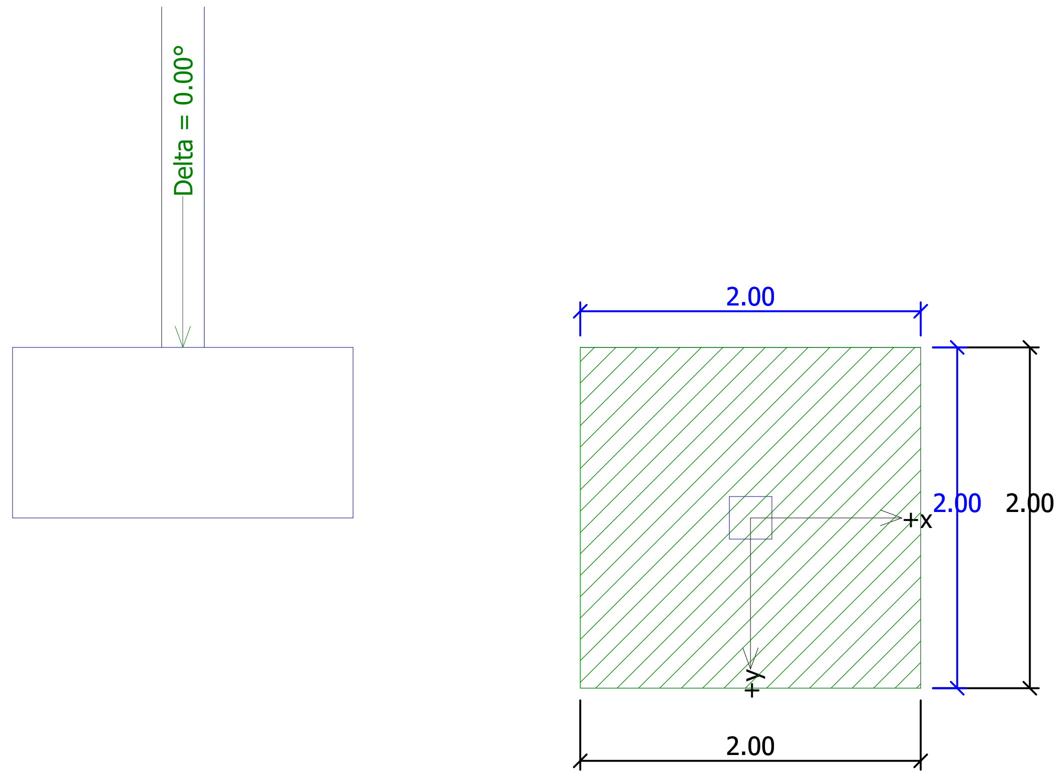
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 478.42 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap. **Stage - analysis : 1 - 1**



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 5.0 mm
Settlement of mid point of edge x - 2 = 5.0 mm
Settlement of mid point of edge y - 1 = 5.0 mm
Settlement of mid point of edge y - 2 = 5.0 mm
Settlement of foundation center point = 7.8 mm
Settlement of characteristic point = 5.6 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 27.90 MPa
Foundation in the longitudinal direction is rigid (k=134.41)
Foundation in the direction of width is rigid (k=134.41)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

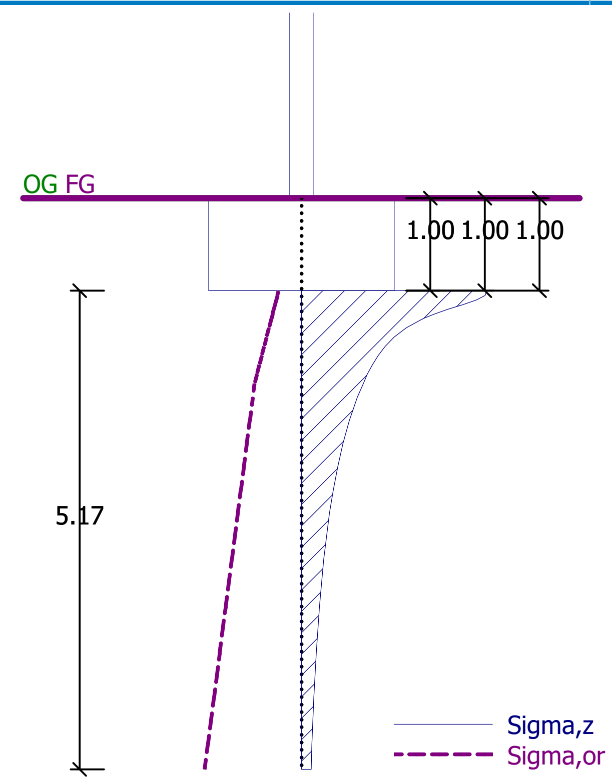
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 5.6 mm
Depth of influence zone = 5.17 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement	Stage - analysis : 1 - 1



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Granular Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1254.18	13.84	Yes
ULS	No	0.00	0.00	181.81	1254.18	14.50	Yes
SLS	Yes	0.00	0.00	173.56	582.47	29.80	Yes
SLS	No	0.00	0.00	173.56	582.47	29.80	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 3.67 m
Length of slip surface l_{sp} = 11.86 m

Design bearing capacity of found.soil R_d = 582.47 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 478.42 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 11.4 mm
Settlement of mid point of edge x - 2 = 11.4 mm
Settlement of mid point of edge y - 1 = 11.4 mm
Settlement of mid point of edge y - 2 = 11.4 mm
Settlement of foundation center point = 14.2 mm
Settlement of characteristic point = 12.0 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 33.41 MPa
Foundation in the longitudinal direction is rigid (k=112.23)
Foundation in the direction of width is rigid (k=112.23)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 12.0 mm
Depth of influence zone = 9.42 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Granular Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load		Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
	new	change							
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge		Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
	new	change								
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1254.18	13.84	Yes
ULS	No	0.00	0.00	181.81	1254.18	14.50	Yes
SLS	Yes	0.00	0.00	173.56	582.47	29.80	Yes
SLS	No	0.00	0.00	173.56	582.47	29.80	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 3.67 m
Length of slip surface l_{sp} = 11.86 m

Design bearing capacity of found.soil R_d = 582.47 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 478.42 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 15.4 mm
Settlement of mid point of edge x - 2 = 15.4 mm
Settlement of mid point of edge y - 1 = 15.4 mm
Settlement of mid point of edge y - 2 = 15.4 mm
Settlement of foundation center point = 18.2 mm
Settlement of characteristic point = 16.0 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 40.94 MPa
Foundation in the longitudinal direction is rigid (k=91.59)
Foundation in the direction of width is rigid (k=91.59)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 16.0 mm
Depth of influence zone = 11.15 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 600.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil = 9.38 kN
Force transmitted by shear strength of SRC = 590.62 kN
Considered column perimeter u_0 = 1.00 m
Shear resistance at the column perimeter $v_{Ed,max}$ = 0.62 MPa
Resistance at the column perimeter $v_{Rd,max}$ = 2.94 MPa

Critical section without shear reinforcement

Force transmitted into found. soil = 187.50 kN
Force transmitted by shear strength of SRC = 412.50 kN
Distance of section from the column = 0.48 m
Section perimeter u_{cr} = 3.99 m
Shear stress at section v_{Ed} = 0.11 MPa
Shear resistance of section without shear reinforcement $v_{Rd,c}$ = 1.10 MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Spread footing verification

Input data

Project

Task : Holyhead
Part : Deep Bedrock - GranularDeposits - Foundation 4m x 4m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)									
Permanent design situation									
		Combination 1				Combination 2			
		Unfavourable		Favourable		Unfavourable		Favourable	
Permanent actions :	γ_G =	1.35	[–]	1.00	[–]	1.00	[–]	1.00	[–]

Partial factors for soil parameters (M)					
Permanent design situation					
		Combination 1		Combination 2	
Partial factor on internal friction :	γ_ϕ =	1.00	[–]	1.25	[–]
Partial factor on effective cohesion :	γ_c =	1.00	[–]	1.25	[–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00	[–]	1.40	[–]
Partial factor on unconfined strength :	γ_v =	1.00	[–]	1.40	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m³]	γ_{su} [kN/m³]	δ [°]
1	Superficial Granular Deposits		34.00	0.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Granular Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	34.00 °
Cohesion of soil :	c_{ef}	=	0.00 kPa
Oedometric modulus :	E_{oed}	=	31.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

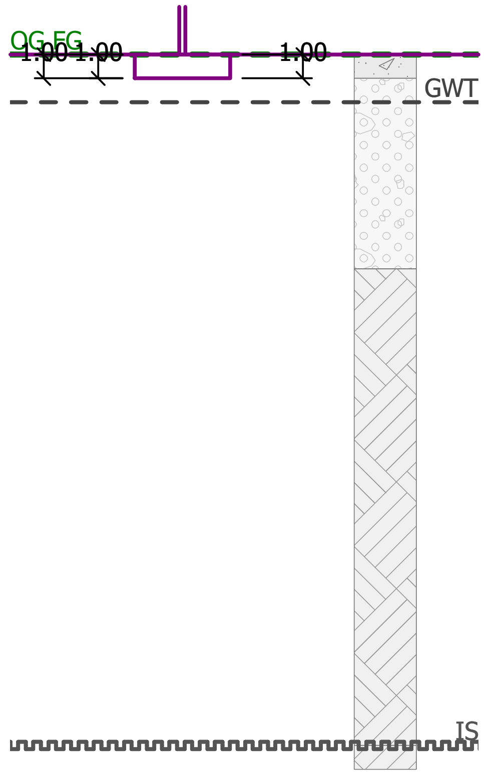
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation	Stage - analysis : 1 - 0



Geometry of structure

Foundation type: centric spread footing

Spread footing length	x	=	4.00	m
Spread footing width	y	=	4.00	m
Column width in the direction of x	c_x	=	0.25	m
Column width in the direction of y	c_y	=	0.25	m
Spread footing volume		=	16.00	m^3

Material of structure

Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

Concrete : C 20/25

Cylinder compressive strength	f_{ck}	=	20.00	MPa
Tensile strength	f_{ctm}	=	2.20	MPa
Elasticity modulus	E_{cm}	=	30000.00	MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

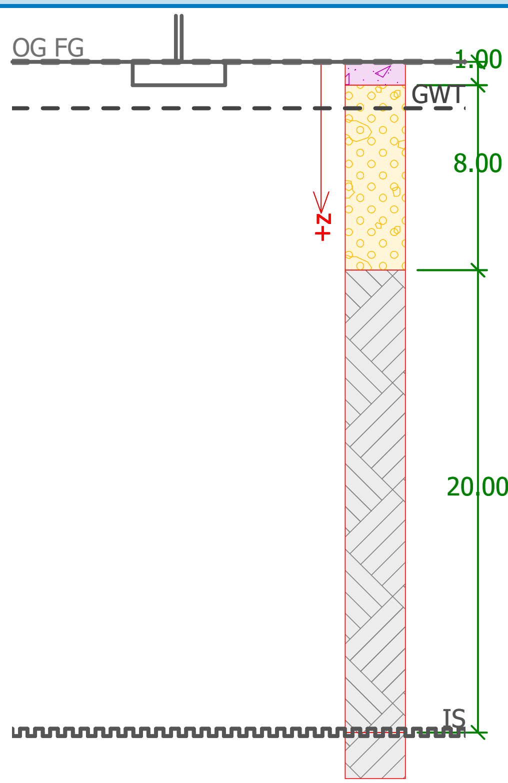
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	8.00	Superficial Granular Deposits	
3	20.00	Schist	
4	-	Schist	

Name : Profile and assignment	Stage - analysis : 1 - 0



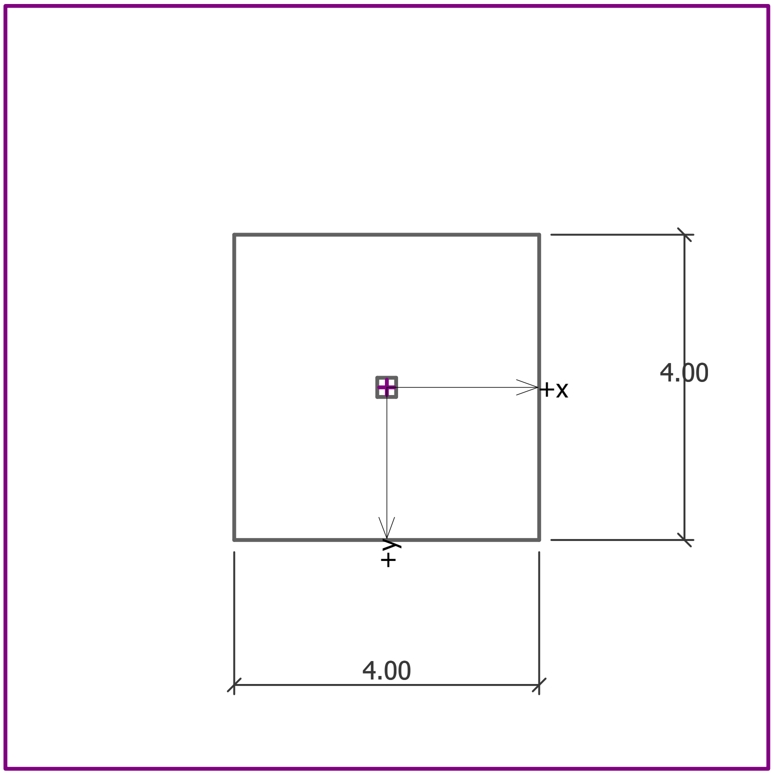
Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge	Stage - analysis : 1 - 0



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1529.02	11.35	Yes
ULS	No	0.00	0.00	181.81	1529.02	11.89	Yes
SLS	Yes	0.00	0.00	173.56	693.03	25.04	Yes
SLS	No	0.00	0.00	173.56	693.03	25.04	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 7.33 m
Length of slip surface l_{sp} = 23.72 m

Design bearing capacity of found.soil R_d = 693.03 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

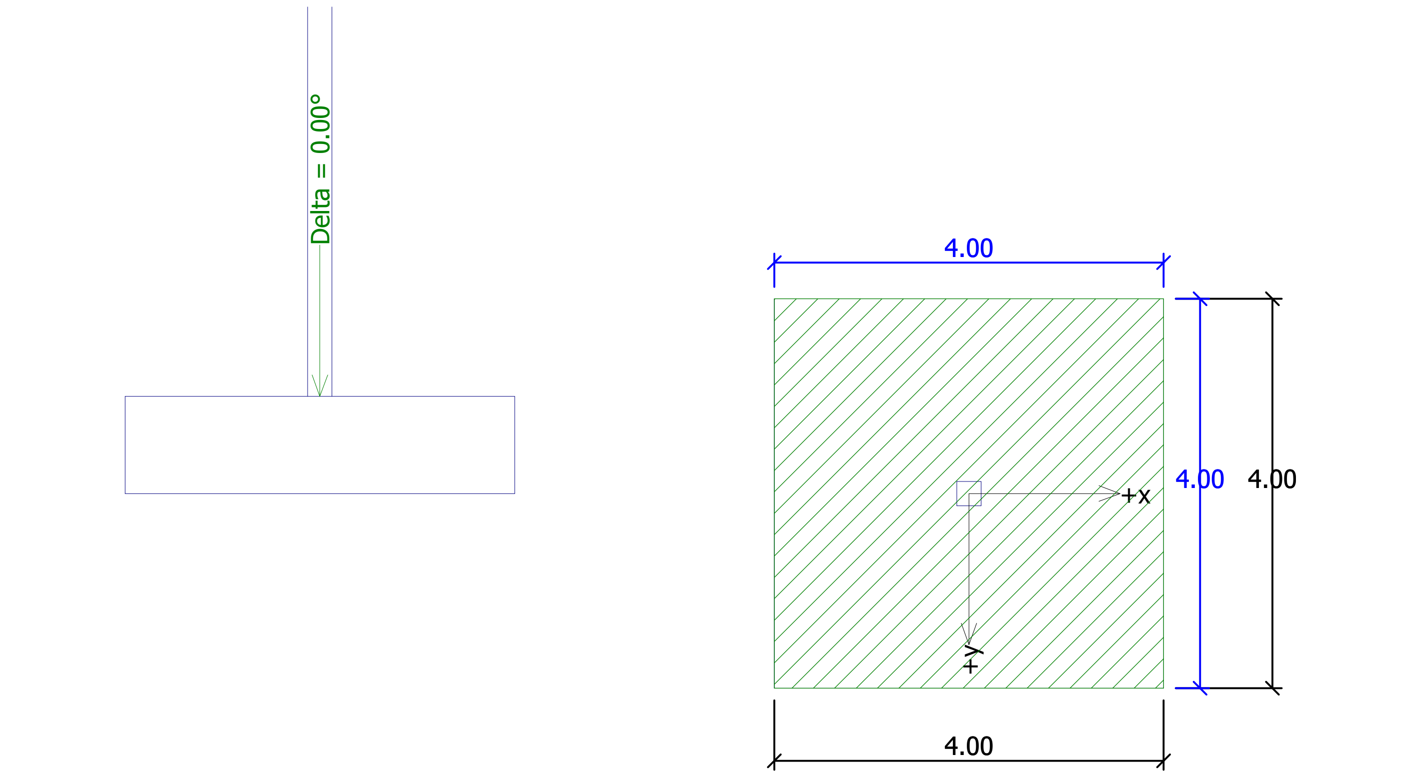
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 1893.39 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap.	Stage - analysis : 1 - 1



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 10.0 mm
Settlement of mid point of edge x - 2 = 10.0 mm
Settlement of mid point of edge y - 1 = 10.0 mm
Settlement of mid point of edge y - 2 = 10.0 mm
Settlement of foundation center point = 15.9 mm
Settlement of characteristic point = 11.3 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 32.27 MPa
Foundation in the longitudinal direction is rigid (k=14.52)
Foundation in the direction of width is rigid (k=14.52)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

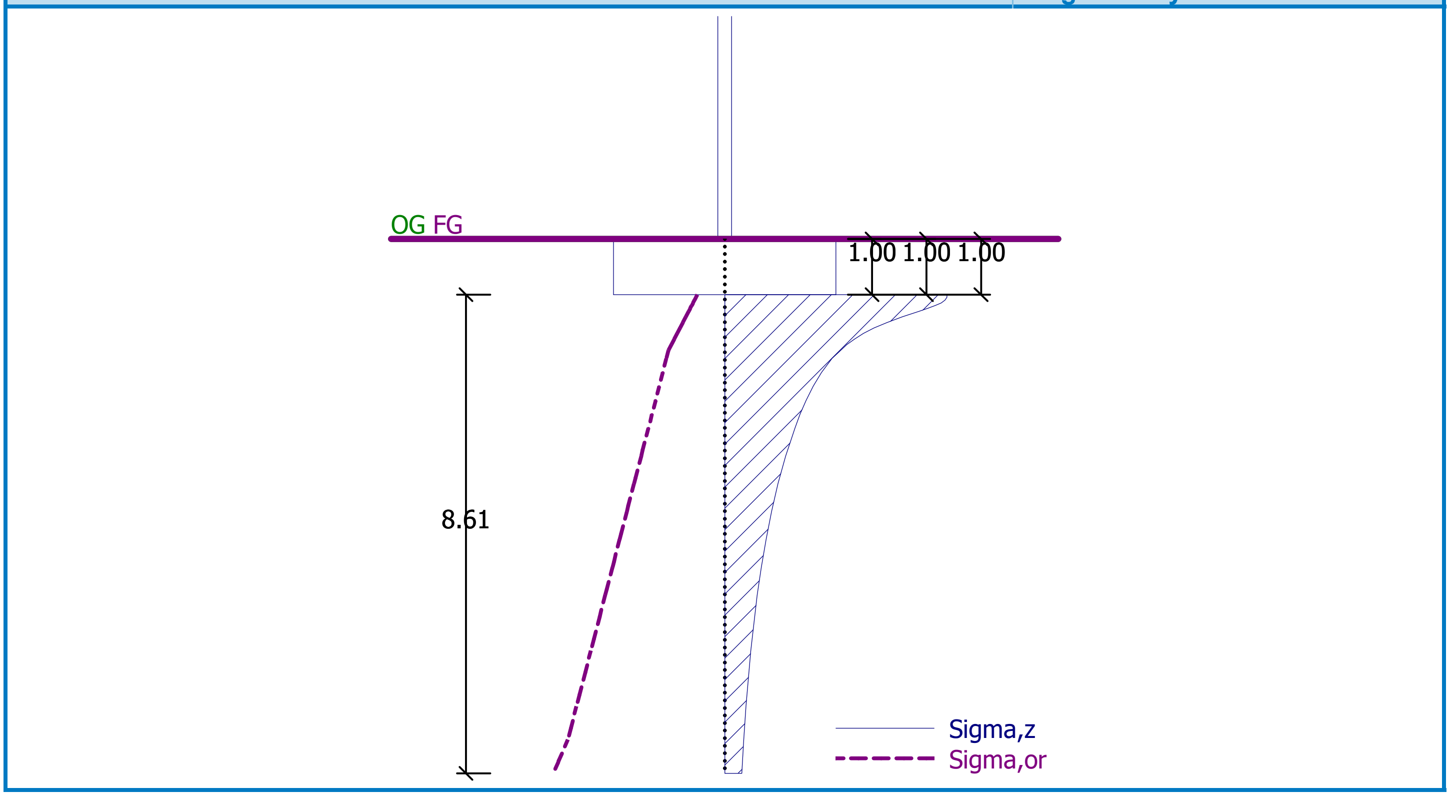
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 11.3 mm
Depth of influence zone = 8.61 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement **Stage - analysis : 1 - 1**



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Granular Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1529.02	11.35	Yes
ULS	No	0.00	0.00	181.81	1529.02	11.89	Yes
SLS	Yes	0.00	0.00	173.56	693.03	25.04	Yes
SLS	No	0.00	0.00	173.56	693.03	25.04	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 7.33 m
Length of slip surface l_{sp} = 23.72 m

Design bearing capacity of found.soil R_d = 693.03 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 1893.39 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 16.0 mm
Settlement of mid point of edge x - 2 = 16.0 mm
Settlement of mid point of edge y - 1 = 16.0 mm
Settlement of mid point of edge y - 2 = 16.0 mm
Settlement of foundation center point = 21.9 mm
Settlement of characteristic point = 17.3 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 39.69 MPa
Foundation in the longitudinal direction is rigid (k=11.81)
Foundation in the direction of width is rigid (k=11.81)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 17.3 mm
Depth of influence zone = 11.22 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	8.00	Superficial Granular Deposits	
3	20.00	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1529.02	11.35	Yes
ULS	No	0.00	0.00	181.81	1529.02	11.89	Yes
SLS	Yes	0.00	0.00	173.56	693.03	25.04	Yes
SLS	No	0.00	0.00	173.56	693.03	25.04	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 7.33 m
Length of slip surface l_{sp} = 23.72 m

Design bearing capacity of found.soil R_d = 693.03 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 1893.39 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 20.0 mm
Settlement of mid point of edge x - 2 = 20.0 mm
Settlement of mid point of edge y - 1 = 20.0 mm
Settlement of mid point of edge y - 2 = 20.0 mm
Settlement of foundation center point = 26.0 mm
Settlement of characteristic point = 21.4 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 45.50 MPa
Foundation in the longitudinal direction is rigid (k=10.30)
Foundation in the direction of width is rigid (k=10.30)

Verification of load eccentricity

Max. excentricity in direction of base length	e_x	=	0.000<0.333
Max. eccentricity in direction of base width	e_y	=	0.000<0.333
Max. overall eccentricity	e_t	=	0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:

Foundation settlement = 21.4 mm
Depth of influence zone = 12.63 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter	=	16.0	mm
Number of bars	=	11	
Reinforcement cover	=	40.0	mm
Cross-section width	=	4.00	m
Cross-section depth	=	1.00	m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}

Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter	=	16.0	mm
Number of bars	=	11	
Reinforcement cover	=	40.0	mm
Cross-section width	=	4.00	m
Cross-section depth	=	1.00	m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}

Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 2400.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil		=	9.38	kN
Force transmitted by shear strength of SRC		=	2390.62	kN
Considered column perimeter	u_0	=	1.00	m
Shear resistance at the column perimeter	$v_{Ed,max}$	=	2.51	MPa
Resistance at the column perimeter	$v_{Rd,max}$	=	2.94	MPa

Critical section without shear reinforcement

Force transmitted into found. soil		=	356.61	kN
Force transmitted by shear strength of SRC		=	2043.39	kN
Distance of section from the column		=	0.71	m
Section perimeter	u_{cr}	=	5.49	m
Shear stress at section	v_{Ed}	=	0.39	MPa
Shear resistance of section without shear reinforcement	$v_{Rd,c}$	=	0.74	MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Spread footing verification

Input data

Project

Task : Holyhead
Part : Shallow Bedrock - Cohesive Deposits - Foundation 2m x 2m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)									
Permanent design situation									
		Combination 1				Combination 2			
		Unfavourable		Favourable		Unfavourable		Favourable	
Permanent actions :	γ_G =	1.35	[–]	1.00	[–]	1.00	[–]	1.00	[–]

Partial factors for soil parameters (M)					
Permanent design situation					
		Combination 1		Combination 2	
Partial factor on internal friction :	γ_ϕ =	1.00	[–]	1.25	[–]
Partial factor on effective cohesion :	γ_c =	1.00	[–]	1.25	[–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00	[–]	1.40	[–]
Partial factor on unconfined strength :	γ_v =	1.00	[–]	1.40	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Superficial Cohesive Deposits		0.00	50.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Cohesive Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	0.00 °
Cohesion of soil :	c_{ef}	=	50.00 kPa
Oedometric modulus :	E_{oed}	=	9.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

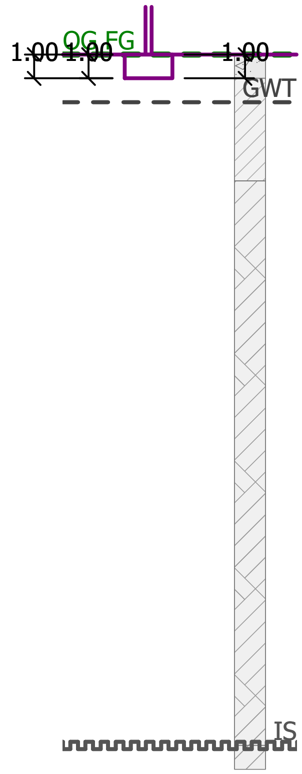
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation	Stage - analysis : 1 - 0



Geometry of structure

Foundation type: centric spread footing

Spread footing length	x	=	2.00	m
Spread footing width	y	=	2.00	m
Column width in the direction of x	c_x	=	0.25	m
Column width in the direction of y	c_y	=	0.25	m
Spread footing volume		=	4.00	m^3

Material of structure

Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

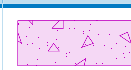
Concrete : C 20/25

Cylinder compressive strength	f_{ck}	=	20.00	MPa
Tensile strength	f_{ctm}	=	2.20	MPa
Elasticity modulus	E_{cm}	=	30000.00	MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

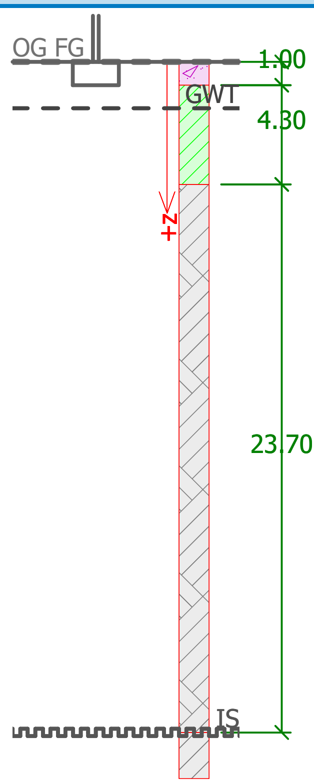
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	4.30	Superficial Cohesive Deposits	
3	23.70	Schist	
4	-	Schist	

Name : Profile and assignment | **Stage - analysis : 1 - 0**



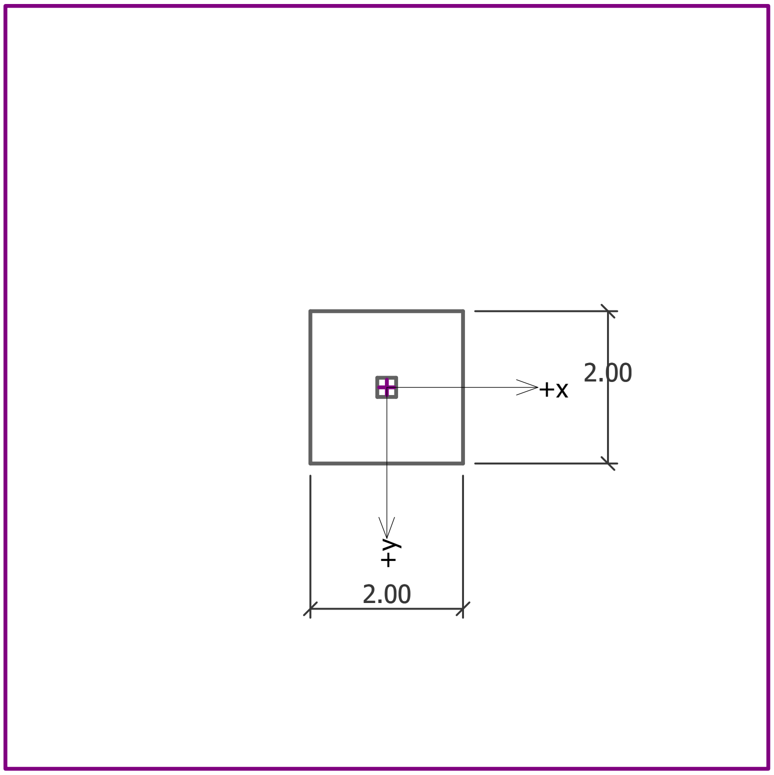
Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		ULS	Design	600.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge	Stage - analysis : 1 - 0



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 1.42 m
Length of slip surface l_{sp} = 3.00 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

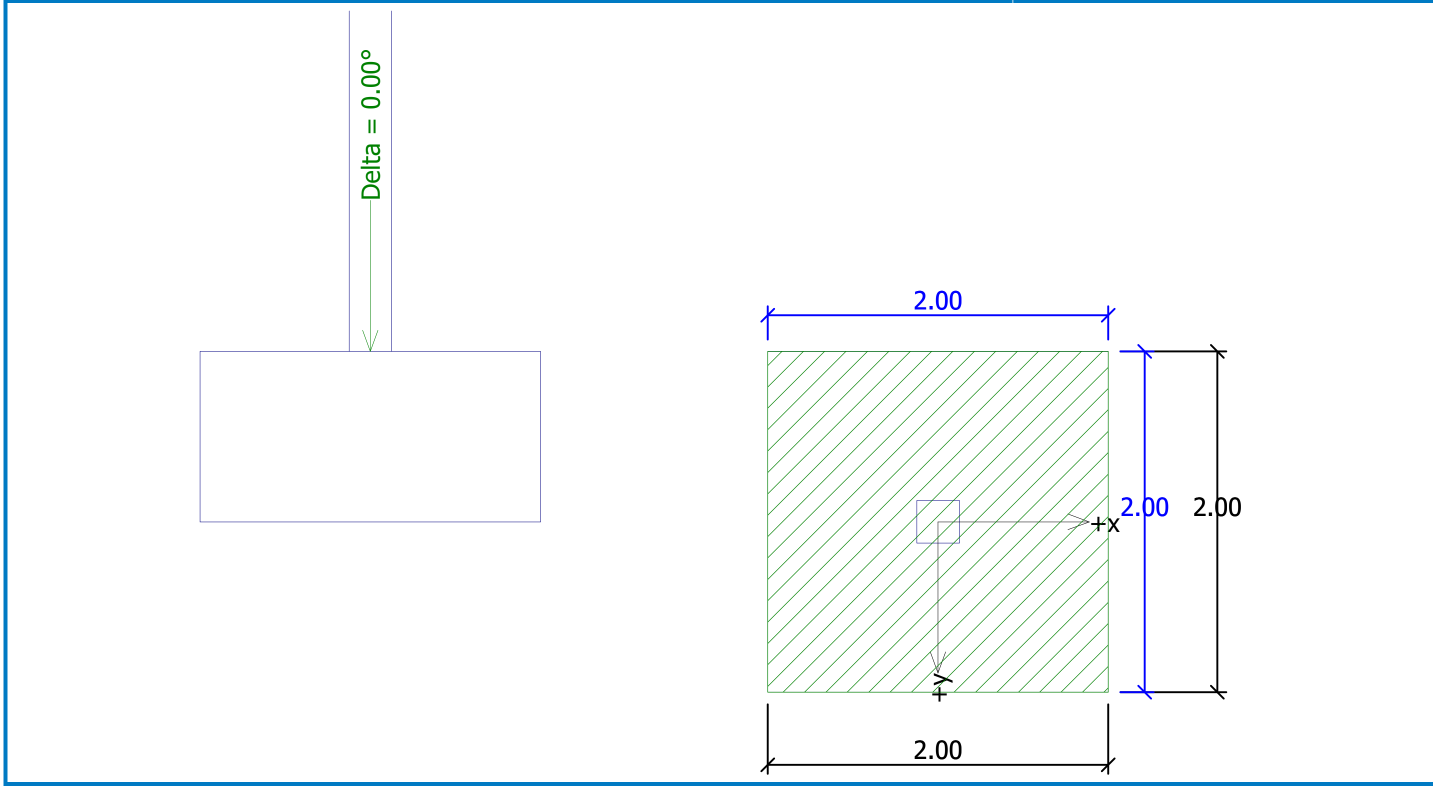
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 210.14 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap. **Stage - analysis : 1 - 1**



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 16.6 mm
Settlement of mid point of edge x - 2 = 16.6 mm
Settlement of mid point of edge y - 1 = 16.6 mm
Settlement of mid point of edge y - 2 = 16.6 mm
Settlement of foundation center point = 25.9 mm
Settlement of characteristic point = 18.7 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 14.98 MPa
Foundation in the longitudinal direction is rigid (k=250.32)
Foundation in the direction of width is rigid (k=250.32)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

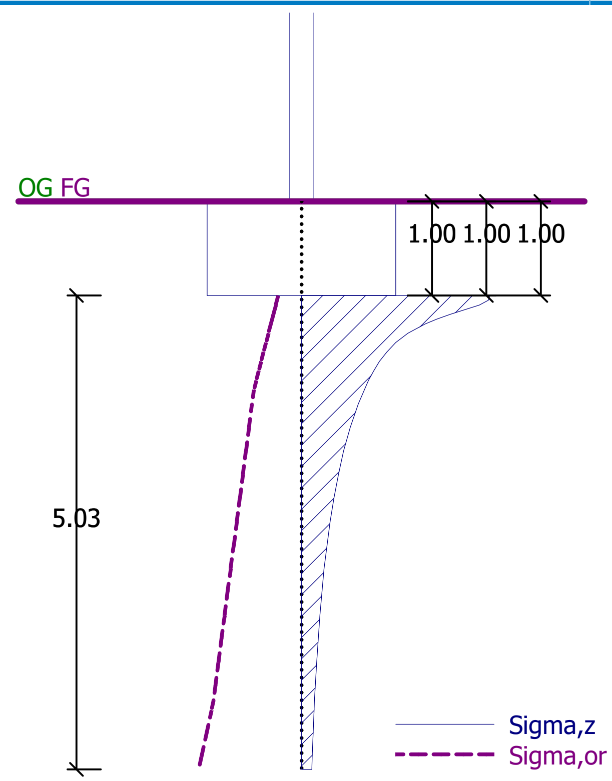
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 18.7 mm
Depth of influence zone = 5.03 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement	Stage - analysis : 1 - 1



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Cohesive Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 1.42 m
Length of slip surface l_{sp} = 3.00 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 210.14 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 29.9 mm
Settlement of mid point of edge x - 2 = 29.9 mm
Settlement of mid point of edge y - 1 = 29.9 mm
Settlement of mid point of edge y - 2 = 29.9 mm
Settlement of foundation center point = 39.2 mm
Settlement of characteristic point = 32.0 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 38.42 MPa
Foundation in the longitudinal direction is rigid (k=97.62)
Foundation in the direction of width is rigid (k=97.62)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 32.0 mm
Depth of influence zone = 8.77 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Cohesive Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 1.42 m
Length of slip surface l_{sp} = 3.00 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 210.14 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 38.9 mm
Settlement of mid point of edge x - 2 = 38.9 mm
Settlement of mid point of edge y - 1 = 38.9 mm
Settlement of mid point of edge y - 2 = 38.9 mm
Settlement of foundation center point = 48.3 mm
Settlement of characteristic point = 41.0 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 50.15 MPa
Foundation in the longitudinal direction is rigid (k=74.78)
Foundation in the direction of width is rigid (k=74.78)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 41.0 mm
Depth of influence zone = 10.49 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 600.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil		=	9.38	kN
Force transmitted by shear strength of SRC		=	590.62	kN
Considered column perimeter	u_0	=	1.00	m
Shear resistance at the column perimeter	$v_{Ed,max}$	=	0.62	MPa
Resistance at the column perimeter	$v_{Rd,max}$	=	2.94	MPa

Critical section without shear reinforcement

Force transmitted into found. soil		=	187.50	kN
Force transmitted by shear strength of SRC		=	412.50	kN
Distance of section from the column		=	0.48	m
Section perimeter	u_{cr}	=	3.99	m
Shear stress at section	v_{Ed}	=	0.11	MPa
Shear resistance of section without shear reinforcement	$v_{Rd,c}$	=	1.10	MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Spread footing verification

Input data

Project

Task : Holyhead
Part : Shallow Bedrock - Cohesive Deposits - Foundation 4m x 4m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)									
Permanent design situation									
		Combination 1				Combination 2			
		Unfavourable		Favourable		Unfavourable		Favourable	
Permanent actions :	γ_G =	1.35	[–]	1.00	[–]	1.00	[–]	1.00	[–]

Partial factors for soil parameters (M)					
Permanent design situation					
		Combination 1		Combination 2	
Partial factor on internal friction :	γ_ϕ =	1.00	[–]	1.25	[–]
Partial factor on effective cohesion :	γ_c =	1.00	[–]	1.25	[–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00	[–]	1.40	[–]
Partial factor on unconfined strength :	γ_v =	1.00	[–]	1.40	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Superficial Cohesive Deposits		0.00	50.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Cohesive Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	0.00 °
Cohesion of soil :	c_{ef}	=	50.00 kPa
Oedometric modulus :	E_{oed}	=	9.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

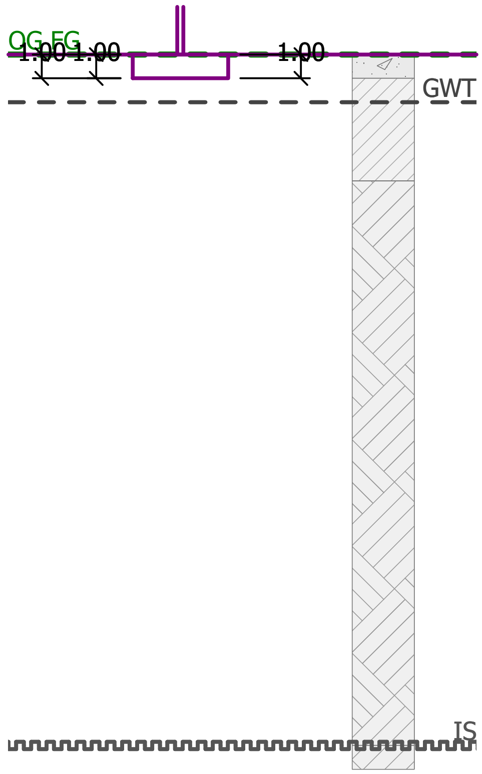
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation	Stage - analysis : 1 - 0



Geometry of structure

Foundation type: centric spread footing

Spread footing length	x	=	4.00	m
Spread footing width	y	=	4.00	m
Column width in the direction of x	c_x	=	0.25	m
Column width in the direction of y	c_y	=	0.25	m
Spread footing volume		=	16.00	m^3

Material of structure

Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

Concrete : C 20/25

Cylinder compressive strength	f_{ck}	=	20.00	MPa
Tensile strength	f_{ctm}	=	2.20	MPa
Elasticity modulus	E_{cm}	=	30000.00	MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

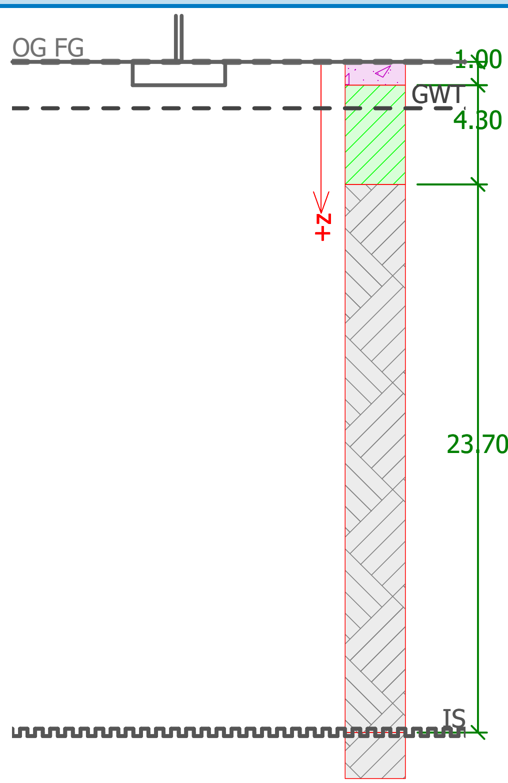
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	4.30	Superficial Cohesive Deposits	
3	23.70	Schist	
4	-	Schist	

Name : Profile and assignment **Stage - analysis : 1 - 0**



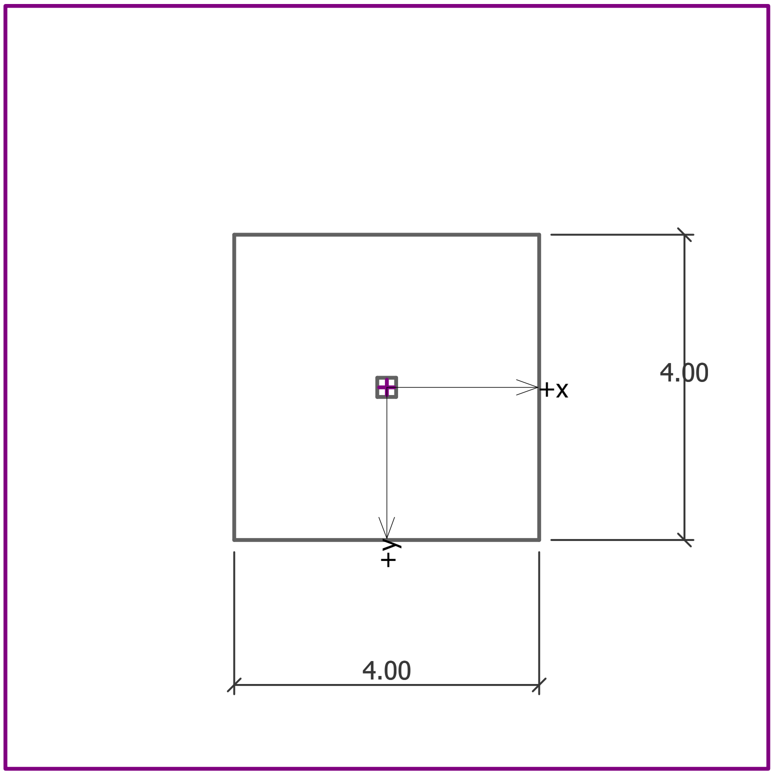
Load

No.	Load		Name	Type	N	M_x	M_y	H_x	H_y
	new	change			[kN]	[kNm]	[kNm]	[kN]	[kN]
1	YES		ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge		Name	x_s	y_s	x	y	q	α	h
	new	change		[m]	[m]	[m]	[m]	[kPa]	[°]	[m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge **Stage - analysis : 1 - 0**



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 2.83 m
Length of slip surface l_{sp} = 6.01 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

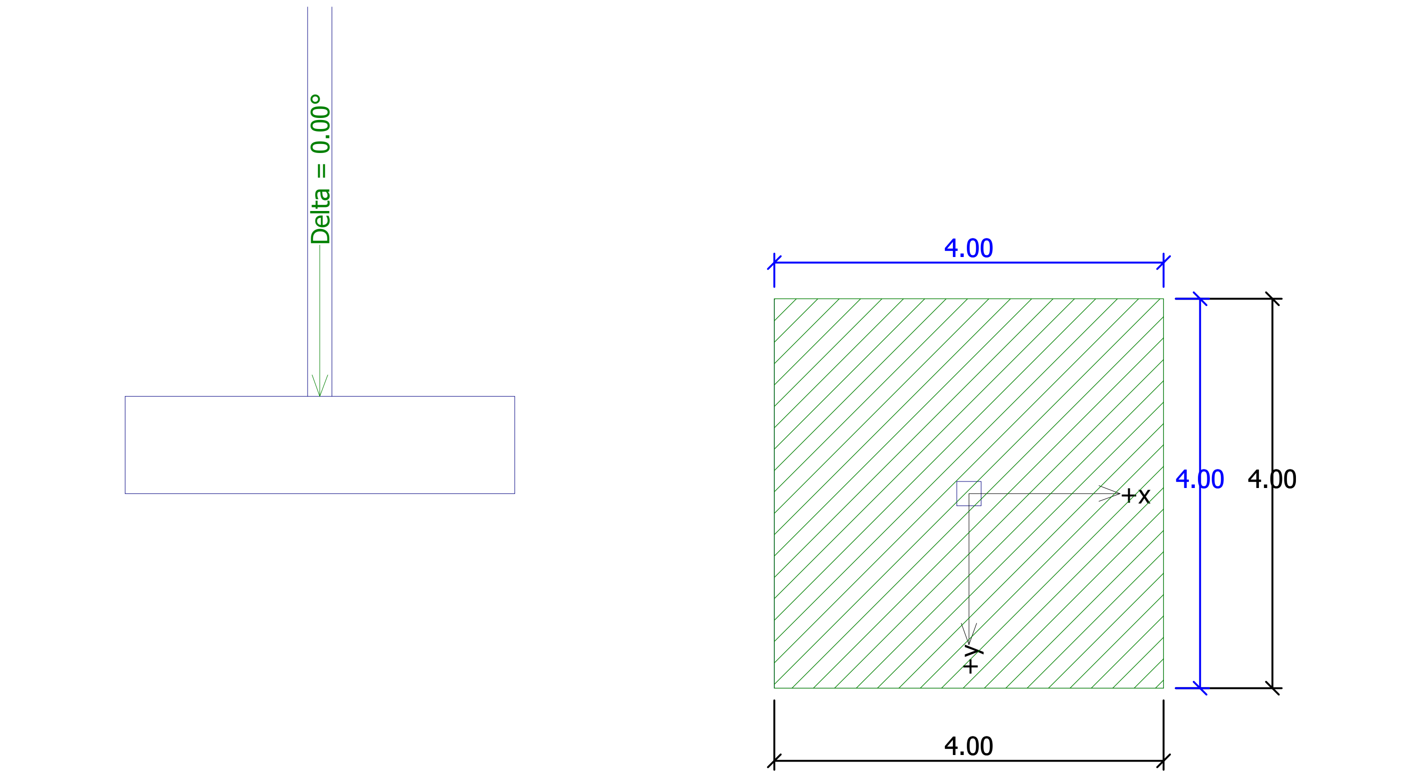
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 820.28 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap. | **Stage - analysis : 1 - 1**



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 26.3 mm
Settlement of mid point of edge x - 2 = 26.3 mm
Settlement of mid point of edge y - 1 = 26.3 mm
Settlement of mid point of edge y - 2 = 26.3 mm
Settlement of foundation center point = 44.9 mm
Settlement of characteristic point = 31.0 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 35.99 MPa
Foundation in the longitudinal direction is rigid (k=13.03)
Foundation in the direction of width is rigid (k=13.03)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

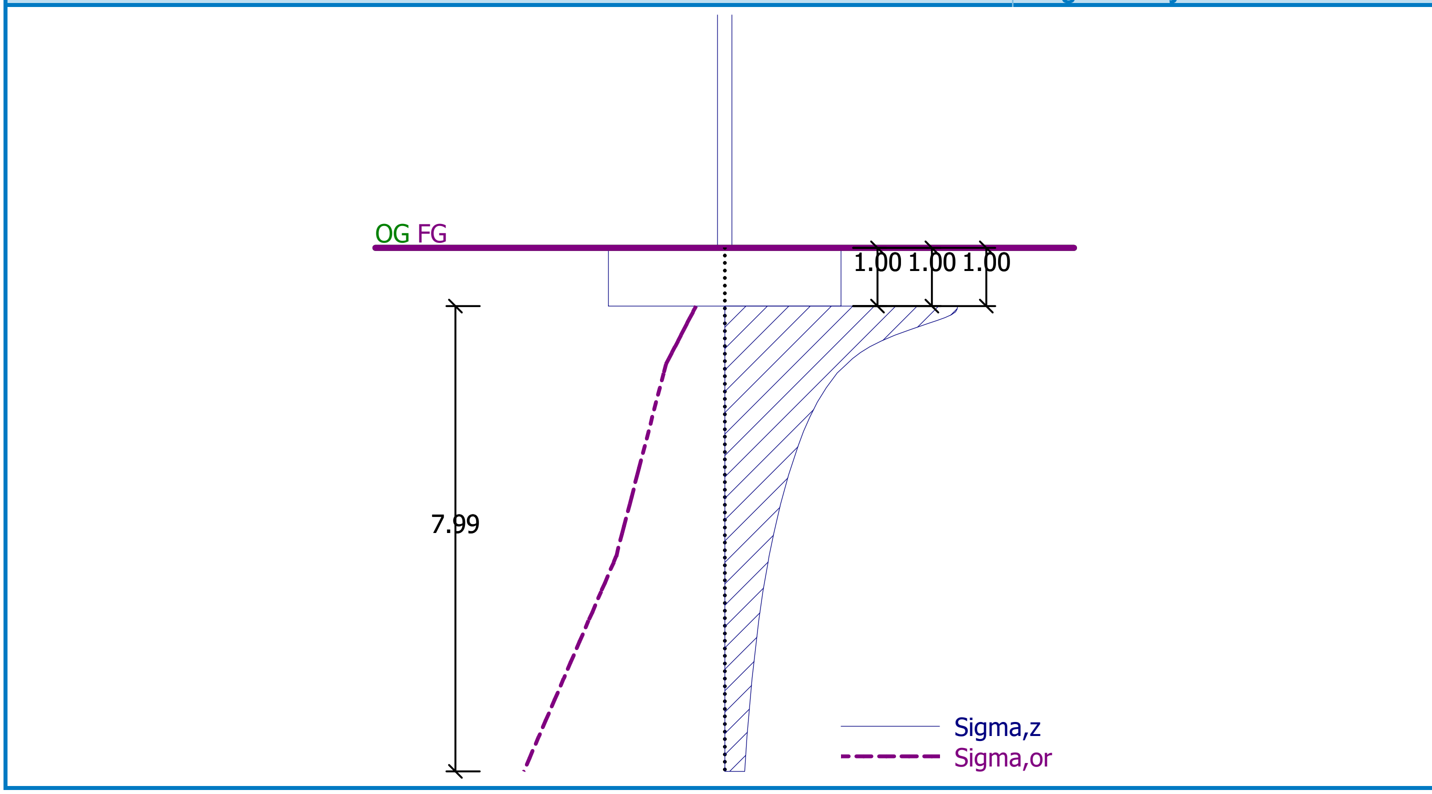
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 31.0 mm
Depth of influence zone = 7.99 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement	**Stage - analysis : 1 - 1**



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Cohesive Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 2.83 m
Length of slip surface l_{sp} = 6.01 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 820.28 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 39.8 mm
Settlement of mid point of edge x - 2 = 39.8 mm
Settlement of mid point of edge y - 1 = 39.8 mm
Settlement of mid point of edge y - 2 = 39.8 mm
Settlement of foundation center point = 58.4 mm
Settlement of characteristic point = 44.4 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 48.85 MPa
Foundation in the longitudinal direction is rigid (k=9.60)
Foundation in the direction of width is rigid (k=9.60)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 44.4 mm
Depth of influence zone = 10.58 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Cohesive Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load		Name	Type	N	M_x	M_y	H_x	H_y
	new	change			[kN]	[kNm]	[kNm]	[kN]	[kN]
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge		Name	x_s	y_s	x	y	q	α	h
	new	change		[m]	[m]	[m]	[m]	[kPa]	[°]	[m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	276.08	62.87	Yes
ULS	No	0.00	0.00	181.81	276.08	65.85	Yes
SLS	Yes	0.00	0.00	173.56	224.66	77.25	Yes
SLS	No	0.00	0.00	173.56	224.66	77.25	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 2.83 m
Length of slip surface l_{sp} = 6.01 m

Design bearing capacity of found.soil R_d = 224.66 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 820.28 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 48.8 mm
Settlement of mid point of edge x - 2 = 48.8 mm
Settlement of mid point of edge y - 1 = 48.8 mm
Settlement of mid point of edge y - 2 = 48.8 mm
Settlement of foundation center point = 67.4 mm
Settlement of characteristic point = 53.5 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 54.57 MPa
Foundation in the longitudinal direction is rigid (k=8.59)
Foundation in the direction of width is rigid (k=8.59)

Verification of load eccentricity

Max. excentricity in direction of base length	e_x	=	0.000<0.333
Max. eccentricity in direction of base width	e_y	=	0.000<0.333
Max. overall eccentricity	e_t	=	0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 53.5 mm
Depth of influence zone = 11.99 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter	=	16.0	mm
Number of bars	=	11	
Reinforcement cover	=	40.0	mm
Cross-section width	=	4.00	m
Cross-section depth	=	1.00	m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter	=	16.0	mm
Number of bars	=	11	
Reinforcement cover	=	40.0	mm
Cross-section width	=	4.00	m
Cross-section depth	=	1.00	m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 2400.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil		=	9.38	kN
Force transmitted by shear strength of SRC		=	2390.62	kN
Considered column perimeter	u_0	=	1.00	m
Shear resistance at the column perimeter	$v_{Ed,max}$	=	2.51	MPa
Resistance at the column perimeter	$v_{Rd,max}$	=	2.94	MPa

Critical section without shear reinforcement

Force transmitted into found. soil		=	356.61	kN
Force transmitted by shear strength of SRC		=	2043.39	kN
Distance of section from the column		=	0.71	m
Section perimeter	u_{cr}	=	5.49	m
Shear stress at section	v_{Ed}	=	0.39	MPa
Shear resistance of section without shear reinforcement	$v_{Rd,c}$	=	0.74	MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Spread footing verification

Input data

Project

Task : Holyhead
Part : Shallow Bedrock - Granular Deposits - Foundation 2m x 2m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)									
Permanent design situation									
		Combination 1				Combination 2			
		Unfavourable		Favourable		Unfavourable		Favourable	
Permanent actions :	γ_G =	1.35	[–]	1.00	[–]	1.00	[–]	1.00	[–]

Partial factors for soil parameters (M)					
Permanent design situation					
		Combination 1		Combination 2	
Partial factor on internal friction :	γ_ϕ =	1.00	[–]	1.25	[–]
Partial factor on effective cohesion :	γ_c =	1.00	[–]	1.25	[–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00	[–]	1.40	[–]
Partial factor on unconfined strength :	γ_v =	1.00	[–]	1.40	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Superficial Granular Deposits		34.00	0.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Granular Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	34.00 °
Cohesion of soil :	c_{ef}	=	0.00 kPa
Oedometric modulus :	E_{oed}	=	31.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

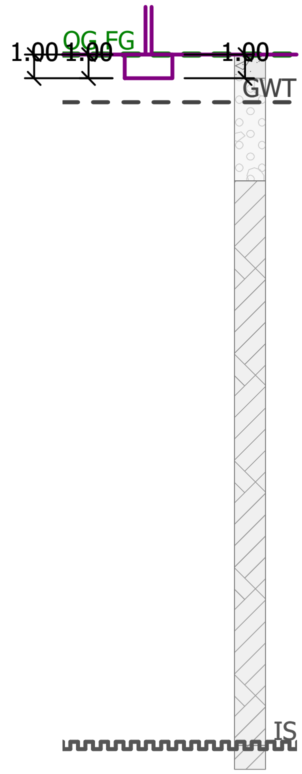
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation | **Stage - analysis : 1 - 0**



Geometry of structure

Foundation type: centric spread footing

Spread footing length x = 2.00 m
Spread footing width y = 2.00 m
Column width in the direction of x c_x = 0.25 m
Column width in the direction of y c_y = 0.25 m
Spread footing volume = 4.00 m^3

Material of structure

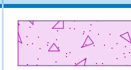
Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

Concrete : C 20/25
Cylinder compressive strength f_{ck} = 20.00 MPa
Tensile strength f_{ctm} = 2.20 MPa
Elasticity modulus E_{cm} = 30000.00 MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

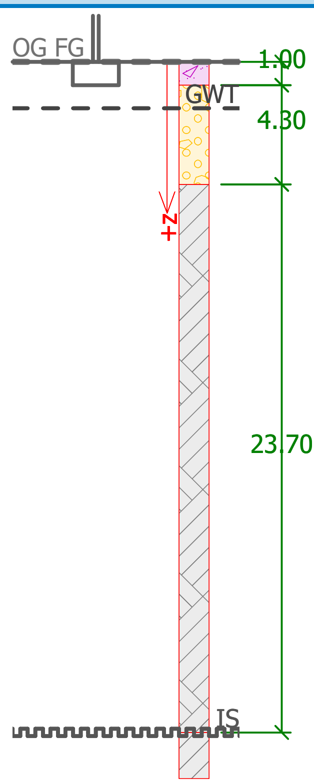
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	4.30	Superficial Granular Deposits	
3	23.70	Schist	
4	-	Schist	

Name : Profile and assignment | **Stage - analysis : 1 - 0**



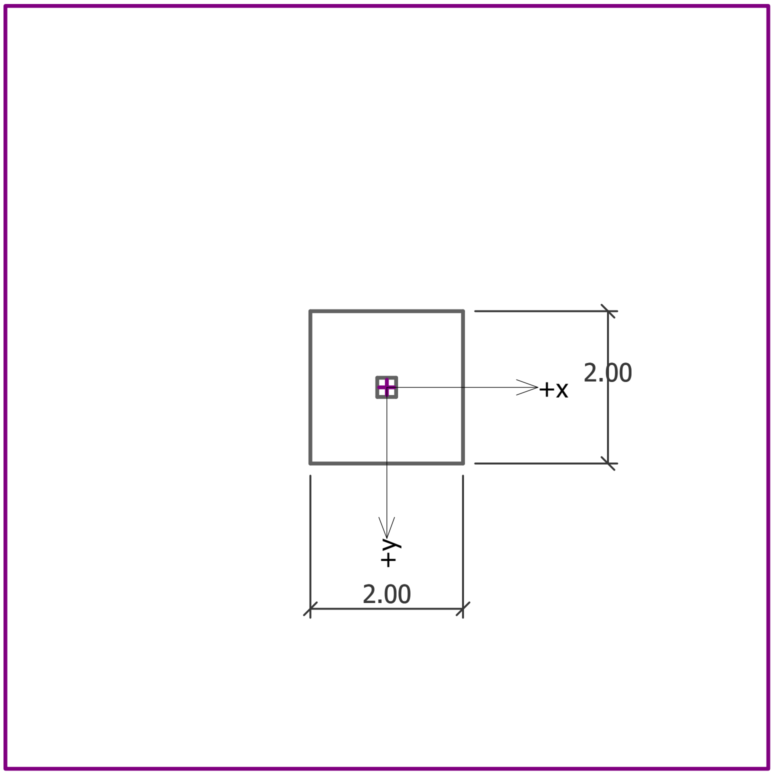
Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		ULS	Design	600.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge **Stage - analysis : 1 - 0**



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1254.18	13.84	Yes
ULS	No	0.00	0.00	181.81	1254.18	14.50	Yes
SLS	Yes	0.00	0.00	173.56	582.47	29.80	Yes
SLS	No	0.00	0.00	173.56	582.47	29.80	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 3.67 m
Length of slip surface l_{sp} = 11.86 m

Design bearing capacity of found.soil R_d = 582.47 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

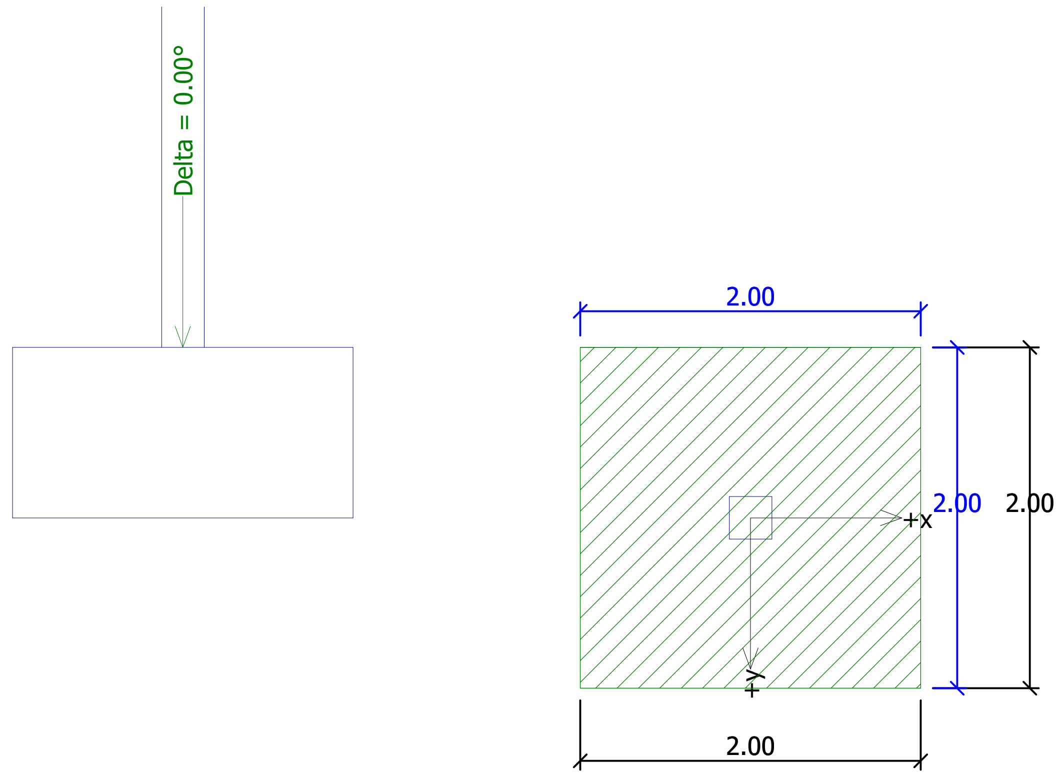
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 478.42 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap. **Stage - analysis : 1 - 1**



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 4.8 mm
Settlement of mid point of edge x - 2 = 4.8 mm
Settlement of mid point of edge y - 1 = 4.8 mm
Settlement of mid point of edge y - 2 = 4.8 mm
Settlement of foundation center point = 7.5 mm
Settlement of characteristic point = 5.5 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 33.24 MPa
Foundation in the longitudinal direction is rigid (k=112.83)
Foundation in the direction of width is rigid (k=112.83)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

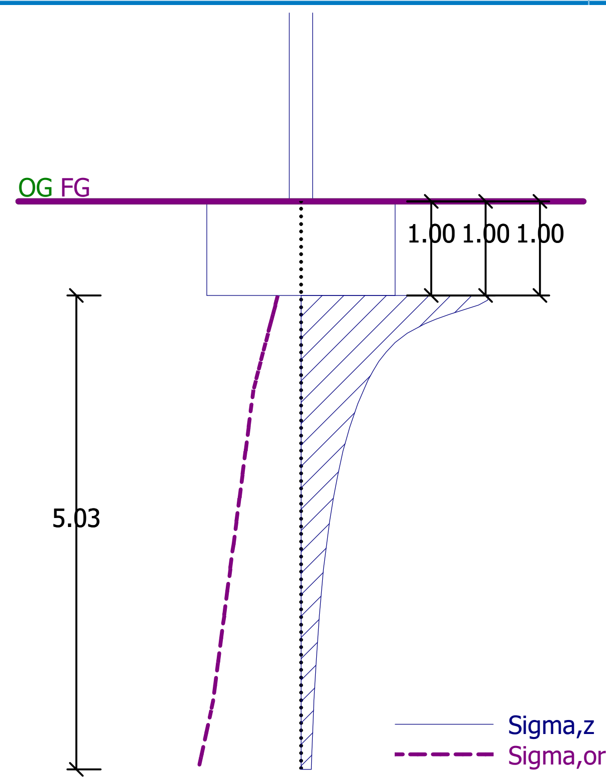
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 5.5 mm
Depth of influence zone = 5.03 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement	Stage - analysis : 1 - 1



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Granular Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1254.18	13.84	Yes
ULS	No	0.00	0.00	181.81	1254.18	14.50	Yes
SLS	Yes	0.00	0.00	173.56	582.47	29.80	Yes
SLS	No	0.00	0.00	173.56	582.47	29.80	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing $G = 94.25$ kN
Computed weight of overburden $Z = 0.00$ kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface $z_{sp} = 3.67$ m
Length of slip surface $l_{sp} = 11.86$ m

Design bearing capacity of found.soil $R_d = 582.47$ kPa
Extreme contact stress $\sigma = 173.56$ kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length $e_x = 0.000<0.333$
Max. eccentricity in direction of base width $e_y = 0.000<0.333$
Max. overall eccentricity $e_t = 0.000<0.333$

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 478.42 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 9.0 mm
Settlement of mid point of edge x - 2 = 9.0 mm
Settlement of mid point of edge y - 1 = 9.0 mm
Settlement of mid point of edge y - 2 = 9.0 mm
Settlement of foundation center point = 11.7 mm
Settlement of characteristic point = 9.6 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 53.47 MPa
Foundation in the longitudinal direction is rigid (k=70.13)
Foundation in the direction of width is rigid (k=70.13)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 9.6 mm
Depth of influence zone = 8.77 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Granular Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	600.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	600.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	1254.18	13.84	Yes
ULS	No	0.00	0.00	181.81	1254.18	14.50	Yes
SLS	Yes	0.00	0.00	173.56	582.47	29.80	Yes
SLS	No	0.00	0.00	173.56	582.47	29.80	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 3.67 m
Length of slip surface l_{sp} = 11.86 m

Design bearing capacity of found.soil R_d = 582.47 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 10.14 kN

Horizontal bearing capacity R_{dh} = 478.42 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 94.25 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 11.9 mm
Settlement of mid point of edge x - 2 = 11.9 mm
Settlement of mid point of edge y - 1 = 11.9 mm
Settlement of mid point of edge y - 2 = 11.9 mm
Settlement of foundation center point = 14.6 mm
Settlement of characteristic point = 12.5 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 63.60 MPa
Foundation in the longitudinal direction is rigid (k=58.97)
Foundation in the direction of width is rigid (k=58.97)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 12.5 mm
Depth of influence zone = 10.49 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 2.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.12 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 600.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil		=	9.38	kN
Force transmitted by shear strength of SRC		=	590.62	kN
Considered column perimeter	u_0	=	1.00	m
Shear resistance at the column perimeter	$v_{Ed,max}$	=	0.62	MPa
Resistance at the column perimeter	$v_{Rd,max}$	=	2.94	MPa

Critical section without shear reinforcement

Force transmitted into found. soil		=	187.50	kN
Force transmitted by shear strength of SRC		=	412.50	kN
Distance of section from the column		=	0.48	m
Section perimeter	u_{cr}	=	3.99	m
Shear stress at section	v_{Ed}	=	0.11	MPa
Shear resistance of section without shear reinforcement	$v_{Rd,c}$	=	1.10	MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Spread footing verification

Input data

Project

Task : Holyhead
Part : Deep Bedrock - Shallow Deposits - Foundation 4m x 4m
Author : KN
Date : 8/27/2024

Settings

United Kingdom - EN 1997

Materials and standards

Concrete structures : EN 1992-1-1 (EC2)
Coefficients EN 1992-1-1 : standard

Settlement

Analysis method : Analysis using oedometric modulus
Restriction of influence zone : by percentage of Sigma,Or
Coeff. of restriction of influence zone : 10.0 [%]

Spread Footing

Analysis for drained conditions : EC 7-1 (EN 1997-1:2003)
Analysis of uplift : Standard
Allowable eccentricity : 0.333
Verification methodology : according to EN 1997
Design approach : 1 - reduction of actions and soil parameters

Partial factors on actions (A)									
Permanent design situation									
		Combination 1				Combination 2			
		Unfavourable		Favourable		Unfavourable		Favourable	
Permanent actions :	γ_G =	1.35	[–]	1.00	[–]	1.00	[–]	1.00	[–]

Partial factors for soil parameters (M)					
Permanent design situation					
		Combination 1		Combination 2	
Partial factor on internal friction :	γ_ϕ =	1.00	[–]	1.25	[–]
Partial factor on effective cohesion :	γ_c =	1.00	[–]	1.25	[–]
Partial factor on undrained shear strength :	γ_{cu} =	1.00	[–]	1.40	[–]
Partial factor on unconfined strength :	γ_v =	1.00	[–]	1.40	[–]

Basic soil parameters

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]	γ_{su} [kN/m^3]	δ [°]
1	Superficial Granular Deposits		34.00	0.00	20.00	10.00	
2	Schist		30.00	500.00	26.60	16.60	
3	Engineered fill		27.80	94.40	19.00	9.20	

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Superficial Granular Deposits

Unit weight :	γ	=	20.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	34.00 °
Cohesion of soil :	c_{ef}	=	0.00 kPa
Oedometric modulus :	E_{oed}	=	31.00 MPa
Saturated unit weight :	γ_{sat}	=	20.00 kN/m^3

Schist

Unit weight :	γ	=	26.60 kN/m^3
Angle of internal friction :	φ_{ef}	=	30.00 °
Cohesion of soil :	c_{ef}	=	500.00 kPa
Oedometric modulus :	E_{oed}	=	200.00 MPa
Saturated unit weight :	γ_{sat}	=	26.60 kN/m^3

Engineered fill

Unit weight :	γ	=	19.00 kN/m^3
Angle of internal friction :	φ_{ef}	=	27.80 °
Cohesion of soil :	c_{ef}	=	94.40 kPa
Oedometric modulus :	E_{oed}	=	4.00 MPa
Saturated unit weight :	γ_{sat}	=	19.20 kN/m^3

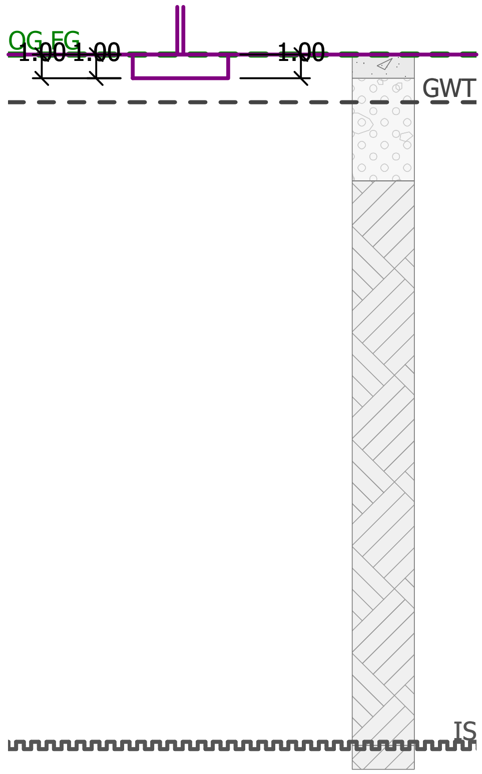
Foundation

Foundation type: centric spread footing

Depth from original ground surface	h_z	=	1.00	m
Depth of footing bottom	d	=	1.00	m
Foundation thickness	t	=	1.00	m
Incl. of finished grade	s_1	=	0.00	°
Incl. of footing bottom	s_2	=	0.00	°

Unit weight of soil above foundation = 20.00 kN/m^3

Name : Foundation | **Stage - analysis : 1 - 0**



Geometry of structure

Foundation type: centric spread footing

Spread footing length	x	=	4.00	m
Spread footing width	y	=	4.00	m
Column width in the direction of x	c_x	=	0.25	m
Column width in the direction of y	c_y	=	0.25	m
Spread footing volume		=	16.00	m^3

Material of structure

Unit weight γ = 23.56 kN/m^3
Analysis of concrete structures carried out according to the standard EN 1992-1-1 (EC2).

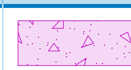
Concrete : C 20/25

Cylinder compressive strength	f_{ck}	=	20.00	MPa
Tensile strength	f_{ctm}	=	2.20	MPa
Elasticity modulus	E_{cm}	=	30000.00	MPa

Longitudinal steel : B500
Yield strength f_{yk} = 500.00 MPa

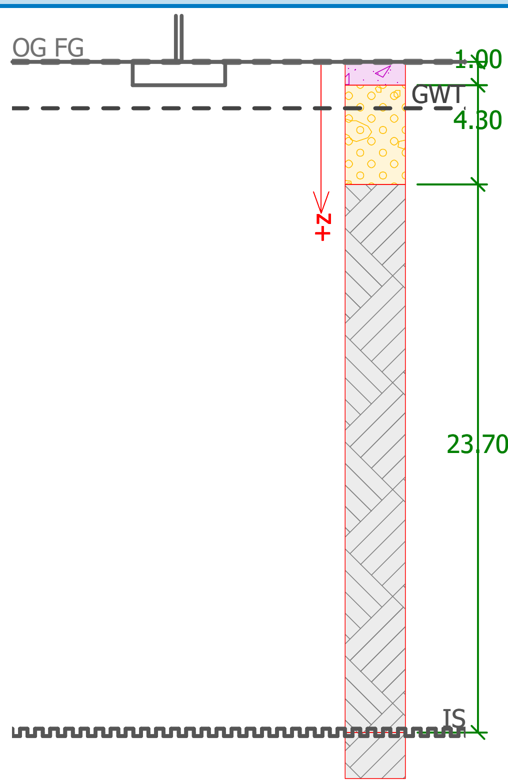
Transverse steel: B500
Yield strength f_{yk} = 500.00 MPa

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	

No.	Layer [m]	Assigned soil	Pattern
2	4.30	Superficial Granular Deposits	
3	23.70	Schist	
4	-	Schist	

Name : Profile and assignment	Stage - analysis : 1 - 0



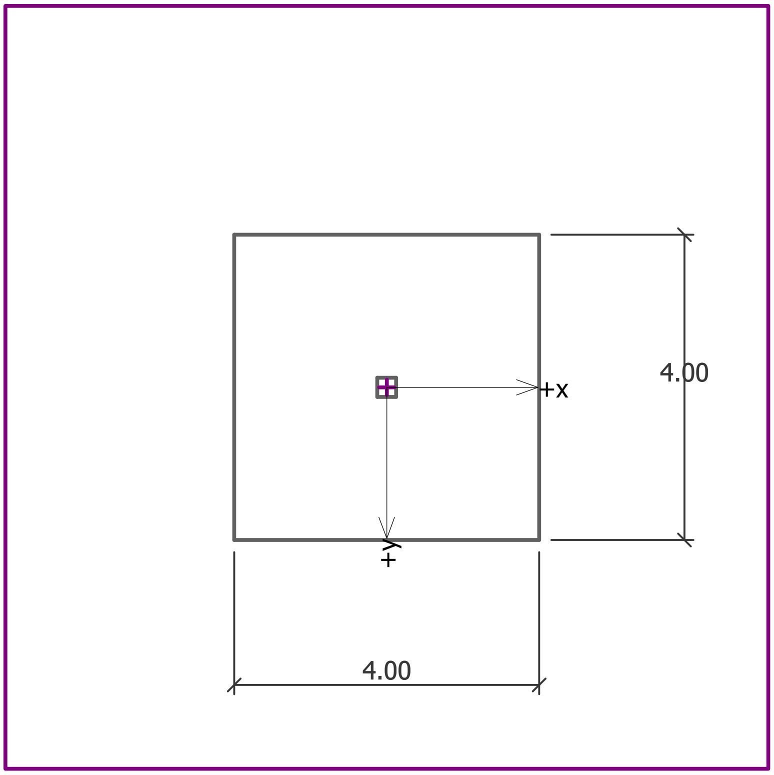
Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	YES		ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	YES		SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	YES		Slab loading	0.00	0.00	10.00	10.00	0.00	0.00	0.00

Name : Surcharge | **Stage - analysis : 1 - 0**



GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Global settings

Type of analysis : analysis for drained conditions

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 1)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	14172.46	1.22	Yes
ULS	No	0.00	0.00	181.81	14172.46	1.28	Yes
SLS	Yes	0.00	0.00	173.56	7026.51	2.47	Yes
SLS	No	0.00	0.00	173.56	7026.51	2.47	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle

Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 6.84 m
Length of slip surface l_{sp} = 21.44 m

Design bearing capacity of found.soil R_d = 7026.51 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

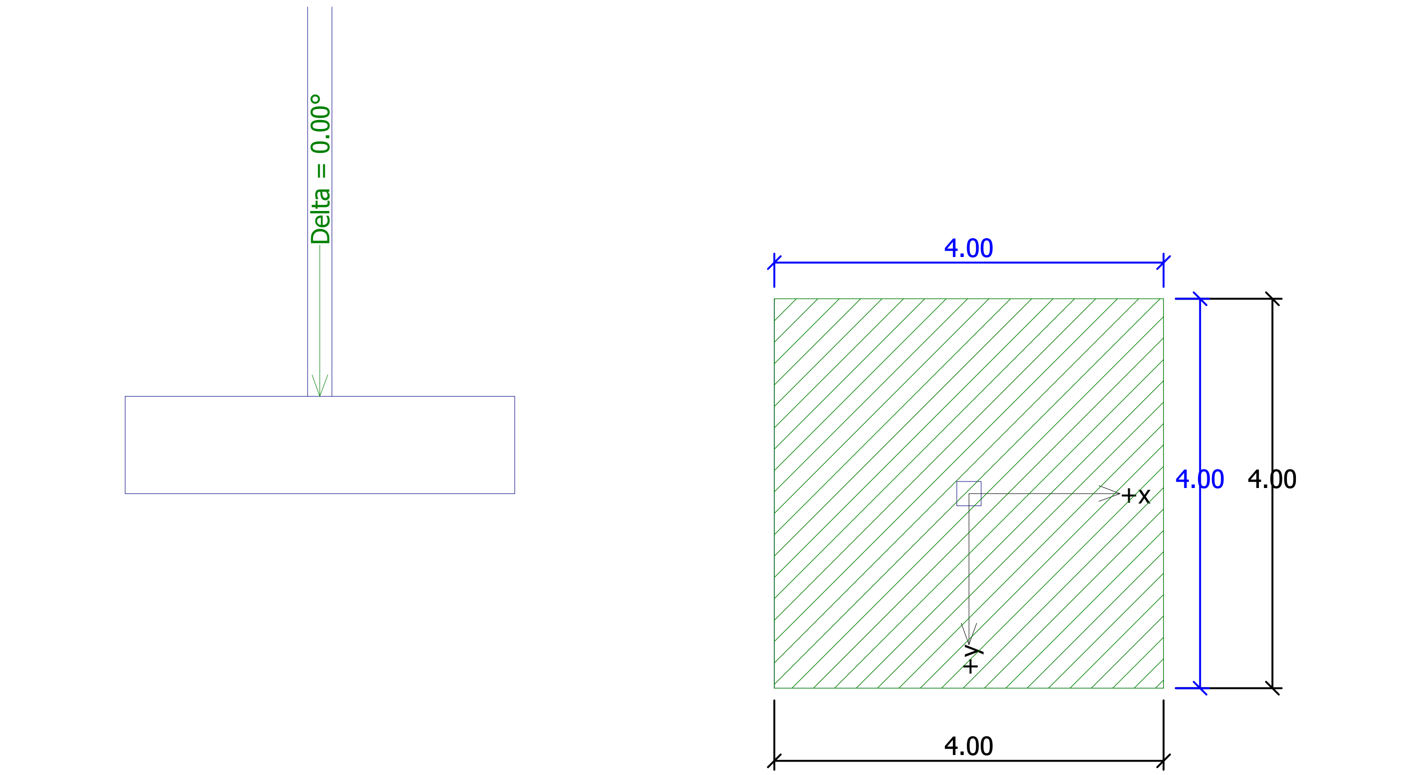
Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 1893.39 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Name : Bearing cap. | **Stage - analysis : 1 - 1**



Verification No. 1 (Stage of construction 1)

Settlement and rotation of foundation - input data

Analysis carried out for the load case No. 2.(SLS)
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing	G	=	377.01	kN
Computed weight of overburden	Z	=	0.00	kN

Settlement of mid point of edge x - 1	=	7.9	mm
Settlement of mid point of edge x - 2	=	7.9	mm
Settlement of mid point of edge y - 1	=	7.9	mm
Settlement of mid point of edge y - 2	=	7.9	mm
Settlement of foundation center point	=	13.4	mm
Settlement of characteristic point	=	9.2	mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 51.37 MPa
Foundation in the longitudinal direction is rigid (k=9.12)
Foundation in the direction of width is rigid (k=9.12)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333

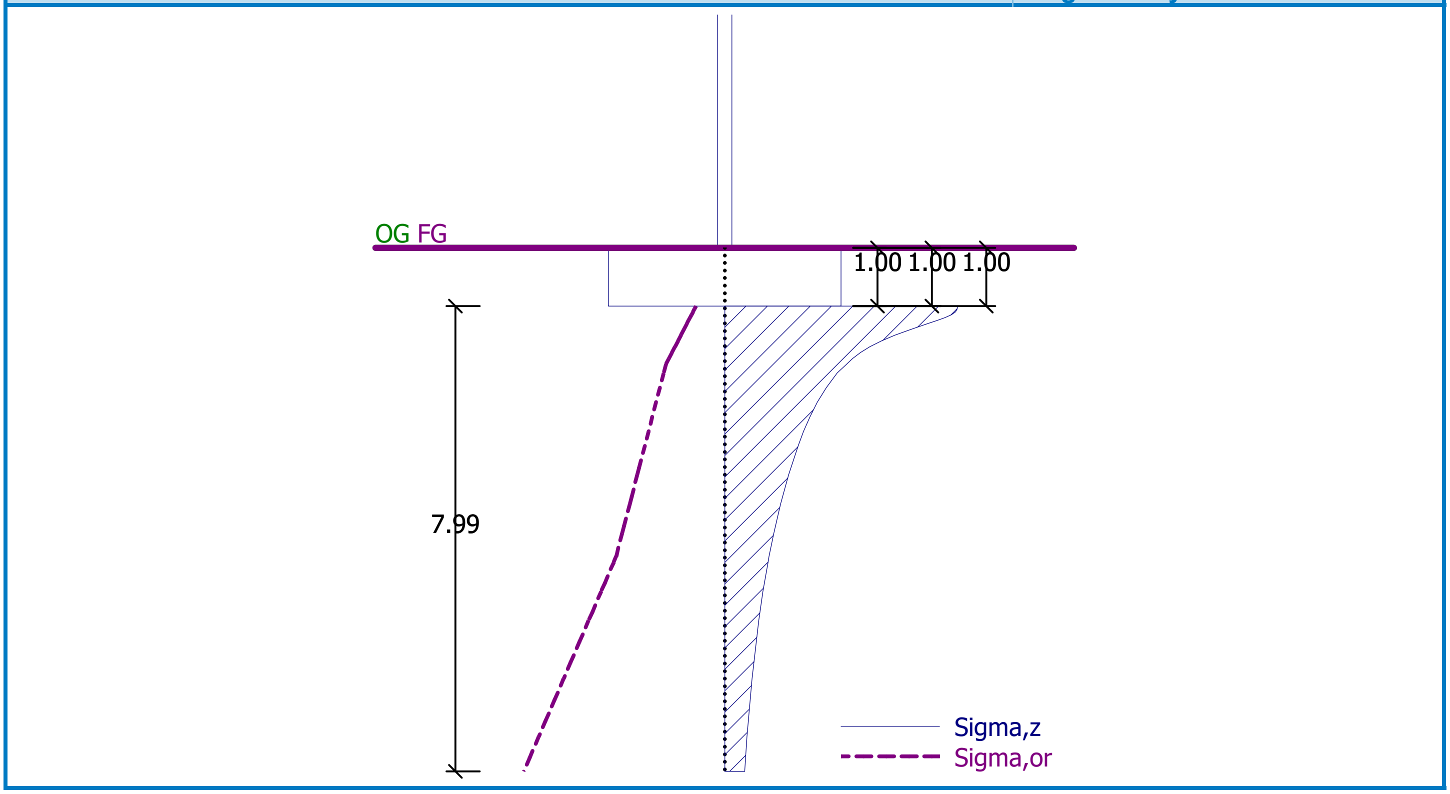
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 9.2 mm
Depth of influence zone = 7.99 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Name : Settlement **Stage - analysis : 1 - 1**



Input data (Stage of construction 2)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Granular Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	30.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 2)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	14172.46	1.22	Yes
ULS	No	0.00	0.00	181.81	14172.46	1.28	Yes
SLS	Yes	0.00	0.00	173.56	7026.51	2.47	Yes
SLS	No	0.00	0.00	173.56	7026.51	2.47	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 6.84 m
Length of slip surface l_{sp} = 21.44 m

Design bearing capacity of found.soil R_d = 7026.51 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 1893.39 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 2)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 12.2 mm
Settlement of mid point of edge x - 2 = 12.2 mm
Settlement of mid point of edge y - 1 = 12.2 mm
Settlement of mid point of edge y - 2 = 12.2 mm
Settlement of foundation center point = 17.7 mm
Settlement of characteristic point = 13.6 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 62.48 MPa
Foundation in the longitudinal direction is rigid (k=7.50)
Foundation in the direction of width is rigid (k=7.50)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 13.6 mm
Depth of influence zone = 10.58 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Input data (Stage of construction 3)

Geological profile and assigned soils

No.	Layer [m]	Assigned soil	Pattern
1	1.00	Engineered fill	
2	4.30	Superficial Granular Deposits	
3	23.70	Schist	
4	-	Schist	

Load

No.	Load new	Load change	Name	Type	N [kN]	M_x [kNm]	M_y [kNm]	H_x [kN]	H_y [kN]
1	NO	NO	ULS	Design	2400.00	0.00	0.00	0.00	0.00
2	NO	NO	SLS	Service	2400.00	0.00	0.00	0.00	0.00

Surface surcharges in the vicinity of footing

No.	Surcharge new	Surcharge change	Name	x_s [m]	y_s [m]	x [m]	y [m]	q [kPa]	α [°]	h [m]
1	NO	YES	Slab loading	0.00	0.00	10.00	10.00	50.00	0.00	0.00

GWT + incompressible subsoil

The ground water table is at a depth of 2.00 m from the original terrain.
Incompressible subsoil is at a depth of 29.00 m from the original terrain.

Settings of the stage of construction

Design situation : permanent

Verification No. 1 (Stage of construction 3)

Load case verification

Name	Self w. in favor	e_x [m]	e_y [m]	σ [kPa]	R_d [kPa]	Utilization [%]	Is satisfied
ULS	Yes	0.00	0.00	173.56	14172.46	1.22	Yes
ULS	No	0.00	0.00	181.81	14172.46	1.28	Yes
SLS	Yes	0.00	0.00	173.56	7026.51	2.47	Yes
SLS	No	0.00	0.00	173.56	7026.51	2.47	Yes

Analysis carried out with automatic selection of the most unfavourable load cases.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Vertical bearing capacity check

Shape of contact stress : rectangle
Most severe load case No. 2. (SLS)

Parameters of slip surface below foundation:
Depth of slip surface z_{sp} = 6.84 m
Length of slip surface l_{sp} = 21.44 m

Design bearing capacity of found.soil R_d = 7026.51 kPa
Extreme contact stress σ = 173.56 kPa

Bearing capacity in the vertical direction is SATISFACTORY

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Horizontal bearing capacity check

Most severe load case No. 1. (ULS)

Earth resistance: at rest
Design magnitude of earth resistance S_{pd} = 20.28 kN

Horizontal bearing capacity R_{dh} = 1893.39 kN
Extreme horizontal force H = 0.00 kN

Bearing capacity in the horizontal direction is SATISFACTORY

Bearing capacity of foundation is SATISFACTORY

Verification No. 1 (Stage of construction 3)

Settlement and rotation of foundation - input data

Analysis carried out with automatic selection of the most unfavourable load cases.
Analysis carried out with accounting for coefficient κ_1 (influence of foundation depth).
Analysis carried out with accounting for coefficient κ_2 (influence of incompressible subsoil).
Stress at the footing bottom considered from the finished grade.

Computed weight of spread footing G = 377.01 kN
Computed weight of overburden Z = 0.00 kN

Settlement of mid point of edge x - 1 = 15.1 mm
Settlement of mid point of edge x - 2 = 15.1 mm
Settlement of mid point of edge y - 1 = 15.1 mm
Settlement of mid point of edge y - 2 = 15.1 mm
Settlement of foundation center point = 20.6 mm
Settlement of characteristic point = 16.5 mm

(1-max.compressed edge; 2-min.compressed edge)

Settlement and rotation of foundation - results

Foundation stiffness:
Computed weighted average modulus of deformation E_{def} = 67.41 MPa
Foundation in the longitudinal direction is rigid (k=6.95)
Foundation in the direction of width is rigid (k=6.95)

Verification of load eccentricity

Max. excentricity in direction of base length e_x = 0.000<0.333
Max. eccentricity in direction of base width e_y = 0.000<0.333
Max. overall eccentricity e_t = 0.000<0.333

Eccentricity of load is SATISFACTORY

Overall settlement and rotation of foundation:
Foundation settlement = 16.5 mm
Depth of influence zone = 11.99 m

Rotation in direction of x = 0.000 (tan*1000); (0.0E+00 °)
Rotation in direction of y = 0.000 (tan*1000); (0.0E+00 °)

Dimensioning No. 1 (Stage of construction 3)

Analysis carried out with automatic selection of the most unfavourable load cases.

Verification of longitudinal reinforcement of foundation in the direction of x

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 4.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Verification of longitudinal reinforcement of foundation in the direction of y

Bar diameter = 16.0 mm
Number of bars = 11
Reinforcement cover = 40.0 mm
Cross-section width = 4.00 m
Cross-section depth = 1.00 m

Reinforcement ratio ρ = 0.06 % < 0.13 % = ρ_{min}
Cross-section is NOT SATISFACTORY; increase reinforcement ratio.

Spread footing for punching shear failure check

Column normal force = 2400.00 kN

Maximum resistance at the column perimeter

Force transmitted into found. soil = 9.38 kN
Force transmitted by shear strength of SRC = 2390.62 kN
Considered column perimeter u_0 = 1.00 m
Shear resistance at the column perimeter $v_{Ed,max}$ = 2.51 MPa
Resistance at the column perimeter $v_{Rd,max}$ = 2.94 MPa

Critical section without shear reinforcement

Force transmitted into found. soil = 356.61 kN
Force transmitted by shear strength of SRC = 2043.39 kN
Distance of section from the column = 0.71 m
Section perimeter u_{cr} = 5.49 m
Shear stress at section v_{Ed} = 0.39 MPa
Shear resistance of section without shear reinforcement $v_{Rd,c}$ = 0.74 MPa

$v_{Ed} < v_{Rd,c}$ => Reinforcement is not required

Spread footing for punching shear is SATISFACTORY

Appendix B – HBGS FAAP Historical Review and Summary Report

Former Anglesey Aluminium Plant
Geotechnical Remediation Strategy
Volume 1 - Historical Review and Summary Report

Table of Revisions

Date	Revision No.	Comments
22 October 2023	-	First Issue for Comment
24 October 2023	1	Final Issue

Job Title	**Geotechnical Remediation Strategy** **Historical Review and Summary Report**		
Client	Anglesey Land Holdings		
Location	Holyhead; Isle of Anglesey		
Date	24 October 2023	**Revision**	1
Prepared by	Nguyen Thi Mai Linh BSc MSc	**Signature**	
Reviewed by	Eur Ing Robert Hutchison BEng MSc MBA FGS M.ASCE CEng MICE	**Signature**	
Authorised by	Eur Ing Robert Hutchison BEng MSc MBA FGS M.ASCE CEng MICE	**Signature**	

Contents

List of Notations

Symbol	Units	Meaning
Is50	MPa	Point load results were normalised for a 50mm diameter specimen
NMC	%	Nature Moisture content
LL	%	Liquid Limit
PL	%	Plastic Limit
PI	%	Plasticity Index
CBR	%	California Bearing Ratio
k	m/s	Permeability
E_{intact}	GPa	Young’s module of intact rock
RQD	%	Rock Quality Designation
c	kPa	The cohesion component of shear strength
Ø	*°(degree)*	The friction angle
q_c	MPa	The ultimate bearing capacity of the rock mass.
E_m	GPa	The deformation modulus of the rock mass.
n	Non-unit	Poisson ratio
γ_b	kg/m^3	The bulk unit weight
USC	MPa	Unconfined Compression Test

1. Introduction

1.1 Introduction

This report serves as a comprehensive "Historical Review and Summary Report" of all existing ground investigation available reports related to the site located east of Holyhead, Anglesey. The site is earmarked for the construction of various mixed use industrial developments and has undergone various levels of investigation to assess its suitability for development. The focus of this document is primarily on the Geotechnical aspects, although it also encompasses environmental considerations, including land contamination, that could significantly impact future development.

1.2 Structure of the Historical Review and Summary Report

This report summarises the findings from the numerous site investigations conducted at the former Anglesey Aluminium site in a tiered structure:

1.2.1 The Tier 1 summaries provide overviews of the most relevant investigation reports that characterise the overall geological, hydrogeological, and contamination conditions across the entire site. The Tier 1 reports contain boreholes as well as geotechnical information about the soil and rock properties.

1.2.2 The Tier 2 summaries give more detailed reviews of supplementary reports that provide additional insights into site conditions within specific areas or from certain time periods. The Tier 2 reports contain borehole data but limited or no geotechnical testing information. They may include environmental data on contamination.

1.2.3 The Tier 3 summaries are very concise overviews of peripheral reports that are less directly applicable to the current status of the site but still contribute to the overall understanding. The Tier 3 reports do not contain any new borehole or geotechnical data.

In summary, the Tier 1 reports provide the most critical site-wide geological, hydrogeological, and geotechnical data as well as contamination assessments. The Tier 2 reports fill in gaps for particular zones and eras. The Tier 3 reports provide context but contain no new technical information. This tiered structure aims to consolidate the key findings from decades of site assessments into a robust conceptual site model for the former Anglesey Aluminium site.

1.3 Limitations of the Historical Review and Summary Report

The aim is to review and synthesize the findings from the numerous site assessments conducted over the past several decades, in order to establish a robust understanding of the ground conditions and any risks that need to be addressed to enable appropriate redevelopment of the former industrial site. There are some significant limitations to the historical review and summary report:

The Tier 1 geotechnical design report by Mott MacDonald in 2010 provides the most substantive geotechnical data for the site through field and lab testing. This includes SPT testing, strength and compressibility parameters, permeability values, etc.

The numerous environmental reports in Tiers 2 and 3 contributed useful borehole logs and contamination sampling results but did not include significant additional geotechnical data.

There appears to be a gap in geotechnical data from investigations after 2010. More recent assessments focused heavily on delineating and remediating contamination issues.

2. Tier 1 summaries

The Tier 1 Summary chapter provides overviews of the most critical site-wide investigations that offered extensive geotechnical and contamination data for the former Anglesey Aluminium site, especially concentrated in the main construction area. Four major studies from 2010-2012 are summarised that together provide robust characterisation of ground conditions, geotechnical properties, hydrogeology, and risks. Multiple boreholes in the Tier 1 reports contained key geotechnical testing like SPTs, strength, consolidation, classification, and permeability tests to profile site-wide soil and rock conditions. The Tier 1 summaries compile the salient findings from these comprehensive assessments that represent the core technical baseline understanding of the subsurface status for the entire former industrial site.

2.1 2010 07 Anglesey Aluminium REP, Ground Investigation Report, Mott MacDonald

- The study site comprises the main REP site and the conveyor route, each with unique topographical and geological attributes.
- The conveyor route's topography varies, with the western end at 4.8 m AOD, the central part between 2.0 m and 2.5 m AOD, and the eastern end around 8.0 m AOD.
- Weathered rock along the conveyor route has a thickness of about 0.5 m at its extreme ends.
- The REP site's elevation predominantly ranges between 8 m and 9 m AOD.
- Made ground at the main REP site exceeds a thickness of 3 m in eight specific locations, while the rest of the site generally has made ground under 3 m thick.
- Superficial geological deposits near a supposed fault line are over 9.3 m thick, around 4.3 m to the west of the fault, and generally between 1 m and 2 m for the rest of the site. However, the eastern side of the fault has deposits around 6m thick, but this measurement requires cautious interpretation.
- The solid geology reveals that the depth to the intact rockhead varies across the site, from ground level exposures to 12 m below ground level. Depths are less than 3 m bgl along the eastern site boundary and beneath the wood chip storage area, up to 5 m bgl in the southeastern corner of the wood chip storage area, around 7 m bgl beneath the proposed Boiler House, 12 m bgl beneath the Baghouse area and the northern end of the Ash Silos, and reduces to around 7 m bgl towards the proposed Stack.
- During the site investigation, groundwater was detected at 52 locations, with inflow rates ranging from minor seepage to rapid flows that jeopardised the stability of some trial pits.

- Figure 2.1.1 depicts the depth to water measured 20 minutes post-strike, suggesting localised groundwater pockets in the superficial geology due to the disparity between trial pits and boreholes. Standing water was also observed at multiple sites.

Figure 2.1.1: Steady state water depth for each location

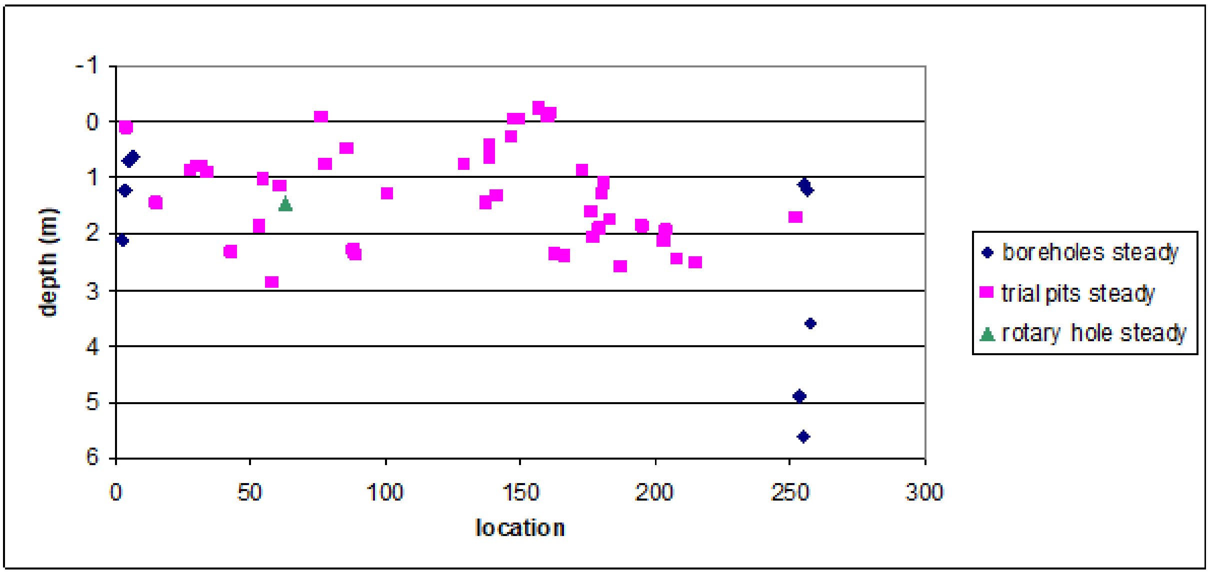
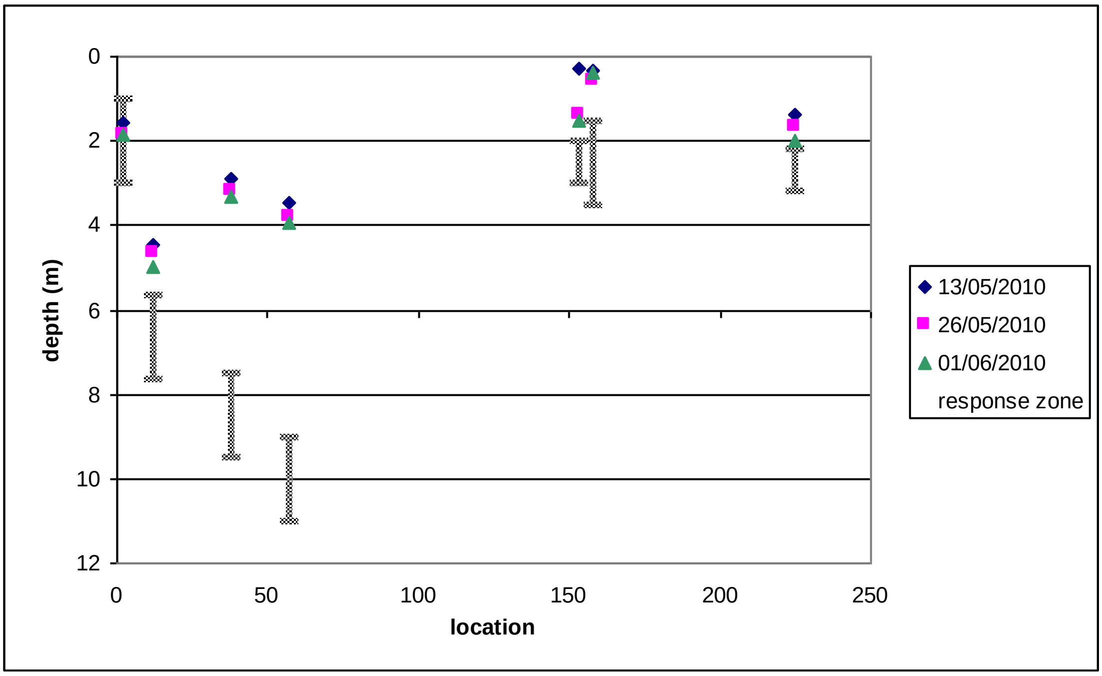


Figure 2.1.2: Depth to groundwater at monitoring points between 13/05/2010 and 01/06/2010 and response zone for monitoring locations



- Continuous groundwater monitoring, as shown in Figure 2.1.2, indicates consistent levels, especially in deeper areas of the reclamation yard and above the rockhead across the site.
- Potential ground risks at the site are linked to the made ground and the presence of large reinforced concrete blocks underground, while ongoing gas monitoring from May 13 to June 1, 2010 showed no significant issues, and recent studies, including those on contamination and leachability.
- Geotechnical parameters for the site were determined through a variety of methods, and it is assumed that most of the made ground will need to be removed before construction, with excavated material either stored on-site or taken to a landfill.
- The geotechnical testing in the proposed conveyor route and reclamation yard areas revealed a variable composition of superficial materials. The superficial material proved to be very variable comprising of both cohesive and granular deposits. This summarises the cohesive and granular classification testing for the named areas:
    - Classification summary of cohesive superficial material in Table 2.1.

Table 2.1: Classification Summary of Cohesive Superficial Material in Conveyor route and Reclamation yard.

Location (bh)	**Depth** (m)	**Clay** (%)	**Silt** (%)	**Sand** (%)	**Gravel** (%)	**Cobbles** (%)
2 Conveyor route	1.0	16	37	35	12	0
6 Conveyor route	0.8	7	18	28	47	0
11 Conveyor route	1.5	12	37	33	18	0
11 Conveyor route	2.5	15	33	30	23	0
Average conveyor route		**12.5**	**31.3**	**31.5**	**25**	**0**
253 Reclamation yard	1.0	4	24	30	38	4
253 Reclamation yard	2.8	8	31	29	32	0
257 Reclamation yard	3.8	5	77	18	0	0
Average reclamation yard		**5.7**	**44**	**25.7**	**23**	**1.3**

- Classification summary of granular superficial material in Table 2.2:

Table 2.2: Classification Summary of Cohesive granular superficial material in Conveyor route and Reclamation yard.

Location (bh)	**Depth (m)**	**Clay (%)**	**Silt (%)**	**Sand (%)**	**Gravel (%)**	**Cobbles (%)**
2 Conveyor route	2.8	0	1	8	55	36
3 Conveyor route	2.2	5	8	29	59	0
4 Conveyor route	1.7	0	14	20	64	2
5 Conveyor route	1.8	4	13	34	45	5
7 Conveyor route	1.0	0	9	90	1	0

Location (bh)	Depth (m)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	Cobbles (%)
8 Conveyor route	1.0	5	11	85	0	0
9 Conveyor route	0.5	5	15	26	44	11
Average Conveyor route		**2.7**	**10.1**	**41.2**	**38.3**	**7.7**
23 Reclamation yard	1.0	2	19	21	52	5
23 Reclamation yard	2.0	0	8	35	57	0
253 Reclamation yard	4.8	2	21	29	45	2
253 Reclamation yard	5.8	0	5	8	42	45
257 Reclamation yard	6.8	0	4	19	77	0
Average Reclamation yard		**0.8**	**11.4**	**22.4**	**54.6**	**10.4**
177 Wooded area	1.0	3	17	33	47	0
177 Wooded area	3.0	2	21	25	51	1
Average wooded area		**2.5**	**19**	**29**	**49**	**0.5**

- Plasticity results along conveyor route. Natural moisture contents are generally considerably higher than plastic limits indicating that the material is normally consolidated in Table 2.3:

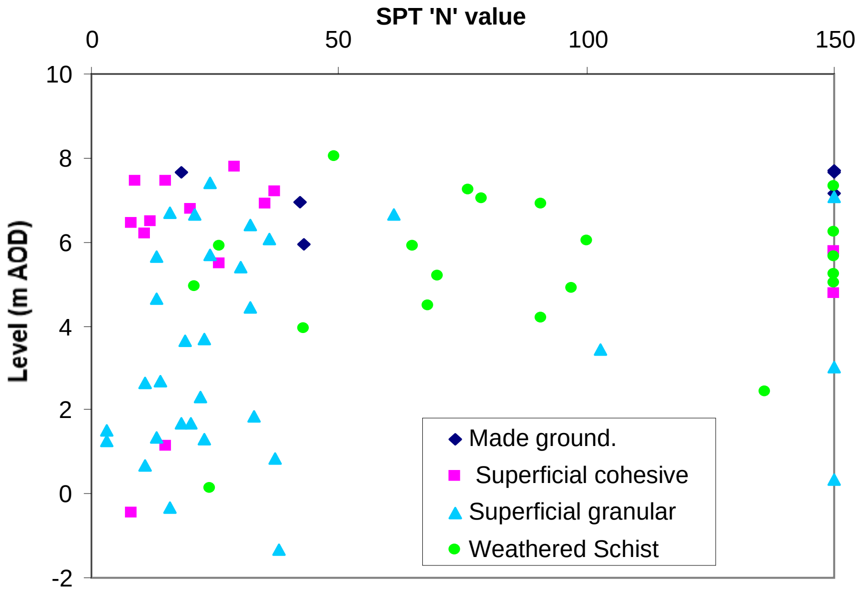
Table 2.3: Plasticity results along conveyor route

Moisture content (%)	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Classification	
113	56	50	6	MH	High plasticity Silt
632	678	385	293	O	Organic Peat
90	128	66	62	ME	Extremely high plasticity Silt
28	44	30	14	MI	Intermediate plasticity Silt

- The shear vane test in soft silt was abandoned due to the material being too stiff for the test equipment, and SPT tests at Anglesey Aluminium REP showed high counts, leading to the use of empirical relationships for strength calculations. The geotechnical investigation reveals four distinct strata with varying SPT counts with depth in Figure 2.1.3:
- The geotechnical investigation at the site identified four key strata with distinct Standard Penetration Test (SPT) counts and strength characteristics. Made Ground showed highly variable

SPT counts, making it difficult to establish a typical strength range. The Superficial Cohesive stratum had SPT counts generally between 8-37, leading to a design undrained shear strength of 50 kPa Superficial Granular material had similar SPT counts but included more outliers, with a φ_{max} value calculated between 34-42 degrees for most areas. Lastly, Weathered Schist exhibited higher SPT counts ranging from 50-100 and φ_{max} value between 40-43 degrees.

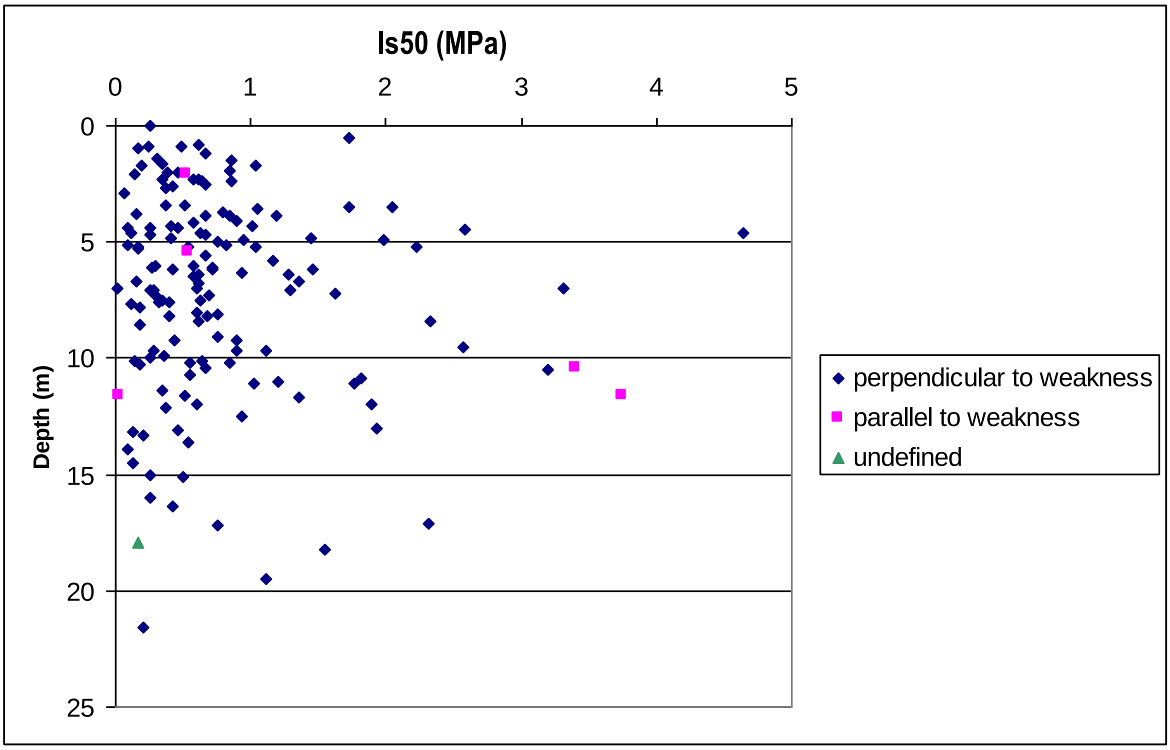
Figure 2.1.3: SPT profile through made ground and drift deposits at Anglesey Aluminium REP



- The conducted CBR tests, averaging a value of 11.5%, indicate that the current material is inadequate for pavement construction, necessitating the use of capping material. Due to the presence of unsuitable elements like weathered bedrock, comprehensive excavation and capping are essential for proper pavement formation.
- The soakaway tests conducted at the site exhibited minimal soil infiltration, hindering the determination of permeability from these tests at specified locations: 15, 141, and 252 (made ground) and 53 and 183 (granular superficial deposits). Consequently, permeability estimates will rely on particle size distribution analysis, guided by Hazen's law. Derived permeability values for the granular superficial deposits are indicated to be around 1e-6 m/s.

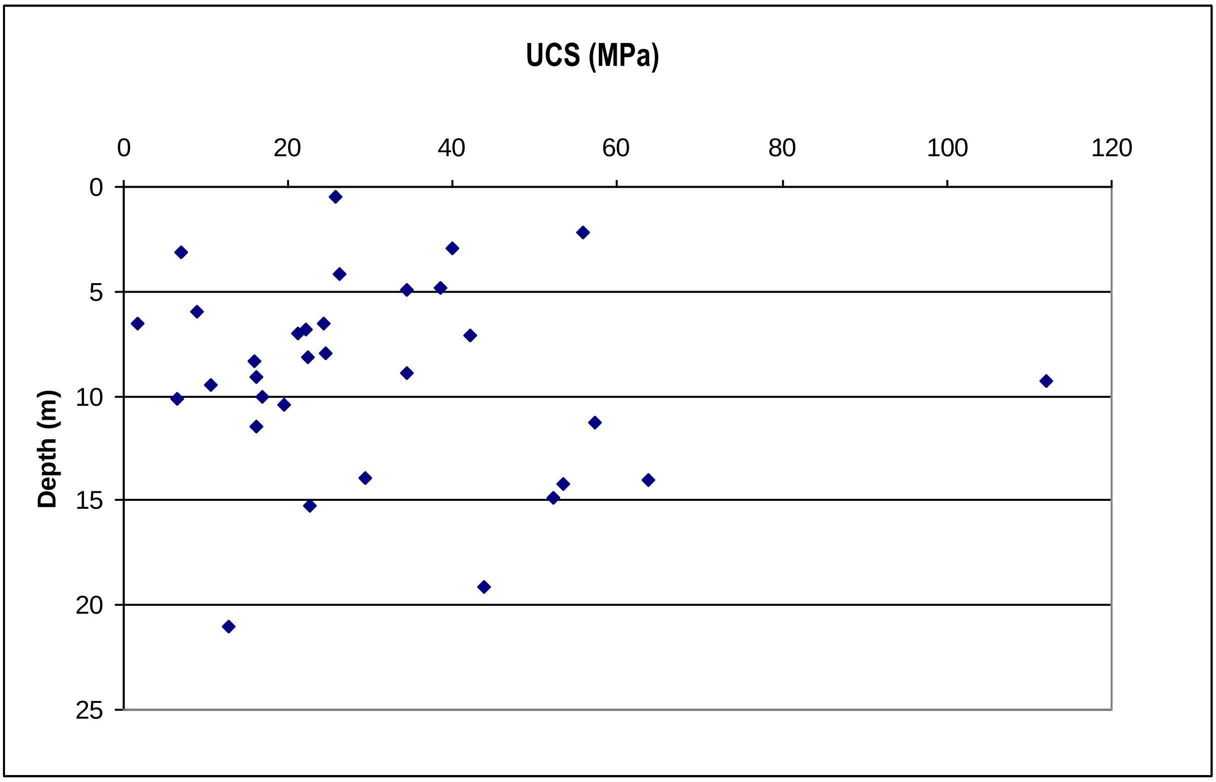
- Geotechnical parameters for the rock mass:
  - Among the 163 scheduled paired diametral and axial point load tests, several were invalidated due to fractures not occurring at the load application points, primarily stemming from weaknesses in the rock, often oriented at 0-10 degrees.
  - The remaining valid results consisted of 62 paired samples and 95 single diametral samples, with the normalised point load data (Is50) displaying no discernible depth-related pattern and an overall average Is50 value of 0.8 MPa as illustrated in Figure 2.1.4:

Figure 2.1.4: Diametral point load test results for entire REP site



- A total of 32 Unconfined Compressive Strength (UCS) tests were performed on samples extracted from 26 boreholes, with 29 of these tests including measurements of Young's modulus and Poisson's ratio.
- The scatter plot in Figure 2.1.5 illustrates the distribution of UCS values with depth, showing the majority of results falling between 10 and 35 MPa, while the average UCS value was calculated at 30 MPa. However, outliers were notable, with a minimum UCS of 1.72 MPa and a maximum of 112 MPa.

Figure 2.1.5: UCS results from 32 samples obtained from across the entire REP site



- The Rock Quality Designation (RQD) varies with depth and between boreholes, displaying little correlation, with an average RQD of 22% and a range of 15% to 30%, providing insights into the heterogeneous nature of rock quality in the studied geological area:

Figure 2.1.6a: Variation of RQD with depth for the REP site

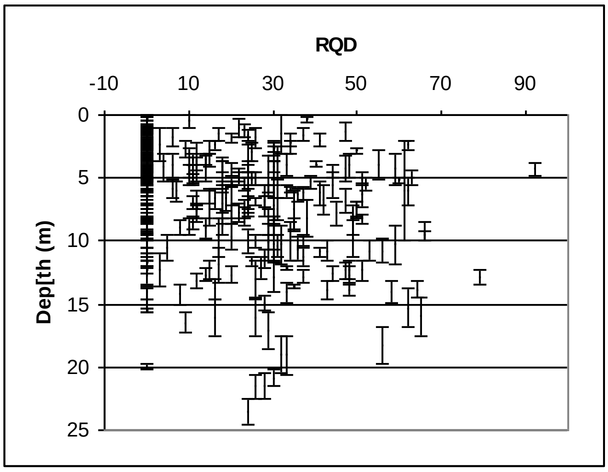
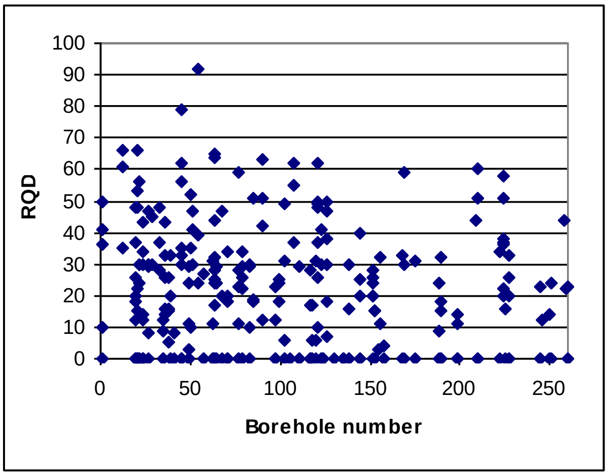


Figure 2.1.6b: Variation of RQD across exploration locations



- Summarises the determined parameters for the intact rock encountered during the site investigation in Table 2.4.

Table 2.4: Summary of Intact Rock Parameters from Site Investigation

Parameter		**units**
UCS	5 - 25	MPa
Anisotropic ratio	1	-
Young's modulus	1 at surface increasing with depth by 0.5 GPa/m	GPa
Poisson ratio	0.4	-
Bulk density	2660 at surface increasing with depth by 4 Kg/m^3	Kg/m^3

- Geotechnical tests for the superficial strata in Table 2.5.

Table 2.5: Geotechnical Test Results for Superficial Strata.

Strata		NMC %	SPT	LL	PL	PI	CBR %	k m/s
Superficial granular		**-**	**11 - 36**	**-**	**-**	**-**	**9.2 - 16**	**1e-6**
Superficial cohesive		**28 - 113**	**8 - 37**	**44 - 128**	**30 - 66**	**6 - 62**	**-**	**-**
Weathered Schist		**-**	**50 - 100**	**-**	**-**	**-**	**-**	**-**

- Geotechnical test for the rock mass in Table 2.6.

Table 2.6: Geotechnical Test Results for Rock Mass

Strata	**Point Load (MPa)**	**UCS MPa**	**E_{intact} GPa**	**n**	**γ_b kg/m^3**	**NMC %**	**RQD %**
Schist	0.1 – 0.8	10 - 35	1 - 15	0.3 – 0.6	2620 - 2770	0.1 – 0.7	15 - 30

- Effective stress parameters derived for the superficial strata in Table 2.6.

Table 2.7: Effective Stress Parameters for Superficial Strata

Strata	**c_u (KPa)**	**Ø**
Superficial granular	-	34º – 42º (30º at centre of conveyor route)
Superficial cohesive	50	-

Strata	c_u (KPa)	Ø
Weathered Schist	-	40 º - 43 º

- Geotechnical parameters derived for the rock mass in Table 2.8.

Table 2.8: Geotechnical Parameters for Rock Mass

Strata	γ_b (kg/m³)	q_c (MPa)	c (MPa)	Ø	E_m (GPa)	n
Schist	2660 + 4z	1.65 – 8.25	0.5 – 2.5	30 º	0.2 + 0.1z	0.4
z = depth below surface in metres						

- During the investigation, it was observed that groundwater in low-lying marshy areas along the conveyor route exhibited tidal fluctuations, and hydraulic connections between neighbouring rotary holes were evident, occasionally spanning over 20 meters apart, although the hydraulic connectivity between groundwater beneath the main REP site and local surface water bodies, including the Irish Sea, remained unconfirmed.
- Summary of chemical analysis within superficial geology with 10 samples in Table 2.9.

Table 2.9: Summary of Chemical Analysis for Superficial Geology Samples (10 samples)

pH	Water soluble Sulphate (SO_4) g/l	Chloride (g/l)	Organic content (%)
1.9 – 8.3	0.01 – 0.34	0.02 – 2.68	<0.5 – 4.2

- Soil contamination levels were below guideline values for commercial/industrial land use, indicating low risk to human health depending on pollutant linkages.
- Leachate testing identified several exceedances of EQS levels for aluminium, chromium, copper, fluoride, pH, and PAHs in multiple samples, indicating a potential risk to marine life if relevant pollutant linkages present.
- WAC testing classified made ground samples into categories ranging from inert waste to hazardous waste. Some samples require pre-treatment before disposal due to contaminants like total organic carbon, TPH, fluoride, and antimony.

2.2 2010 07 Report on a Ground Investigation at Anglesey Aluminium REP, Holyhead (Volume One), Soil Engineering

The site investigation for this project involved a combination of boreholes and trial pits, each serving specific purposes. There were 16 cable percussion boreholes, including both 250mm and 200mm diameter ones, drilled to depths ranging from 1.2 to 8.0m. These boreholes were primarily used for soil sampling and Standard Penetration Test (SPT) testing. Additionally, one borehole (No. 2) among them was utilised for the installation of a gas monitoring well.

- In contrast, 54 rotary drilled boreholes were extended to varying depths, ranging from 2.3 to 24.5m, using a 92mm diameter core barrel. The primary objectives of these boreholes were bedrock coring, Rock Quality Designation (RQD) and fracture logging, as well as rock sampling.
- Furthermore, 148 trial pits were excavated to depths ranging from 0.1 to 3.3 meters. These trial pits were mainly concentrated in the central and eastern parts of the site and were used to assess near-surface conditions, collect soil samples, and perform in-situ California Bearing Ratio (CBR) and permeability testing.
- Summary the geotechnical investigations in Table 2.10.

Table 2.10: Summary of Geotechnical Investigation

Strata	Depth (m)	Description and Observations
Made Ground	0.10 - 2.60	Comprising schist, metal, concrete, refractory brick, with a notable thickness increase in the reclamation yard area.
Superficial Deposits	Up to 6.70 (may extend deeper)	Variable sequence of sand, gravel, cobbles, and schist boulders.
Precambrian/Cambrian Schists	Variable	Encountered in all rotary boreholes, locally observed at ground level, overlain by made ground and superficial deposits in some areas. Upper section inconsistently weathered.

- Groundwater data on May 13, 2010, revealed varying depths of groundwater across multiple boreholes, with levels ranging from 0.28 to 4.45m
- The in-situ California Bearing Ratio (CBR) tests conducted at depths between 0.55m and 0.75m produced CBR values ranging from 9.2% to 16%.
- The monitoring, conducted on days of low or falling atmospheric pressure, when possible, revealed no detectable Methane or Carbon Dioxide readings, while the Oxygen levels recorded were within the range of 19.6% to 21.0%.

2.3 2010 07 Anglesey Aluminium REP, Geotechnical Design Report, Mott MacDonald

- This report concludes that pile foundations are required for heavy structures in the northwest of the site, including the stack, baghouse, boiler building, ash silos, north cooling towers and conveyor route.
- The ground conditions in these areas consist of deep superficial deposits and weathered bedrock with insufficient bearing capacity.
- Two ground models are presented - one for the main site and one for the conveyor route. The main site has up to 9.3m of superficial deposits and weathered rock overlying intact bedrock. The conveyor route has up to 4.65m of superficial deposits over bedrock.
- Driven piles are recommended as they are unaffected by groundwater, loose soils or variable rockhead depths. They can be driven to refusal to obtain maximum capacity.
- Potential issues with driven piles include deviation from obstructions like boulders and concrete, and damage during driving. Most obstructions found were shallow and can be excavated.
- Piles should be end-bearing on competent rock. The superficial soils will provide minimal shaft support so capacity is governed by the pile shaft strength.
- Geotechnical parameters are provided for pile design in Table 2.11.

Table 2.11: Geotechnical parameters are provided for pile design

γ_b kg/m^3	q_c MPa	c MPa	Ø °	E_m GPa	n
2660 + 4z	1.65 – 8.25	0.5 – 2.5	30	0.2 + 0.1z	0.4

- Spread or strip foundations are recommended for lighter structures in other areas of the site, including the south cooling towers, water treatment building, fire pump house, wood chip buildings, silos, auxiliary boiler and other small buildings.
- These areas have shallow bedrock (less than 2m deep) or competent dense sands and gravels.
- Two ground models are presented - one for shallow bedrock areas and one for the deeper bedrock in the former reclamation yard in Table 2.12.

Table 2.12: Geotechnical Parameters for Pile Design

Ground model	Strata	Depth to top (m bgl)	Thickness (m)	Comments
Shallow bedrock	Peat / topsoil	0.0	0.0 – 0.25	Located locally along and within the wood to the southeast of the REP site. Generally, not more than 0.1m thick.

Ground model	**Strata**	**Depth to top (m bgl)**	**Thickness (m)**	**Comments**
	Made Ground	0.0	0.0 – 1.0	Generally, around 0.5m thick locally up to 2.5m. Comprised of schist, metal, concrete, and refractory brick with other materials.
	Superficial Deposits	1.5	0.0 – 4.3 generally 1.0	Present in majority of locations. Variable sequences of sands, clays, gravels and cobbles. Thicknesses generally defined within trial pits. Higher, inconsistent thicknesses were defined within rotary open holes which had difficulty distinguishing between weathered schist and gravel. The higher thicknesses may therefore be overestimated.
	Schist	1.0 – 3.0	20+	Mica schists of the Mona Complex. The Schist was strong narrowly foliated, micaceous with very narrow folded bands of quartz. Discontinuities were closely spaced and planar smooth. Depth to schist generally around 2.0 m bgl although locally up to 5 m bgl. The upper weathered section of the Schist is of inconsistent depth and varies from 0.0 – 2.5m thick.
Reclamation yard	Made Ground	0.0	1.0 – >2.0	Generally, around 1.4m thick locally over 2.1m. Comprised of schist, metal, concrete, and refractory brick with other materials. Local obstructions encountered at 2.1m
	Superficial Deposits	1.4	1.5 – 8.0	Encountered within majority of locations. Generally, consists of sands, gravels and boulders although locally clay and silts present. Higher thicknesses towards the suspected fault area.
	Schist	1.0 – 5.0	20+	Mica schists of the Mona Complex. The upper weathered section of the Schist is of inconsistent depth and varies from 0.0 – 6.0m thick. Weathered zone contains intact thicknesses also. The Schist was strong narrowly foliated, micaceous with very narrow folded bands of quartz. Discontinuities were closely spaced and planar smooth.

- In shallow bedrock areas, strip foundations can be constructed after excavating to rockhead. In deeper areas, foundations may be seated on superficial deposits. All made ground exceeding 2m depth should be removed.
- Geotechnical parameters are provided for foundation design:

Table 2.13: Geotechnical Parameters for Foundation Design

Strata		γ_b kg/m^3	c MPa	Ø °	E_m GPa	n
Superficial deposits	cohesive	2.0 – 2.3 *	c_u = 0.050	0	E_u = 0.012 E' = 0.009	0.1 (drained)
	granular	2.0 – 2.3 *	-	34 – 42	E' = 0.031	0.2
Weathered rock		2660	-	40 - 43	E' = 0.185	0.2
Rock mass		2660 + 4z	0.5 – 2.5	30	0.2 + 0.1z	0.4

z = depth below surface in metres

- For cohesive materials, undrained stiffness was estimated from SPT N-values. Drained stiffness was calculated from undrained stiffness and plasticity.
- Granular stiffness values were taken from Burland & Burbidge's relationship between E'/N and SPT N-value.
- The foundation type for the steam turbine building is uncertain due to varying ground conditions.
- The northwestern part has deep deposits/weathered rock that would suit piles. The southeastern part has shallow bedrock that would suit spread foundations.
- Option 1 is a combination of driven piles in the northwest and spread foundations in the southeast. This is economical but may cause differential settlement.
- Option 2 is complete spread foundations with excavation to rockhead in the northwest. This prevents settlement issues but is costly and disruptive.
- Option 3 is driven piles in the northwest transitioning to concrete piers towards the southeast. This reduces the distinction between foundation types.
- Excavations for piers may be impacted by groundwater ingress.
- The final foundation design will depend on further structural information and findings during construction.
- A fluid design approach is recommended considering the varying ground conditions across the building footprint.
- Excavations will be required for a pump house near the cooling towers and a runoff pond north of the site.

- The pump house excavation will be in made ground and bedrock. Support will be needed, and groundwater is likely due to the proximity to cooling towers. Pumping and drainage will be required.
- The runoff pond will be in made ground and superficial deposits. Made ground should be excavated first, then sheet piles installed to allow excavation to design depth.
- Groundwater was encountered in a trial pit here but not at depth. Localised seepage is likely. Pumping and drainage will be needed.
- The pump house excavation will penetrate the bedrock so can be excavated by chiselling.
- If water pressure is significant in the pump house excavation, sealing may be required prior to foundation construction.
- Based on guidance, the concrete used for groundworks should have a Design Sulphate Class of DS-1 and an Aggressive Chemical Environment for Concrete (ACEC) classification ranging from AC-1s to AC-4z.
- The near-surface materials across the site are inadequate as a road subgrade.
- Excavation and capping will be required in all pavement areas prior to subbase construction. The depth will depend on the underlying conditions and traffic loads.
- Key hazards include unstable excavations, hard rock, high groundwater, obstructions, and insufficient bearing capacity.
- Control measures like support, drainage, and removal of obstructions are proposed. Risk management is needed for specific structures, the Table 2.14 is provided for general information only:

Table 2.14: Proposed Control Measures and Risk Management for Structures (For General Information Only)

Hazard / Risk	Cause	Probability	Impact	Risk Rating	Consequence
Unstable temporary excavations	Loose granular soil and seepage forces	3	1	3	Collapse of excavations
Breaking out hard strata	Higher than expected moderately strong rockhead level during excavations	3	2	6	Increased time spent excavating through the hard strata
High groundwater	Tidal area along conveyor route. Prolonged wet weather.	3	3	9	Hinder excavations and spread foundation construction
Flooding	Storms	3	2	6	Flooding of excavations

Hazard / Risk	Cause	Probability	Impact	Risk Rating	Consequence
Encountering obstructions to pile driving	Mass concrete in made ground boulders in superficial material	5	2	10	Delays to foundation construction as obstruction excavated
Insufficient bearing capacity	Rockhead lower than expected below the proposed foundation level	2	3	6	Foundation failure

Probability range: 1 = negligible to 5 = very likely
Impact range: 1 = very low to 5 = very high

2.4 2012 05 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works - Phase A, Golder Associates Ltd

- The geology comprises drift deposits of clay, sand and gravel overlying metamorphic bedrock (mica schist). A fault runs north-south across the site.
- Hydrogeology - perched groundwater lenses present in drift deposits. Limited groundwater within weathered schist bedrock.
- Background soil and groundwater quality data collected from offsite area for comparison.
- IPPC Zone 3, located in the southwest of the site, underwent three intrusive investigations in 1994, 2007, and 2012:
    - Summary of Geological Investigations in IPPC Zone 3 in Table 2.15.

Table 2.15: Summary of Geological Investigations in IPPC Zone 3

Investigation By	Year	Boreholes / Trial Pits	Depth Investigated	Strata
Wallace Evans	1994	TP48-TP50	0.7m	No geology logs available
Golder Associates	2007	GA01-GA04, GA05 (TP)	0-2.6m	Made ground, silty clay, sandy silt, gravel
Golder Associates	2012	GAPH01, GAPH02	0-3m	Made ground, clay, weathered schist

-
    - The 1994 investigation, conducted for the permit application, involved three trial pits and analysis of three soil samples for PAHs, pH, and PCBs.
    - Investigations during site operation in 2007 and at the time of surrender in 2012 included boreholes and additional analysis for metals, cyanide, TPH, and other parameters.
    - Comparing maximum concentrations over time showed that PAH levels were in a similar order of magnitude between the permit application and surrender (45.8 mg/kg and 71.5 mg/kg, respectively).
    - pH levels slightly increased from 8.55 during the permit application to 11.2 at surrender but remained within acceptable limits.
    - Some metals were found at concentrations higher than background levels, but these were not attributed to site activities.
    - Total Petroleum Hydrocarbons (TPH) levels at surrender were lower than background concentrations (748 mg/kg compared to 1450 mg/kg).
- Overall, there is no evidence that activities conducted in IPPC Zone 3 led to a deterioration in soil quality. The soil in this zone is considered to be in a satisfactory state, allowing for the surrender of the permit for this zone.

- IPPCC Zone 6, also known as the A-Frame, encompasses the site area dedicated to alumina storage, connected to the dock by an underground conveyor. Located within the facility, it played a pivotal role in the transfer of alumina to the fume treatment centre and pot rooms, underwent three intrusive investigations in 1994, 2007, and 2012:
  - Summary of Geological Investigations in IPPC Zone 6 in Table 2.16:

Table 2.16 Summary of Geological Investigations in IPPC Zone 6:

Investigation By	Year	Boreholes / Trial Pits	Depth Investigated	Strata
Wallace Evans	1994	TP63-TP66	0-2.9m	Clayey silt, silty clay, silty sand, peat, clay
Golder Associates	2007	GA31	0-3m	Made ground, silty clay with rock fragments
Golder Associates	2012	GAPH04-GAPH06, GAPH08	0-3m	Made ground, weathered schist, silty clay, peat

  - 1994 permit application: 10 trial pits, 10 soil samples analysed for metals, fluoride, oils, pH.
  - 2012 surrender: 3 boreholes, 1 trial pit, 12 soil samples analysed for metals, fluoride.
  - Higher metals and fluoride generally within range of baseline and background levels.
  - Maximum aluminium was 58,300 mg/kg compared to 20,000 mg/kg background. Likely represented baseline conditions based on 3 of 12 samples exceeding background.
  - 12 soil samples over 18 years provides evidence that activities did not impact soil quality.
- In IPPC Zone 7, also known as the Rectifier Yard, situated in the southeastern part of the site, the 132 Kv supply from the National Grid enters the facility. This zone includes a substantial substation, individual transformers, and a bunded area for mobile tanks containing transformer oil, with reports of minor staining on the unsurfaced ground near the tanks.
  - Summary of Geological Investigations in IPPC Zone 7 in Table 2.17:

Table 2.17: Summary of Geological Investigations in IPPC Zone 7

Investigation By	Year	Boreholes / Trial Pits	Depth Investigated	Strata
Wallace Evans	1994	TP67-TP69	0 – 1.3m	Gravel, silty clay, weathered schist
Golder Associates	2007	AAM-REC-01 to -11	0 - 4m	Made ground, clay, silt, sand, gravel, weathered schist
Golder Associates	2012	GABH05-GABH07	0 - 5.85m	Made ground, clayey sand, silt, gravel, weathered schist

  - 1994 permit application: 2 trial pits, 3 soil samples analysed for metals, oils, pH.
  - 2008 operation: 11 boreholes, 35 soil samples analysed for oils and 4 for metals, PAHs.

- 2012 surrender: 3 boreholes, 8 soil samples analysed for metals, TPH, PAHs. 4 groundwater samples.
- Higher levels of some metals and PAHs compared to 1994 but within background range.
- Maximum TPH in soil was 2,050,000 mg/kg in 2012 compared to 7.4 mg/kg in 1994. Related to 2008 transformer fire. Requires future management.
- Over 46 soil samples and 4 groundwater samples over 18 years provides substantial evidence.

- Previous assessments post-fire indicated that the removal of free product hydrocarbons would not be suitable given the estimated small volume. The geology in this zone includes made ground, drift deposits of clay and sand, and schist bedrock. While soil impacts are evident, the residual TPH concentrations in groundwater may not necessitate remediation from a risk perspective.
- The zone referred to as "Zone A" in the provided text corresponds to the area containing the sewage treatment plant, stacks, and fume treatment plant. This zone is located to the north of the anode plant and east/southeast of IPPC Zone 6 within the site.
    - Summary of Geological Investigations in IPPC Zone A in Table 2.18.

Table 2.18: Summary of Geological Investigations in IPPC Zone A

Investigation By	Year	Boreholes / Trial Pits	Depth Investigated	Strata
Golder Associates	2007	GA29, GA30	0- 2.2m	Made ground, weathered schist
Golder Associates	2012	GAPH14, GAPH15	0-3m	Made ground

- 2007 operation: 3 boreholes, 5 soil samples analysed for wide range of contaminants.
- 2012 surrender: 2 boreholes, 4 soil samples analysed for metals, TPH, PAHs, pH.

- Most results within background range - no evidence of impacts. 9 soil samples over 5 years provides good evidence that activities did not impact soil quality.

3. Tier 2 summaries

The Tier 2 Summary chapter overviews supplementary investigations offering additional focused insights into site conditions within specific areas. In contrast to the Tier 1 reports, the 22 Tier 2 reports summarised, spanning 1994-2021, generally contain borehole data but limited or no geotechnical testing information. While not containing substantial new geotechnical data, the Tier 2 reports help fill gaps by providing details on contamination and subsurface conditions in discrete zones like the Rectifier Yard and Pitch Tanks or at different periods over decades of site operation.

3.1 1994 09 Anglesey Aluminium Metal Limited, Environmental Assessment, Groundwater and Land Contamination Survey Report, Wallace Evans Limited

- The Wallace Evans investigations were conducted between 1992 and 1993, with the results reported in 1994. The investigations focused on seven areas of the site, two of which fall within the North Road area. All boreholes and trial pits were backfilled after the investigations were completed. The scope of work involved:
- In the former Hawkins Heap area, 33 trial pits were excavated and one borehole was drilled. Chemical analysis of this area showed elevated levels of aluminium, fluoride, copper and zinc in the soil.
- In the Alumina Storage area, three trial pits were dug and one borehole was drilled, however only one trial pit and borehole are actually within the North Road area. Chemical results here showed no particularly high contaminant concentrations in the soils, except for iron which was found at a higher level than would be expected naturally.
- Ground conditions summaries:
    - Made Ground does not appear to have been identified at the investigation locations.
    - Drift deposits were encountered at the locations investigated. They varied from silty fine sand with schist gravel to sandy silty clay with occasional schist boulders. Peat horizons were associated with marshy, waterlogged areas.
    - Schist bedrock was found at most locations, characterised as green, slightly to highly weathered mica schist. The depth to bedrock ranged from 0.6 to 3.6 meters below ground level (mbgl), with the shallowest bedrock located in the North Road area.

3.2 2001 09 Phase 1a Site Report for IPPC Application at the Anglesey Aluminium Facility, Holyhead, Anglesey, URS

- In 2001, URS carried out 4 boreholes to around 3m below ground level (mbgl) in the PDU area. No investigation was done in the PDU black edge footprint.
- One borehole was done in Area A but no data was available.
- Two boreholes were undertaken in Area B but again no data was present.
- One borehole (BH60) was carried out in Area C where made ground was found to 1.2 mbgl, comprising reworked surface deposits and schist with ash and firebrick. The borehole did not reach bedrock or groundwater, but they are likely present around 2.5 mbgl.
- No groundwater samples were collected.
- Sampling was limited to PCBs, phenols and PAHs.
- In 2001, URS also dug 3 trial pits (referenced TP67, TP68, TP69) around the boundary of Area 4. One trial pit log was provided (TP67), which went down to 1.4 mbgl where shallow groundwater caused inflow into the pit. This trial pit did not encounter made ground or bedrock but seems to have been excavated into weathered schist.
- A summary of soil lab data was presented but not assessed; however, concentrations did not seem high compared to commercial end use.

3.3 2008 01 Phase II Environmental Site Investigation of Anglesey Aluminium Metal Ltd, Penrhos Works, Holyhead, Anglesey, Golder Associates Ltd

Summarises the field observations made during the site investigation at Penrhos Works:

- Made ground consisting of sandy silt with gravel fill material to depths of up to 3m. Waste materials like brick, glass, and ash were frequently encountered.
- Drift deposits of sandy silt with rock fragments were encountered underlying the made ground in some locations to depths of 0.3-3m.
- Weathered schist bedrock was encountered underlying the made ground and drift deposits starting at ground level in some locations. The total depth was unproven but encountered to a maximum depth of 3m.
- Groundwater seepages were encountered in 9 boreholes at depths ranging from 0.7m to 3m. The water likely represents localised perched water.
- Evidence of contamination based on field observations was limited. Slight odours were noted in a few locations. Elevated PID readings were detected in 6 samples, with a maximum of 85.1 ppm in fill near Davik's Building.
- Overall, visual and olfactory observations did not indicate significant or widespread contamination. The most noteworthy evidence was near Davik's Building and the Vehicle Refuelling area.
- Summary of Key Numerical Findings from Chemical Analysis in Table 3.1.

Table 3.1: Summary of Key Numerical Findings from Chemical Analysis

Contaminant	Maximum Detected Concentration	Guideline Exceeded	Number of Exceedances
Aluminium	180,000 mg/kg	US EPA PRG: 100,000 mg/kg	2 samples
Benzo(a)pyrene	95 mg/kg	Golder SSV: 30 mg/kg	3 samples
Total PAHs	1,100 mg/kg	-	-
TPH	4,200 mg/kg	-	-
Aluminium (leachate)	190,000 µg/L	Drinking Water Standard: 200 µg/L	12 samples
Chromium (leachate)	31 µg/L	EQS: 15 µg/L	1 sample
Copper (leachate)	28 µg/L	EQS: 5 µg/L	6 samples
Benzo(a)pyrene (leachate)	0.049 µg/L	Drinking Water Standard: 0.01 µg/L	2 samples

- In summary, the numerical results highlighted some localised areas of elevated contaminant concentrations, especially for aluminium, benzo(a)pyrene, and leachable metals, though widespread significant contamination was not found.
- Baseline conditions were previously set under the Integrated Pollution Prevention and Control (IPPC) permit based on site investigations done by Wallace Evans in 1992/1993.
- Golder's 2007 sampling included some areas previously used to determine baseline - North Road (Hawkins Heap) and Ponderosa.

- Compared to the baseline data, Golder found lower concentrations of most contaminants in these areas in Table 3.2.

Table 3.2: Comparison of Contaminant Concentrations of North Road and Ponderosa to Baseline Data

Area	Contaminant	2007 Max Concentration	1993 Max Concentration
North Road	Cadmium	1.2 mg/kg	12.2 mg/kg
	Copper	480 mg/kg	1283 mg/kg
	Nickel	220 mg/kg	61067 mg/kg
	Fluoride	94 mg/kg	130963 mg/kg
	Zinc	2300 mg/kg	188 mg/kg
	Chloride	1900 mg/kg	527 mg/kg
Ponderosa	Copper	30 mg/kg	2599 mg/kg
	Lead	52 mg/kg	1171 mg/kg
	Zinc	120 mg/kg	1304 mg/kg
	Cyanide	0 mg/kg	20.3 mg/kg

- Zinc and chloride showed increases in maximum concentrations compared to baseline.
- Overall, the results indicate an improvement in land quality in North Road and Ponderosa since the 1990s baseline study. This is likely attributed to changes in practices and waste management at the site over time.
- In general, comparison with historical baseline data shows lower contaminant levels today in previously impacted areas like North Road and Ponderosa. This demonstrates improved land quality, likely due to better site practices and waste management over time.
- The leachable metal concentrations that exceeded guidelines demonstrate that these contaminants can mobilise from soil into groundwater under laboratory conditions. This indicates they may leach under site conditions too and impact groundwater and surface water quality. Even though on-site groundwater may not be usable, it could act as a pathway to off-site surface waters.
- For example, aluminium was detected in the leachate at concentrations up to 190,000 µg/L, far above the drinking water standard of 200 µg/L. This suggests aluminium could readily leach from soils on site.
- Similarly, benzo(a)pyrene exceeded the health-protective soil guideline of 30 mg/kg in 3 different areas by 2-3 times. This indicates there are localised "hot spots" where benzo(a)pyrene levels pose a health risk if soil is inhaled/ingested by site workers or future users.
- The concentrations show these contaminants are present at levels where they may pose risks under site conditions based on their properties and how they move and interact in the environment.

3.4 2012 04 Anglesey Aluminium Metal Ltd, North Road Benchmark Investigation, Golder Associates Ltd

- Summary Ground Conditions in Table 3.3.

Table 3.3: Summary of Ground Conditions:

Area of Site	Investigation Locations in the Area	Description of Ground Conditions
Western extent North Road Area	TP27 to TP31	- Made Ground: Comprising gravelly sand with cobbles proven to a maximum depth of 2.9 m BGL (1.07 m OD). Made ground was absent from trial pit TP27 located in the extreme north of the area. - Natural Ground/Drift/Weathered Schist: Comprising gravelly sand at trial pit TP27 and proven to a depth of 2.1 mbgl (2.85 m OD) and silt at trial pit TP29 and proven to a depth of 3.0 mbgl (0.97 m OD). Drift deposits were not encountered in any other location. - Bedrock/Schist: Not encountered in any location.
North Road Area	GA02-GA04 and TP09 to TP26	- Made Ground: Identified in all locations. In 9 of the 18 trial pits, the full thickness of made ground was not proven within the excavated depth of between 0.2 and 2.9 mbgl. In the remaining 9 trial pits, made ground was proven to be between 0.3 and 1.8m thick. The maximum thickness of made ground encountered in the boreholes was 5.3m at borehole GA4 located in the Reclamation Yard. Made ground comprised sands, clay and gravel with schist cobbles, some red brick, plastic, and concrete. - Natural Ground/Drift/Weathered Schist: Sandy gravel identified in trial pit TP25 (in the north), and borehole GA03 (in the north) to a maximum depth of 3.0 mbgl (a minimum elevation of 5.40 m OD). This unit was absent from all other locations. - Bedrock/Schist: Identified in all borehole locations. Schist was proven to a depth of 7.2 mbgl (2.00 m OD) at borehole GA03 located in the north of the area.

- Hydrocarbon odours, staining and elevated PID readings indicating possible impacts were noted at 4 locations (TP23, TP24, TP25, GA04) near the Reclamation Yard.
- PID readings ranged from 1.6 ppm to 164 ppm. Readings above 100 ppm were detected at 4 locations (TP22, TP23, TP24, TP26).
- Groundwater elevations were variable across the site, ranging from 4.89 to 6.21 m OD in December and 5.44 to 6.13 m OD in January.
- The variability suggests groundwater is not a continuous body and is influenced by made ground and structures.

- Soil: Benzo(a)pyrene exceeded GAC at 2 locations: TP20 (1.0m) at 12 mg/kg and TP25 (0.5m) at 2.2 mg/kg. Chrysotile asbestos was detected at 3 locations: TP12 (0.5m), TP21 (1.0m), TP24 (0.5m). Other soil contaminants were below GAC for commercial land use.
- Maximum concentrations of most contaminants decreased compared to 1993 baseline data.
- Asbestos was not previously detected but now detected in 3 samples.
- Groundwater Screening: Fluoride, aluminium, barium, copper, and TPH exceeded GAC in some samples. Most exceedances were marginal, in the order of magnitude of the GAC.
- Ground Gas: Maximum methane was 0.1% v/v. Maximum carbon dioxide was 0.7% v/v. Below screening criteria of 1% v/v methane and 1.5% v/v carbon dioxide.
- Overall, the results indicate some low-level contamination related to historical activities, but do not appear to pose unacceptable risk for commercial/industrial land use based on the data collected.

3.5 2013 02 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase B, Golder Associates Ltd.

- Golder conducted site investigation works at the former Anglesey Aluminium Metal Ltd site in Holyhead in July, September and November 2012. The works were to provide data for consideration in the surrender application for Environmental Permit BL1100IX (Phase B).
- The scope comprised cable percussion drilling of 3 boreholes, hand-dug trial pits at 9 locations, installation of groundwater monitoring boreholes, soil and groundwater sampling and analysis, and waste classification and disposal.
- The objectives were to provide representative soil and groundwater data for areas being considered for permit surrender, obtain information on ground conditions, install monitoring boreholes, and classify waste soils.
- 12 intrusive locations were investigated - 3 boreholes drilled to 8-9 m depth and 9 trial pits excavated to 0.4-3.1 m depth.
- Soil samples were collected at a frequency of 1 per 0.5 m or at lithological changes. Groundwater samples were collected on installation of boreholes and during two subsequent monitoring rounds.
- Ground conditions summaries:
    - The intrusive investigation in the Anode Plant area encountered made ground to depths of between 1.5 and 4 metres below ground level (mbgl). The made ground generally comprised reworked natural soils and demolition rubble associated with the construction of the site. Weathered schist bedrock was encountered underlying the made ground to depths of around 4 to 7 mbgl. The weathered profile primarily comprised highly fractured and slightly foliated schist. Competent schist bedrock was encountered beneath the weathered horizon. The profile indicates that some excavation of the superficial cover deposits likely occurred during construction of the Anode Plant.
    - Zone D Stores Area: The single intrusive investigation completed in Zone D encountered made ground to the full investigation depth of 2.1 mbgl. The made ground comprised a heterogeneous mix of concrete, bricks and reworked natural soils. The limited investigation depth did not allow the natural strata to be proven but comparison with nearby historical borehole records suggests the made ground directly overlies schist bedrock in this area. The made ground reflects demolition and construction activities when the stores area was developed.
- Soil samples were tested for metals, TPH, PAHs, PCBs, cyanide, pH, fluoride and sulphide. Groundwater samples were tested for metals, TPH, PAHs, cyanide and fluoride.
- Waste classification testing was completed on selected samples for disposal purposes.

Summary Soil and Made Ground Factual Data of Surrender Site Condition - Anode Plant in north-western part of the site, formerly contained furnace for baking green anodes:

- Total of 12 intrusive locations were investigated, including 3 boreholes and 9 trial pits.

- The boreholes were advanced to depths of 8.4-9.5 m below ground level using cable percussion drilling methods.
- The trial pits were hand dug to depths of 0.4-3.1 m below ground level using a 5-tonne excavator.
- The geology generally comprised made ground overlying weathered schist bedrock. Made ground thickness was variable across the site, ranging from 0.4 m to over 4 m.
- A total of 21 soil samples were collected from the boreholes and trial pits and submitted for laboratory analysis.
- The soil samples were tested for a range of parameters including metals, total petroleum hydrocarbons (TPH), polycyclic aromatic hydrocarbons (PAHs), cyanide, pH, fluoride and sulphide.
- The results showed some elevated concentrations of TPH, PAH and fluoride in certain samples, with maximum concentrations of 1,120,000 µg/kg TPH, 368,000 µg/kg PAH, and 42.6 mg/kg fluoride.
- Concentrations of metals, cyanide and sulphide were relatively low in comparison to generic assessment criteria.

Summary Soil and Made Ground Factual Data of Surrender site condition Zone D - Stores

- Groundwater levels were measured in 3 newly installed boreholes. Levels varied from 2.6 to 5.9 m below ground level.
- A total of 8 rounds of groundwater sampling were completed.
- Samples were collected from the 3 new boreholes as well as 1 existing site location.
- The samples were tested for a range of parameters including metals, TPH, PAHs, cyanide and fluoride.
- Elevated concentrations of TPH and fluoride were detected in some samples.
- Concentrations of metals and cyanide were low compared to generic assessment criteria.

3.6 2013 05 Anglesey Aluminium Metal Limited and Anglesey Aluminium Metal Renewables, Conceptual Site Model, Golder Associates Ltd

- The study area is a former aluminium smelting site in Anglesey, Wales that operated from 1970-2007. It covers around 56 hectares.
- The 56-hectare former smelting site is located near Holyhead in northwest Anglesey, Wales.
- The site is bound by major roads to the north, east and south. Retail and industrial areas lie to the west.
- Penrhos Beach and Bay are 180m north of the site. Residential areas of Holyhead are 800m northwest.
- The site topography is generally flat, with elevations of 7.3 to 9.3m above sea level.
- Main site buildings are clustered in the centre. The Potlines, Metal Products, and Rectifier Yard facilities are located here.
- An unsurfaced area called North Road extends along the northeastern part of the site.
- Around 24 hectares of the site have soil, grass or gravel cover. The remainder is buildings/concrete.
- Three drainage systems are present: trade effluent, sewage effluent, and stormwater. Onsite oil-water interceptors are connected.
- Below ground structures include building foundations, process equipment, pipelines, and drainage systems.
- The site was farmland before the smelter was constructed in 1970 involving major excavation and earthworks.
- Key raw materials were alumina, coke and pitch. Aluminium was produced via electrolysis in Potlines.
- Ancillary areas included fuel/oil storage, vehicle refuelling, waste handling, railway, workshops, offices etc.
- Various documented and anecdotal releases of materials to ground occurred over decades of operation.
- Several areas of potential environmental concern were identified based on historical activities, including storage of fuels and oils, a rectifier yard, vehicle refuelling, waste handling areas etc.
- Intrusive investigations have been carried out in the study area since the 1990s. These found contaminants in soil and groundwater, including metals, fluoride, PAHs, PCBs, TPH and NAPL.
- Topography: The site is relatively flat with elevations ranging from 7.3 to 9.3m above sea level.
- Hydrology: Average annual rainfall is 841mm based on data from a weather station 6km southeast of the site. Infiltration rates vary across the site based on surface cover - lower were concrete/buildings, higher in open areas. Nearest surface water is Penrhos Beach and Bay in Holyhead Bay 180m north of the site.

- Geology:
  - Published pre-construction maps show the site is mostly underlain by glacial till deposits over schist bedrock.
  - Bedrock outcrops in the Pitch Tanks, Vehicle Refuelling, and North Road areas. A fault crosses the site.
  - Pre-construction boreholes confirmed the published geology.
  - Post-construction data shows made ground over fractured schist bedrock across most of the site.
  - This indicates glacial deposits were largely removed during site construction excavations.
  - Made ground thickness varies from 0.1 to 9.5m. Natural glacial deposits remain mainly in the Rectifier Yard and North Road.
  - The schist exhibits fracturing and quartz veins. Variable weathering is present in the shallow subsurface.
  - Bedrock originally lay above site level in the Pitch Tanks and Vehicle Refuelling areas before being excavated down during construction.
- The bedrock and superficial deposits are designated as Secondary B and Secondary Undifferentiated aquifers respectively.
- Two hydrogeological units are present - superficial deposits and bedrock.
- Groundwater is encountered at 1.5-4m depth in both units. It is unconfined.
- Superficial deposits comprise made ground and glacial drift. Estimated porosity 25-50%.
- Hydraulic conductivity of superficial deposits is 1.95×10^{-5} to 5.23×10^{-6} m/s based on in-situ testing.
- Bedrock comprises fractured schist. Porosity estimated at 0-10% in unweather rock, 30-60% in weathered rock.
- Hydraulic conductivity ranges over several orders of magnitude from 1.57×10^{-4} to 4.39×10^{-10}m/s based on in-situ testing.
- Groundwater flow is primarily horizontal in both units based on measured water levels.
- Water levels range from 4-5m OD in northwest to 6-7m OD in southeast. Flow is northwards towards the sea.
- Below ground structures like foundations and drains influence local flow.
- Groundwater exhibits variations in response to tidal and rainfall events based on continuous monitoring data.
- Areas of Potential Concern (APCs):
  - 13 APCs were identified based on historical site activities like fuel storage, waste handling etc.
  - Non-Aqueous Phase Liquids (NAPL).

- NAPL containing PAHs, PCBs and TPH found in 3 APCs - Pitch Tanks, Rectifier Yard, Vehicle Refuelling.

- Pitch Tanks - up to 4.55m thick DNAPL observed containing 11-41% PCB.
- Rectifier Yard - up to 0.25m thick LNAPL observed, likely transformer oil.
- Vehicle Refuelling - LNAPL observed but limited thickness precluded sampling.
- Contaminants in Soil:
  - Elevated TPH, PAHs and metals present in soils across various APCs.
  - Pitch Tanks - Avg TPH 235 mg/kg, max 2,350 mg/kg. Avg PAH 212 mg/kg, max 6,170 mg/kg. Avg PCB 3 mg/kg, max 35 mg/kg.
  - Rectifier Yard - Avg TPH 309 mg/kg, max 2,250 mg/kg. Avg PAH 0.6 mg/kg, max 3.9 mg/kg.
  - Vehicle Refuelling - Avg TPH 563 mg/kg, max 8,140 mg/kg. Avg PAH 2.2 mg/kg, max 45 mg/kg.
- Contaminants in Groundwater:
  - TPH, PAHs, PCBs and metals detected in groundwater in various APCs.
  - Pitch Tanks - TPH up to 1,480 mg/L. PAH up to 16.8 mg/L. PCB up to 364 µg/L.
  - Rectifier Yard - TPH up to 2,910 mg/L. PAH up to 672 µg/L. PCB up to 197 µg/L.

3.7 2013 06 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Garage Area, Golder Associates Ltd

- The site topography consists of low-lying relatively flat land sloping down to the north and east towards Holyhead Bay and the Irish Sea, with elevations ranging from 3.5 to 9.3 mOD, except for a higher area in the northeast at 9 mOD and raised embankments to the south.
- Surrender site condition – Garage Area:
    - In 2007, 3 boreholes and 1 trial pit were advanced to 0.3-1.38m depth.
    - 5 soil samples were analysed for metals, cyanide, TPH, PAH, etc. No groundwater sampling. Results showed elevated TPH, PAH, metals, cyanide in soils.
    - In 2012-2013, 9 boreholes and 3 probe holes were installed. Groundwater monitored.
    - 43 soil samples analysed for TPH, PAH, metals, etc. Groundwater sampled up to 8 times and analysed for TPH, PAH, VOCs, metals.
    - Soil samples analysed for metals, TPH, PAHs, etc. Elevated levels of some metals, TPH up to 8,140 mg/kg, PAHs up to 16 mg/kg.
    - Groundwater samples had elevated TPH up to 77,800 µg/l, PAHs up to 125 µg/l, VOCs like chlorobenzene up to 148 µg/l.
    - Trace NAPL detected in 2 wells but no widespread significant NAPL layer.
    - Groundwater flow direction is northwest, but levels suggest localised high in Garage area.
    - Field groundwater parameters indicate conditions suitable for biodegradations.
- Some contaminants exceeded background levels but were below generic screening criteria, except TPH exceeded screening levels in 2 locations.
- Detailed quantitative risk assessments completed for soil, groundwater and NAPL. The soil risk assessment concluded no unacceptable risk to human health from TPH. The groundwater risk assessment concluded no unacceptable risk to controlled waters or human health. The NAPL assessment noted no widespread NAPL layer present and negligible risks under current conditions.
- Overall, despite some contaminant detections, the assessments indicate the soil and groundwater are in a satisfactory state in the Garage area.

3.8 2013 08 Summary of Investigation in the Pitch Tanks Area, Anglesey Aluminum Metal Ltd, Pernhos Works, Holyhead, Golder Associates Ltd

- Evidence of historical impacts was identified in the Pitch Tanks area during the investigations.
- Activities Conducted in the Pitch Tanks Area.
- The Pitch Tanks area was used for storage of pitch in 3 above ground storage tanks (ASTs) prior to use in the adjacent Green Carbon Plant.
- The ASTs and pipelines were heated via a closed hot oil system that originally contained PCB oil until it was changed out in the 1970s.
- Historical leaks/spills from the hot oil system likely occurred based on:
- Partial draining/replacement of PCB oils in 1972 and 1976. Estimated 5-15% PCB oil remained after 1976 drainage.
- A fire occurred in 1978 potentially due to a leak.
- Final drain of system in 1982 prior to decommissioning.
- 3 phases of intrusive investigation were completed in the Pitch Tanks area in 2012-2013.
- July 2012 - 2 boreholes, 3 probe holes. NAPL was observed, so 9 more boreholes were immediately installed. 2012 - 8 more boreholes installed. April 2013 - 4 more boreholes installed.
- 23 monitoring wells and 3 probe holes have been installed. Sampling and monitoring conducted.
- NAPL containing PCBs identified in 12 boreholes/probe holes. Free phase measured up to 4.49 m thickness.
- Elevated dissolved phase PCBs, TPH, PAHs detected in groundwater samples within Pitch Tanks area.
- Conceptual Site Model Developed:
    - NAPL source present containing ~11-41% PCBs. Density indicates DNAPL. Entry likely prior to 1982.
    - Dissolved phase groundwater impacts present within Pitch Tanks area.
    - Vapor, soil exposure pathways also present.
    - Migration pathways assessed - groundwater, NAPL in fractures, subsurface features.
    - Human health, controlled waters, ecological receptors identified.
- Generic and quantitative human health risk assessment completed.
    - Concluded acceptable risks assuming continued commercial/industrial use.
    - No remedial targets set based on human health.
- Controlled Waters Risk Assessment

- Modelling suggests achieving remedial PCB goals in groundwater difficult without NAPL removal, which is likely infeasible.
- Uncertainty about whether drift or bedrock pathways dominate migration to coast.

3.9 2014 01 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase C, Golder Associates Ltd

- The site is located at the former Penrhos Works near Holyhead in Anglesey, Wales.
- It is bound by roads, Penrhos Retail Park and a site operated by Anglesey Aluminium Metal Renewables.
- The topography is relatively flat, sloping down towards the coastline.
- The geology comprises schist bedrock overlain by variable thicknesses of superficial deposits like clay, silt and gravel.
- Groundwater is encountered at 1-4m below ground level in the superficial deposits and schist. It flows towards the Irish Sea.
- Summarising soil and groundwater quality data in Table 3.4:

Table 3.4: Summary of Soil and Groundwater Quality Data

Area	Number of Boreholes	Key Soil Findings	Key Groundwater Findings
Rectifier Yard	11 advanced, 5 monitored	- TPH max 2,250 mg/kg	- TPH max 3,400,000 µg/L
		- PAH max 3.91 mg/kg	- PAH max 672 µg/L
		- PCB max 0.29 mg/kg	- PCB max 197 µg/L
Potlines	7 advanced, 5 monitored	- Mineral oil max 7,400 mg/kg	- Chromium max 8.10 µg/L
		- Copper max 239 mg/kg	- Copper max 7 µg/L
		- Vanadium max 36 mg/kg	- Zinc max 6.8 µg/L
Remelt/Ponderosa	11 advanced, 7 monitored	- Aluminum max 24,900 mg/kg	- TPH max 5,820 µg/L
		- PAH max 56.8 mg/kg	- PAH max 238 µg/L
			- PCB max 13 µg/L
Pitch Tanks/Green Carbon	27 advanced, 23 monitored	- TPH max 2,350 mg/kg	- TPH max 3,400,000 µg/L
		- PAH max 6,170 mg/kg	- PAH max 16,800 µg/L
		- PCB max 35.5 mg/kg	- PCB max 364,000 µg/L
Sewage Treatment Plant	3 advanced, 1 monitored	- TPH max 162 mg/kg	- Aluminum max 1,625 µg/L
			- Copper max 11 µg/L

- Summary explaining the key points from the table summarizing soil and groundwater quality data:
- Rectifier Yard: The key soil contaminants detected were TPH, PAH, and PCB, with maximum concentrations only slightly elevated compared to background levels. Groundwater contained sporadic low-level detections of TPH, PAH, and PCB above background concentrations.
- Potlines: Elevated mineral oil, copper, and vanadium were detected in some soil samples compared to baseline, likely from historical activities. Groundwater contaminants like chromium, copper, and zinc were generally at or below background concentrations.

- Remelt/Ponderosa: Maximum soil concentrations of aluminium and PAH were slightly above background levels. Groundwater contained sporadic low-level detections of TPH, PAH and PCB above background.
- Pitch Tanks/Green Carbon: No baseline data but high levels of TPH, PAH and PCB detected in soil due to historical releases. Similarly, high detections of these contaminants in groundwater.
- Sewage Treatment Plant: No baseline data but soil quality was within background concentrations. Groundwater contained slightly elevated aluminium and copper compared to background.
- In summary, the table shows that soil and groundwater quality across the areas was generally within baseline conditions, with only sporadic or isolated instances of contaminants being detected slightly above background. The exceptions were Pitch Tanks/Green Carbon where historical releases prior to the permit led to high levels of hydrocarbons and PCB in soil and groundwater. Overall, there is limited evidence of deterioration in conditions due to the permitted activities.

3.10 2015 12 Anglesey Aluminium Metal Ltd, Factual Report on Further Intrusive Investigation of Pitch Tanks and Anode Bake Area, Golder Associates Ltd

- A total of 14 boreholes were advanced by rotary drilling to depths of up to 22.2m below ground level (bgl). 8 boreholes were located around the periphery of the Pitch Tanks/Green Carbon Plant to target depths of up to 15m bgl based on DNAPL predictive modelling. 2 boreholes in the Anode Bake area were advanced to 20m bgl. 4 boreholes were located in the Pitch Tanks source area.
- Ground Conditions summaries of Pitch Tanks, Anode Bake, Pitch Tanks Source Area, Storm Drain Investigation in Table 3.5.

Table 3.5: Summary of Ground Conditions - Pitch Tanks, Anode Bake, Pitch Tanks Source Area, Storm Drain Investigation

Area of Site	Boreholes in the Area	Description of the Ground Conditions
Pitch Tanks/Green Carbon Periphery	GA15/01 GA15/02 GA15/03 GA15/04 GA15/05 GA15/06 GA15/07 GA15/08	Hard standing: Typically consisting of concrete or tarmac, ranging in thickness from 0.03 – 0.26 m. Made Ground: Typically comprising silty/sandy gravels ranging in thickness from 0.23 – 1.37 m. Natural Ground: bluish grey, lightly-highly fractured mica Schist bedrock. The shallowest depth at which the bedrock was encountered was 0.45 m bGL in GA15/01. Thickness not proven.
Anode Bake	GA15/09 GA15/10	Ground surface: concrete at GA15/09 and gravel at GA15/10. Made Ground: Typically comprising silty/sandy gravel, ranging in thickness from 1.60 – 2.20 m. Drift Deposits: Typically comprising sandy gravels and some clay extending to a maximum 15.50 m bGL at GA15/10. Natural Ground: bluish grey, lightly-highly fractured mica Schist bedrock. The shallowest depth at which the bedrock was encountered was 12.70 m bGL at GA15/09.
Pitch Tanks Source Area	GA15/11 GA15/12 GA15/13B GA15/14	Hard standing: Consisting of concrete at all locations, ranging in thickness from 0.18 – 0.35 m. Made Ground: Typically comprising clayey/sandy gravel and cobbles ranging in thickness from 0.25 – 1.80 m. Natural Ground: bluish grey, lightly-highly fractured mica schist bedrock. The shallowest depth at which the bedrock was encountered was 0.60 m bGL in GA15/13. Thickness not proven.
Storm Drain Investigation	Trench 1 Trench 2	Hard standing: Comprising concrete ranging in thickness from 0.12 – 0.18 m.

		Made Ground: typically comprising silty sandy slightly clayey gravel and cobbles with concrete fragments with clayey gravelly sand and occasional cobbles below 0.9 m bGL. (Trench 1 was limited to 0.4 m bGL due to concrete). Pipe Fill: fine, soft sand surrounding the pipe (top of pipe at 1.5 m bGL).

- In addition, two trenches were excavated to 2m bgl to investigate the fill material around a stormwater drain in the Pitch Tanks area.
- The subsurface comprised made ground overlies natural geology of fractured schist bedrock. Up to 15.5m of drift deposits were encountered in the Anode Bake area.
- NAPL was detected in 3 of the new boreholes and 9 existing boreholes during groundwater monitoring. Thicknesses ranged from 0.01 to 6.9m.
- PID screening recorded up to 49.8 ppm in the Pitch Tanks periphery and 32.6 ppm in the Pitch Tanks source area.
- Laboratory analysis detected average concentrations up to 91,861 mg/kg TPH, 396.7 mg/kg PAH and 12,088 mg/kg PCB in soils. The highest concentrations were in the Pitch Tanks source area.
- NAPL composition analysis indicated the presence of lubricating oils/hydrocarbons in the C13-C30 carbon range.

3.11 2016 03 Renewable Energy Plant (REP) Former Anglesey Aluminium Works Site Investigation Brief, LK Consult (LKC)

This is a site investigation brief prepared by LK Consult for a proposed renewable energy plant to be built on the former Anglesey Aluminium Metal Works site in Holyhead:

- The area within the current planning application boundary, which spans approximately 38 hectares, has been subdivided into 4 areas based on historical land use patterns in Table 3.6.

Table 3.6: Subdivision of Areas within Planning Application Boundary Based on Historical Land Use

Area	Size (hectares)	Former Use	Proposed Use
Area 1	19.4	Undeveloped land, woodland/wetlands	Non-developed woodland Landscaped area
Area 2	11	Reclamation yard, waste storage	Boiler Buildings, Biomass Storage
Area 3	5.3	Contractor's yard	Office Buildings, Fire Station, Car Park and Contractor’s Yard.
Area 4	2.1	Rectifier yard, electrical transformers	Electrical Transformer Zone.

- A Phase 1 study identified potential contamination risks from historical industrial activities.
- An intrusive Phase 2 investigation is proposed to assess risks to human health and groundwater.
- This will involve drilling boreholes, trial pits, installing monitoring wells, and sampling soil, groundwater, and drainage ditches.
- Samples will be analysed for metals, asbestos, hydrocarbons, PAHs, PCBs, etc.
- The recommended investigation work for each site area is summarised in Table 3.7.

Table 3.7: Recommended Investigation Work by Site Area.

Site Area	Window Sample Boreholes	Trial Pits	Rotary Boreholes (Open holes)	Rotary Boreholes (Cored)	Approx. Soil Sample Numbers
1	5	7	-	-	20
2	24	18	10	11	95
3	10	10	5	5	45
4	17	-	7	7	60

3.12 2016 03 Anglesey Aluminium Holyhead, Phase 1 Preliminary Risk Assessment, LK Consult (LKC)

During the site reconnaissance visits conducted between January 15th and January 21st, 2016:

- Several observations and features were identified in Area 1 (Undevelopable Land):
  - This area comprises predominantly heavily vegetated woodland and a wetland area along the north, northeastern, and eastern boundaries of the site.
  - Topographically, a significant portion of this area is elevated above the main former Anglesey Aluminium Metal (AAM) Works site, with the exception of a grassed and marshy area to the north.
  - Limited access to much of this area due to the presence of dense vegetation and marshy terrain in the northern part.
  - An adjacent landscaped grassed area, located next to the main access road off the A5, is mostly level with a raised mound near the inner site boundary fence. This grassed area appears to be regularly maintained.
  - Observations of stormwater drainage was noted within the grassed area.
  - A drainage ditch was observed in the northern part of the landscaped grassed area, with additional drainage ditches noted to the north of the marshy area.
  - Moderate flow was observed in the drainage ditches to the north of the marshy area, and these drainage channels appeared to lead into a chamber near the northern site boundary, with confirmation provided by stormwater drainage plans.
  - Evidence of animal burrows were observed in accessible parts of the vegetated area.
  - No significant contamination was observed on the surface in this area.
  - Several existing boreholes were located in this area and found to be in good condition, including GABH-D-01, GABH-D-02, GABH-D-03, GABH-D-05 (both shallow and deep), and GABH-D-06 (both shallow and deep).
  - There was no evidence of free product or hydrocarbon odours identified in the monitoring wells within this area.
  - No evidence of vegetative stress was observed in this area.
  - It was noted that only a limited portion of this area is currently accessible to vehicles and site investigation equipment.
- Area 2 (North Road area):
  - This area is characterised by rough, open ground with some shrubs and small trees. It is bounded to the north and east by a drainage ditch and the inner site boundary fence.
  - Vegetation cover increases toward the north and east, except for the former Reclamation Yard in the northern part, which has very little vegetation.

    - Topographically, the area is generally level, with a slightly raised area in the north and localised undulating ground.
    - The former Reclamation Yard, located in the northern part of the area, is an unsurfaced area and is separately fenced off from the rest of the North Road area.
    - Observations of several drainage ditches running to the boundary drain in the northeast were made, but no significant flow was observed, and the ditches appeared stagnant.
    - Several areas with standing water were observed across the area, though it was unclear if this was due to localised waterlogging.
    - The concrete floor slab of the former Davik's Building was present in the southeast of the area.
    - Smaller concrete slabs to the southwest of the former Davik's Building were reported to be related to a former fire training area.
    - A small bund, seemingly constructed of gravel, runs along the western boundary of the North Road area adjacent to Forest Drive.
    - No evidence of significant contamination was observed on the surface in this area.
    - Several existing boreholes were located in this area and found to be in good condition, including GA02, APC06, NR02, APC05, GA03, and GA04.
    - There was no evidence of free product or hydrocarbon odours identified in the monitoring wells within this area.
    - No evidence of vegetative stress was noted in this area.
    - No access restrictions were noted, should future site investigation be required.
- Area 3 (Area around Rectifier Yard):
    - This area comprises a vacant gravelled area, a tarmacadam road, and a vegetated area to the northeast of the Rectifier Yard. Additionally, there is a former Contractor's Yard and a grassed area to the southeast.
    - The vegetated area is currently outside of the inner site boundary fence, and only light vegetation was visible.
    - The former Contractor's Yard consists of hardstand areas, while the rest of this area, excluding the tarmacadam road section, is soft covered.
    - A live high-voltage cable enters the site from the southeast of the grassed area and runs to the Rectifier Yard, with the cable's route marked by posts on the surface.
    - Access to the Contractor's Yard was not possible during the site reconnaissance visits, but several shipping containers were noted to be present within this area.
    - The Contractor's Yard was fenced off separately from the rest of the AAM site.
    - Evidence of surface water drainage, including manholes and gully covers, was noted in this area.

  - No evidence of significant contamination was observed on the surface in this area.
  - One existing borehole, GA01, was located in this area and found to be in good condition.
  - There was no evidence of free product or hydrocarbon odours identified in the two monitoring wells within this area.
  - No evidence of vegetative stress was noted in this area.
- Area 4 (Rectifier Yard and Adjacent boundary):
  - The Rectifier Yard is securely fenced and can only be accessed through the control building in the east and a locked double gate on the western side.
  - The surface cover in the yard is predominantly gravel, with a tarmacadam road running through the northern part of the area.
  - The yard contains a significant amount of both above-ground and below-ground electrical infrastructure, including rectifiers and transformers. During the visit, only a small section of the above-ground apparatus in the south of the yard was operational.
  - A control building that includes offices is present on the northeastern boundary of the yard. It is no longer permanently manned and is only used occasionally by the remaining site personnel.
  - Another building is located on the southeastern boundary and is understood to belong to National Grid. Access to this building was not possible during the site visit.
  - Moderate hydrocarbon surface staining was noted in several locations within the Rectifier Yard, primarily in proximity to the transformers.
  - Multiple existing boreholes were located and found to be in good condition within this area. These boreholes include AAM-REC-11 (shallow and deep), AAM-REC-17 (shallow and deep), AAM-REC-09, AAM-REC-15 (shallow and deep), AAM-REC-06, GABH05, AAM-REC-03, GABH06, GABH07, and AAM-REC-01.
  - Evidence of free product was noted in one existing borehole, AAM-REC-11 (shallow), within the Rectifier Yard. Hydrocarbon odours were also detected in AAM-REC-11, AAM-REC-06, and AAM-REC-03.
  - Access restrictions and limited vehicular access to some areas of the Rectifier Yard may be in place should further investigation work be required.
- The monitoring of the boreholes discovered during the site reconnaissance visits are detailed in Table 3.8.

Table 3.8: Monitoring Details for Boreholes Discovered During Site Reconnaissance

Site Area	BH REF	Depth to top of groundwater (mbgl)	Depth to base (mbgl)	Free Product Detected	Hydrocarbon Odours
1	GABH-D-01	2.5	11.6	No	No
	GABH-D-02	1.35	3.15	No	No
	GABH-D-03	4.2	11.5	No	No

Site Area	BH REF	Depth to top of groundwater (mbgl)	Depth to base (mbgl)	Free Product Detected	Hydrocarbon Odours
	GABH-D-05 (shallow)	4.0	10.2	No	No
	GABH-D-05 (deep)	4.6	17.1	No	No
	GABH-D-06 (shallow)	5.35	6.6	No	No
	GABH-D-06 (deep)	5.3	12.9	No	No
2	GA02	2.1	3.95	No	No
	APC06	1.6	6.35	No	No
	NR02	1.2	3.3	No	No
	APC05	0.9	5.95	No	No
	GA04	2.5	5.45	No	No
3	GA01	1.7	4.2	No	No
4	AAM-REC-11 (shallow)	1.8	3.65	Yes	Yes
	AAM-REC-11 (deep)	1.85	9.0	No	No
	AAM-REC-17 (shallow)	1.8	4.5	No	No
	AAM-REC-09	1.85	2.85	No	No
	AAM-REC-15 (shallow)	1.35	5.5	No	No
	AAM-REC-06	1.9	5.15	No	Yes
	GABH05	1.8	2.75	No	No
	GABH06	1.85	4.85	No	No
	GABH07	1.7	4.2	No	No
	AAM-REC-03	1.55	3.00	No	Yes
	AAM-REC-01	1.85	3.35	No	No

- Potential contaminants affecting the current study area include petroleum hydrocarbons, heavy metals, PAHs, aluminium, fluoride, PCBs, sulphate, asbestos, and gas (carbon dioxide and methane).
- The main source of these contaminants was the former processes and activities undertaken at the AAM Works during its operation.
- These contaminants may pose a risk to site users via dermal contact/inhalation pathways and explosion, controlled waters via migration through permeable strata/preferential pathways, buildings and structures through direct contact and explosion, and water pipes via direct contact.

- Preliminary contamination conceptual models have been produced for each site area. These models have identified various risks associated with the contaminants, as summarised for each area:
    - Risk table for land at Area 1 (Undevelopable Land) in Table 3.9.

Table 3.9: Risk Summary for Area 1 (Undevelopable Land)

Pollutant Linkage	Risk	Further Action Required
Non-volatile contaminants posing a risk to site users via dermal contact and inhalation (of soil, dust and fibres).	**Moderate (localised)** (ACM, metals, PAHs, hydrocarbons)	-Localised intrusive investigation required in accessible managed landscaped area near site entrance.
	Low (Aluminium, fluoride)	
Volatile contaminants posing a risk to site users via the inhalation of vapours.	**Low**	-No specific intrusive investigation work for volatile contamination required.
No pollutant linkage identified as no receptor in this area.	N/A	-No further work required.
Mobile contamination posing a risk to Irish Sea via the migration through preferential pathway (storm water drainage and interconnected ditches)	**Low**	-No specific investigation works required in this area but precautionary drainage sampling proposed to confirm low risk to Irish Sea.
Mobile contamination posing a risk to controlled waters via the migration through permeable strata.	**Low**	-No specific investigation works required in this area but wider groundwater and drainage sampling proposed to confirm concentrations leaving site.
No pollutant linkage identified as no receptor in this area.	N/A	-No further work required.

- Risk table for land at Area 2 (North Road area) in Table 3.10.

Table 3.10: Risk Summary for at Area 2 (North Road area).

Pollutant Linkage	Risk	Further Action Required
Non-volatile contaminants posing a risk to site users via dermal contact and inhalation (of oil, dust and fibres).	**Moderate (localised)** (ACM, metals, PAHs, hydrocarbons,	-Confirmatory intrusive investigation to confirm no further hotspots.

	aluminium, fluoride)	
Volatile contaminants posing a risk to site users via the inhalation of vapours.	**Low**	-No specific intrusive investigation work for volatile contamination required.
Gas posing a risk to buildings and site users via the migration of gas into building causing explosion and asphyxiation.	**Low**	-No further work required.
Mobile contamination posing a risk to Irish Sea via the migration through preferential pathway (storm water drainage and interconnected ditches).	**Moderate**	-Sampling of storm water drainage and ditches onsite and prior to outfall into Irish Sea (leaving site in Area 1).
Mobile contamination posing a risk to controlled waters via the migration through permeable strata.	**Low**	-Confirmatory intrusive investigation to confirm no further hotspots. -Groundwater sampling from existing and new monitoring wells.
Organic contaminants posing a risk to waterpipes.	**Moderate**	-Confirmatory intrusive investigation to confirm no further hotspots. -Consultation with water supplier over provision of new potable water pipes. -Barrier pipe is likely to be required.

- Risk for land at Area 3 (Southeast area) in Table 3.11.

Table 3.11: Risk Summary for Area 3 (Southeast Area)

Pollutant Linkage	Risk	Further Action Required
Non-volatile contaminants posing a risk to site users via dermal contact and inhalation (of oil, dust and fibres).	**Low** (ACMs) **Moderate** (Metals, PAHs, hydrocarbons, aluminium, fluoride)	-Confirmatory intrusive investigation to prove no further hotspots.
Volatile contaminants posing a risk to site users via the inhalation of vapours.	**Moderate**	-No specific intrusive investigation work for volatile contamination required.
Gas posing a risk to buildings and site users via the migration of gas into building causing explosion and asphyxiation.	**Low**	-No further work required.
Mobile contamination posing a risk to Irish Sea via the migration through preferential pathway (storm water drainage and interconnected ditches).	**Moderate**	-Sampling of storm water drainage and ditches onsite and prior to outfall into Irish Sea (leaving site in Area 1).

Mobile contamination posing a risk to controlled waters via the migration through permeable strata.	**Low**	-Confirmatory intrusive investigation to confirm no further hotspots. -Groundwater sampling from existing and new monitoring wells.
Organic contaminants posing a risk to waterpipes.	**Moderate**	-Confirmatory intrusive investigation to confirm no further hotspots. -Consultation with water supplier over provision of new potable water pipes. -Barrier pipe is likely to be required.

- Risk for land at Area 4 (Rectifier Yard):

Table 3.12: Risk Summary for Area 4 (Rectifier Yard):

Pollutant Linkage	Risk	Further Action Required
Non-volatile contaminants posing a risk to site users via dermal contact and inhalation (of oil, dust and fibres).	**Low** (ACMs, metals, PAHs, hydrocarbons, aluminium, fluoride)	-Confirmatory intrusive investigation to prove no further hotspots.
Volatile contaminants posing a risk to site users via the inhalation of vapours.	**Low**	-No specific intrusive investigation work for volatile contamination required but delineation and remediation of known hotspot proposed.
Gas posing a risk to buildings and site users via the migration of gas into building causing explosion and asphyxiation.	**Low**	-No further work required.
Mobile contamination posing a risk to Irish Sea via the migration through preferential pathway (storm water drainage and interconnected ditches).	**Moderate**	-Further investigation to delineate groundwater contamination in this area and remediation of NAPL to ALARP principles (betterment). -Sampling of storm water drainage onsite and prior to outfall into Irish Sea (leaving site in Area 1).
Mobile contamination posing a risk to controlled waters via the migration through permeable strata.	**Moderate** (Secondary B Aquifer) **Low** (Irish Sea)	-Further investigation to delineate groundwater contamination in this area and inform remediation of NAPL to ALARP principles (free product removal and betterment). -Groundwater sampling from existing and new monitoring wells.
Organic contaminants posing a risk to waterpipes.	**Moderate**	-Further investigation to delineate groundwater contamination and

		remediation of NAPL to ALARP principles. -Precautionary testing of existing potable water supplies. -Consultation with water supplier over provision of new potable water pipes. -Barrier pipe is likely to be required.

- Based on the above work LKC considers that there is a potential for unexpected contamination to be present on site. Further investigation and assessment will be required for the identified potential pollutant linkages.

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- The subsurface conditions at the site were primarily characterised by a layer of made ground overlying superficial deposits. These deposits consisted of sandy clay, clayey gravel, and localised bands of sands and gravels.
- The underlying geological formation was predominantly weathered schist, often encountered as clayey gravels or cobbles composed of mica schist. The schist bedrock displayed a high mica content, with significant fracturing observed in the shallow bedrock layers. Deeper within the schist bedrock, a lesser degree of fracturing was noted, primarily characterised by sub-horizontal fractures.
- Summary of ground conditions for each site area:
    - Ground conditions for Area 1 in Table 3.13.

Table 3.13: Summary of Ground Conditions - Area 1

Depth to Top of Strata (mbgl)	Depth to Base of Strata (mbgl)	Thinness (m)	Description
0.0	0.1 to 2.6	0.1 to 2.6	MADE GROUND 1 (MG1): Grey and brown sandy clay or clayey sand with gravel predominately of schist. Anthropogenic inclusions such as pottery, ash, metal and wood present in most locations. Made ground evident in all locations in Area 1.
0.1 to 2.8	0.9 to 3.7	0.5 to 1.6	SAND: Grey and brown clayey gravelly SAND occasionally silty. Evident in all exploratory locations in Area 1 apart from WS106, WS107, WS108, TP127 and TP128.
0.2 to 1.3	2.7 to 3.3	1.5 to 2.6	CLAY: Soft to firm sandy and gravelly CLAY. Evident in WS107, WS109 and WS110.
0.2 to 4.2	>1.4 to >4.4	>0.2 to >1.3	POSSIBLE WEATHERED SCHIST: Recovered as clayey gravel and cobbles of mica schist. Encountered in WS107, WS108, WS109, WS110 and TP125

- Ground conditions for Area 2 in Table 3.14.

Table 3.14: Summary of Ground Conditions - Area 2

Depth to Top of Strata (mbgl)	Depth to Base of Strata (mbgl)	Thinness (m)	Description
0.0 to 3.2	0.05 to 4.4	0.05 to 4.2	MADE GROUND 1 (MG1): Grey and brown sandy clay or clayey sand, often gravelly to very gravelly. Anthropogenic inclusions such as refractory brick fragments, slag, pottery, ash, metal and wood present in most locations. Made ground evident in all locations in Area 2 apart from LKRBH12, LKRBH16, TP113, TP114, TP118, TP
0.0 to 0.2	0.1 to 0.3	0.1 to 0.3	MADE GROUND 2 (MG2): Brown and black organic clayey topsoil. Made ground type only evident in LKRBH12, LKRBH13, LKRBH14, LKRBH15, LKRBH16, LKRBH17 and TP111
0.1 to 1.0	0.3 to 1.7	0.3 to 1.2	MADE GROUND 3 (MG3): Black and brown ashy gravelly sand with frequent slag, clinker and other anthropogenic inclusions. Made ground type only evident in TP103, TP107, TP108, TP116, WS115, WS119, WS120, WS125 and WS129.
0.3 to 4.5	1.0 to 8.0	0.6 to 6.2	CLAY: Firm to stiff gravelly and sandy CLAY. Evident in exploratory locations LKRBH2, LKRBH3, LKRBH4, LKRBH5, LKRBH6, LKRBH8, LKRBH9, LKRBH10, LKRBH11, KRBH12, LKRBH19, LKRBH24, TP113 and WS130.
1.0 to 1.8	2.3 to 3.5	0.5 to 2.5	SILT – Light grey gravelly SILT. Evident only in LKRBH11 and TP113.
0.8 to 6.5	1.6 to 8.0	0.1 to 3.5	SAND and GRAVEL: Light grey brown silty SAND and GRAVEL. Evident in LKRBH02, LKRBH03, LKRBH04, LKRBH05, LKRBH06, LKRBH11, LKRBH12, TP112, TP113, TP115, TP116, TP117
0.05 to 8.0	0.3 to 10.5	0.05 to 3.6	WEATHERED SCHIST: Recovered as greenish grey clayey gravel and cobbles of mica schist. Encountered in all rotary boreholes.
0.3 to 10.5	> 5 to 18	>2.2 to 10.3	SCHIST BEDROCK

			Encountered in all rotary boreholes.

- Ground conditions for Area 3 Table 3.15.

Table 3.15: Summary of Ground Conditions - Area 3

Depth to Top of Strata (mbgl)	Depth to Base of Strata (mbgl)	Thinness (m)	Description
0.0 to 0.4	0.2 to 1.4	0.2 to 1.4	MADE GROUND 1 (MG1): Grey and brown sandy clay or clayey sand with gravel predominately of schist. Anthropogenic inclusions such as brick, plastic, timber, ash, rare coal and metal present in TP132, TP138, TP139, WS113, WS112, WS123, WS135, WS134 and WS131. Made ground type evident in all locations in Area 3 except WS111, WS124, WS132 and WS133.
0.0 to 0.3	0.3 to 0.5	0.2 to 0.4	MADE GROUND 2 (MG2): Dark brown organic slightly sandy clay with rare brick and concrete fragments. Only present in WS111-WS113 and WS133. Ash, coal and clinker also present in WS111. Tarmacadam fragments noted in WS112 and WS113.
0.2 to 0.4	0.5 to > 0.5	0.3 to >0.1	CONCRETE Only present in TP137 and WS133-WS135.
0.4 to 1.7	3.8 to 2.4	2.8 to > 2.0	SAND: Grey and brown gravelly SAND or sandy GRAVEL occasionally clayey. Only evident in WS131, TP140, TP139, TP138, TP132 and TP131.
0.1 to 1.1	0.8 to > 4.1	0.5 to > 3.6	CLAY: Soft to firm grey and brown sandy and gravelly CLAY. Evident in all locations in Area 3 apart from WS133, WS134, WS135, WS131, TP139, TP140 and TP137.
0.8 to 3.0	1.5 to 9.0	0.5 to 7.0	WEATHERED SCHIST: Recovered as greenish grey clayey gravel and cobbles of mica schist. Encountered in all rotary boreholes.
1.5 to 9.0	> 12 to 13.5	>3.0 to >10.5	SCHIST BEDROCK Encountered in all rotary boreholes.

- Ground conditions for Area 4 in Table 3.16.

Table 3.16: Summary of Ground Conditions - Area 4

Depth to Top of Strata (mbgl)	Depth to Base of Strata (mbgl)	Thinness (m)	Description
0.0	0.1 to 0.89	0.1 to 0.89	MADE GROUND 4 (MG4): Dark grey and black sandy silty gravel. Made ground evident in all locations in Area 4.
0.1 to 0.7	0.2 to 2.35	0.1 to 2.15	MADE GROUND 1 (MG1): Grey and brown gravelly clay. Evident in all locations in Area 4 apart from GABH05, AAM-REC-11, AAM-REC-15, AAM-REC-08.
0.2 to 1.07	0.4 to 1.9	0.2 to 1.4	MADE GROUND 5 (MG5): Grey and brown silty gravel or gravelly silt. Evident in all locations in Area 4 apart from GABH06, AAM-REC- 03, AAM-REC-09, AAM-REC-15, AAM-REC-04, AAM-REC-06 and AAM-REC-08.
0.4 to 1.15	1.3 to 3.0	0.3 to 2.0	MADE GROUND 6 (MG6): Orange and brown friable clay (reworked). Made ground type only evident in GABH06, AAM-REC-01, GABH07, AAM-REC-03, AAM-REC-05, AAM-REC-04, AAM-REC- 06 and AAM-REC-07.
0.6 to 3.8	1.95 to 4.0	0.5 to 1.9	SILT and CLAY: Grey firm to stiff silty CLAY and SILT with weathered schist gravel. Evident in all locations in Area 4 apart from GABH05, GABH07, AAM-REC-15 and AAM-REC-04.
1.7 to 3.0	2.5 to > 4.0	0.09 to >1.7	SAND and GRAVEL: Light grey brown or orange brown silty SAND and GRAVEL. Evident in all locations in Area 4 apart from GABH06, GABH07, AAM-REC-15 and AAM-REC-06.
0.26 to 3.0	1.18 to > 3.4	0.4 to 0.92	WEATHERED SCHIST: Recovered as greenish grey clayey gravel and cobbles of mica schist. Encountered in GABH06, GABH07, AAM-REC-03, AAM-REC-05 and AAM-REC-15.
1.18 to 4.8	> 4.9 to >9.5	>1.0 to >4.7	SCHIST BEDROCK Encountered in all deep boreholes.

- Summarizing the visual and olfactory evidence of hydrocarbon contamination in 4 areas in Table 3.17.

Table 3.17: Summary of Ground Conditions - Area 4

Area	Borehole Name	Depth (mbgl)	Contamination Evidence
1	WS109	1.9 - 2.8	Weak hydrocarbon odour within natural CLAY.
2	LKRBH03	0.0 - 1.50	Slight diesel odour in MG1.
	LKRBH08	4.0 - 5.0	Moderate diesel odour in natural gravelly CLAY. Visual surface sheen on groundwater.
	LKRBH19	4.0 - 5.0	Slight diesel odour in natural gravelly CLAY.
	TP115	0.8 - 1.3	Slight hydrocarbon odour in MG3.
3	No evidence of contamination.		
4	GABH05	0.7 - 0.85	Hydrocarbon odours in MG5.
	GABH06	0.7 - 2.35	Hydrocarbon odours in MG6.
	AAM-REC-01	0.2 - 0.7	Slight hydrocarbon odours in MG5.
	AAM-REC-03	1.0 - 1.3	Possibly pale oil with slight sheen in MG6.
	AAM-REC-03	3.0 - 3.4	Slight hydrocarbon odour in weathered schist.
	AAM-REC-11	1.07 - 3.09	Strong hydrocarbon odours (fading with depth) in MG5 and silty CLAY.
	AAM-REC-04	3.0	Visible oil in GRAVEL.
	AAM-REC-05	3.0 - 3.4	Visible oil in GRAVEL.

- Summary groundwater levels in each area: Area 1 (Table 3.18); Area 2 (Table 3.19); Area 3 (Table 3.20); Area 4 (Table 3.21).

Table 3.18: Summary of Groundwater Levels - Area 1

BH/TP	Water Strike Depths (mbgl)	Well Response Zone (mbgl)	No. of Monitoring Visits	Monitoring Depths (mbgl)		Sample Taken?	Evidence of Contam?
				DTW	Base		
WS106	2.0 & 4.0	0.5-4.5 (ALL)	1	2.0	4.2	Y	N
WS107	1.5 & 2.7	1.0-3.0 (ALL)	1	1.21	2.7	Y	N
WS108	Dry	Not installed as bedrock at shallow depth.					
WS109	1.7	0.8-2.8 (C)	1	2.25	2.75	Y	N
WS110	3.0	0.8-3.8 (ALL)	1	2.54	3.54	Y	N
TP123	Dry	-	-	-	-	Y	N
TP124	1.9	-	-	-	-	N	N
TP125	Dry	-	-	-	-	N	N

BH/TP	Water Strike Depths (mbgl)	Well Response Zone (mbgl)	No. of Monitoring Visits	Monitoring Depths (mbgl)		Sample Taken?	Evidence of Contam?
				DTW	Base		
TP126	2.4	-	-	-	-	N	N
TP127	Dry	-	-	-	-	N	N
TP128	Dry	-	-	-	-	N	N
TP129	Dry	-	-	-	-	N	N

Response Zones:
MG=Made Ground; SiG = Silty Gravel; ClG = Clayey Gravel; WSc = Weathered Schist; Sc = Schist bedrock; C=Clay.
Notes:
1 Hydrocarbon odours noted during groundwater sampling visit.
2 Hydrocarbon odours noted during drilling and/or trial pitting.

Table 3.19: Summary of Groundwater Levels - Area 2

BH/TP	Water Strike Depths (mbgl)	Well Response Zone (mbgl)	No. of Monitoring Visits	Monitoring Depths (mbgl)		Sample Taken?	Evidence of Contam?
				DTW	**Base**		
WS101	1.1	1.0-1.6 (SiG)	1	1.42	1.6	Y	N
WS102	1.1	0.2-1.2 (MG)	1	0.78	1.12	Y	N
WS103	Dry	Not installed	1	-	-	N	N
WS104	1.8	1.0-2.0 (MG)	1	1.23	1.9	Y	N
WS105	1.2	0.8-1.8 (ClG)	1	0.65	1.94	Y	N
WS119	Dry	Not installed	1	-	-	N	N
RBH01	1.5	5.5-14.5 (Sc)	1	3.02	14.3	Y	N
RBH03	1.5	6.5-15.5 (Sc)	1	3.08	15.5	Y	Y[2]
RBH05	1.5	7.5-16.5 (Sc)	1	2.56	16.2	Y	N
RBH08	4.0	9.0-11.8 (Sc)	1	3.0	11.85	Y	Y[1,2]
RBH09	6.0	4.0-11.0 (Sc)	1	2.07	11.0	Y	N
RBH12	3.0 & 9.0	5.0-14.0 (Sc)	1	1.05	14.0	Y	N
RBH14	6.0	4.0-10.5 (Sc)	1	0.27	10.12	Y	N
RBH17	1.5	5.0-12.0 (Sc)	1	2.15	12.0	Y	N
RBH19	4.0	2.0-5.0 (C)	1	3.0	5.0	Y	Y[1,2]
RBH20	Dry	3.0-5.0 (WSc)	1	1.71	5.0	Y	N
RBH23	5.0	1.5-10.5 (Sc)	1	0.34	10.4	Y	N
RBH24	5.0	4.5-12.5 (Sc)	1	1.55	12.26	Y	N
TP101	0.6	-	-	-	-	N	N
TP102	0.6	-	-	-	-	N	N

BH/TP	Water Strike Depths (mbgl)	Well Response Zone (mbgl)	No. of Monitoring Visits	Monitoring Depths (mbgl)		Sample Taken?	Evidence of Contam?
				DTW	Base		
TP103	1.0	-	-	-	-	N	N
TP104	1.3	-	-	-	-	N	N
TP109	Dry	-	-	-	-	N	N
TP111	0.8	-	-	-	-	N	N
TP112	0.3	-	-	-	-	N	N
TP113	1.6	-	-	-	-	N	N
TP114	0.8	-	-	-	-	N	N
TP115	1.8	-	-	-	-	N	Y[2]
TP117	1.3	-	-	-	-	N	N
TP119	0.3	-	-	-	-	N	N
TP133	Dry	-	-	-	-	N	N

Table 3.20: Summary of Groundwater Levels - Area 3

BH/TP	Water Strike Depths (mbgl)	Well Response Zone (mbgl)	No. of Monitoring Visits	Monitoring Depths (mbgl)		Sample Taken?	Evidence of Contam?
				DTW	Base		
WS111	3.0	1.0-4.0 (C)	1	1.61	3.98	Y	N
WS112	Dry	1.0-3.5 C)	1	0.56	3.39	Y	N
WS113	0.8 & 2.0	1.0-3.0 (C)	1	0.74	3.07	Y	N
WS122	2.0	1.0-3.7 (C)	1	2.06	3.67	Y	N
WS123	Dry	0.4-1.4 (C)	1	0.36	1.4	Y	N
WS124	Dry	Not installed as bedrock at shallow depth					
WS131	1.5	0.6-2.6 (ALL)	1	1.73	2.6	Y	N
WS132	Dry	Not installed as bedrock at shallow depth					
WS133	Dry	Not installed as bedrock at shallow depth					
WS134	Dry	Not installed as bedrock at shallow depth					
WS135	Dry	Not installed as bedrock at shallow depth					
RBH26	3.0	7.0-13.0 (Sc)	1	1.9	13.0	Y	N
RBH27	5.0	5.0-13.0 (Sc)	1	1.73	12.74	Y	N
RBH28	5.0	4.5-13.5 (Sc)	1	1.65	13.49	Y	N
RBH29	6.0	5.0-12.0 (Sc)	1	1.53	12.0	Y	N
RBH30	6.0	4.0-12.0 (Sc)	1	1.62	12.1	Y	N
RBH31	5.0	4.0-12.0 (Sc)	1	1.57	12.0	Y	N

BH/TP	Water Strike Depths (mbgl)	Well Response Zone (mbgl)	No. of Monitoring Visits	Monitoring Depths (mbgl)		Sample Taken?	Evidence of Contam?
				DTW	Base		
RBH32	3.0 & 5.0	3.0-12.0 (Sc)	1	1.5	11.84	Y	N
RBH33	3.0	5.0-12.0 (Sc)	1	1.46	11.82	Y	N
RBH34	3.0	5.0-12.0 (Sc)	1	1.60	11.85	Y	N
TP130	2.3	-	-	-	-	N	N
TP131	2.3	-	-	-	-	N	N
TP135	2.3	-	-	-	-	N	N
TP137	Dry	-	-	-	-	N	N
TP138	1.8	-	-	-	-	N	N
TP139	0.8	-	-	-	-	N	N
TP140	Dry	-	-	-	-	N	N

Table 3.21: Summary of Groundwater Levels - Area 3

BH/TP	Well Response Zone (mbgl)	No. of Monitoring Visits	Monitoring Depths (mbgl)		Sample Taken?	Evidence of Contam?
			DTW	**Base**		
AAM-REC-01	1.0-4.0 (ALL)	1	1.85	3.35	N	N
AAM-REC-03	1.0-3.4 (ALL)	1	1.55	3.0	N	Y[2]
AAM-REC-09	1.0-4.0 (ALL)	1	1.85	3.85	N	N
AAM-REC-11 (shallow)	0.5-4.5 (ALL)	1	1.80	3.65	N	Y[1]
AAM-REC-11 (deep)	6.0-9.5 (Sc)	1	1.85	9.0	N	N
AAM-REC-15 (deep)	2.7-5.5 (Sc)	1	3.5	5.5	N	N
AAM-REC-17	2.0-5.0 (Sc)	1	1.8	4.5	N	N
GABH05	1.5-5.6 (ALL)	1	1.9	5.15	N	N
GABH06	2.4-5.9 (WSc/Sc)	1	1.85	5.8	N	N

- Soil Sampling and Testing summary:
    - A total of 87 soil samples were collected and tested from 35 window sample boreholes, 42 trial pits, and 10 rotary boreholes in Areas 1, 2 and 3.

  - 53 soil samples were analysed for benzo(a)pyrene in Area 2. 2 samples exceeded the commercial Category 4 Screening Level of 77 mg/kg - 210 mg/kg in 1 sample at TP102 0.9-1.5m depth and 99 mg/kg in 1 sample at WS103 0.0-0.5m depth.
  - 53 soil samples were analysed for asbestos in Area 2. Chrysotile asbestos was identified in 3 samples - WS104 at 1.0-1.5m depth, WS116 at 0.6-0.8m depth, and TP142 at 0.05-0.5m depth.
  - No other soil samples exceeded commercial criteria for human health risks.
- Groundwater Sampling and Testing Summary:
  - 28 shallow and 34 deep monitoring wells were installed and sampled across Areas 1-3.
  - 71 groundwater samples were collected and tested from these 62 monitoring wells.
  - Elevated aluminium was detected in 24 out of 44 bedrock groundwater samples, with a maximum concentration of 8,200 μg/L compared to the UKDWS of 200 μg/L.
  - Elevated fluoride was detected in 34 out of 44 bedrock samples (maximum 53 mg/L) and 19 out of 26 superficial samples (maximum 44 mg/L), exceeding the UKDWS of 1.5 mg/L.
  - Elevated copper was detected in 19 out of 44 bedrock samples (maximum 19 μg/L) and 7 out of 26 superficial samples (maximum 19 μg/L), exceeding the EQS for saltwater of 5 μg/L.
  - 1 bedrock groundwater sample exceeded the UKDWS guideline value for total TPH (10 μg/L)
- Surface Water Sampling and Testing Summary:
  - 12 surface water samples were collected from ditches and drains in Areas 1-3.
  - Elevated aluminium (maximum 4,800 μg/L), fluoride (maximum 45 mg/L), and copper (maximum 48 μg/L) were detected, exceeding drinking water standards.
  - No other contaminants exceeded the standards.
- Localised hydrocarbon contamination was identified in 2 adjacent boreholes in a 1.5 m^2 area of Area 2 between 4.0-5.0 mbgl. No impacts found after further delineation.
- The Rectifier Yard has an identified hydrocarbon groundwater plume, including free product, based on previous investigations. Further work is needed to delineate and remediate this.
- Migration to the Irish Sea is considered unlikely. Groundwater and surface water leaving the site is not deemed a significant risk.
- An environmental cover system was recommended in Area 2 soft landscaped areas due to benzo(a)pyrene and asbestos risks. Consultation was also advised for new potable water pipes.
- Revised conceptual site models were developed for the 4 areas: Area 1, Area 2, Area 3 and Area 4.
- For Area 1 (19.4 ha), risks were assessed as low across all 6 pollutant linkages and no specific remediation was required.

- For Area 2 (11 ha), moderate risks were identified for Pollutant Linkage 1 (direct contact) from the 2 benzo(a)pyrene exceedances and 3 asbestos findings in localised areas. Remediation recommendations included an environmental cover layer in soft landscaped areas (approx. 4,500 /m^2 based on proposed layout).
- For Area 3 (5.3 ha), risks were assessed as low across all 6 pollutant linkages and no remediation was required.
- For Area 4 (2.1 ha), a localised hydrocarbon plume is present in groundwater based on previous investigations. Further delineation and remediation were recommended for Pollutant Linkages 4 and 5 based on ALARP principles.
- Migration of contaminants to the Irish Sea was considered unlikely from all areas. Groundwater and surface water leaving the site is not deemed a significant risk.
- Recommendations included consultation with Dwr Cymru Welsh Water regarding new potable water pipes, potential use of barrier pipes (approx. 1,000 m based on proposed layout), and development of an Asbestos Management Plan.
- Remediation works will need to be validated through a Verification Report submitted to regulators. This should include waste documentation, photographic evidence, results of soil chemical testing for approx. 4,500 m3 of imported material etc.
- In summary, whilst some localised risks were identified in Area 2, with appropriate remediation and management the 38-ha site is considered suitable for the proposed renewable energy development. No significant or widespread contamination issues were identified.
- Recommendation summary:
    - Removal of any in-ground structures by grubbing out is recommended prior to development.
    - Hydroponics and aquaculture systems should be self-contained - groundwater should not be abstracted from Areas 2, 3 or 4.
    - Further investigation and remediation of the hydrocarbon plume in Area 4 is recommended based on ALARP principles.
    - A watching brief for potential hydrocarbon contamination is advised in a localised area of Area 2 during development.
    - An environmental cover system is recommended in soft landscaped areas of Area 2, comprising either a physical break layer and imported soil or a geotextile membrane and imported soil to a minimum thickness of 300mm.
    - Consultation with Dwr Cymru Welsh Water is recommended regarding new potable water pipes. Barrier pipes may be required in some localised areas.
    - Precautionary testing of existing potable water supplies in Area 4 is also advised.
- Validation and Verification:
    - Validation testing is recommended for all imported soils used in soft landscaping.

- The environmental cover system depth should be validated by trial pits and photographic evidence.
- A Verification Report detailing remediation and validation works will need to be submitted to regulators. This should include waste documentation, soil chemical testing results, photographic evidence, delineation and remediation details for Area 4, confirmation of potable water supplies, etc.

- Other Considerations:
  - An Asbestos Management Plan is recommended during construction due to the presence of asbestos in made ground in some areas.
  - Unexpected contamination encountered during development should be assessed and may require risk assessment, delineation and remediation.

3.14 2016 05 Anglesey Aluminium REP, Holyhead, Remediation Strategy, LK Consult (LKC)

- The site is 38 hectares total, with Area 1 being 19.4 ha, Area 2 being 11 ha, Area 3 being 5.3 ha, and Area 4 being 2.1 ha.
- 35 window sample boreholes, 42 trial pits, and 34 rotary boreholes were advanced during the site investigation, with depths up to 18m below ground.
- 87 soil samples and 71 groundwater samples were collected and analysed for contaminants.
- Further delineation in Area 4 will involve 17 additional shallow boreholes and 13 deeper rotary boreholes.
- The groundwater remediation system in Area 4 will have capacity to extract around 350 m^3/h.
- Operation of the dual phase vacuum extraction system and chemical oxidation in Area 4 is estimated to take 6-9 months.
- Validation monitoring after remediation will be for a 3-month period.
- The environmental cover layer in landscaped areas of Area 2 will be a minimum of 300mm thick.
- Validation samples will be required for imported soils at rates of 1 per 50 cubic meters for topsoil and 1 per 150 cubic meters for subsoil.
- Key contaminants tested for include metals, PAHs, PCBs, asbestos, cyanide, phenols, hydrocarbons, aluminium, fluoride.

3.15 2016 05 Anglesey Aluminium REP, Holyhead, Escrow Sites Scope of Works, LK Consult (LKC)

The former Anglesey Aluminium Works site was divided into 3 separate Escrow sites based on differences in historical use, contamination sources, locations, containment needs, project management considerations, and legal/financial factors.

- **Escrow 1** - Rectifier Yard and Boundary (Area 4 of planning application boundary): This area contains rectifiers, transformers, a control building and a building owned and operated by National Grid. It is understood that a Transformer fire caused a spill of transformer oil into the ground and free product was present.
- **Escrow 2** – Vehicle Refuelling Area (Garage): This area contained a historical pump island. LKC understand that any tanks in this area have been decommissioned. Diesel impacted water was noted, with free product around the pump island.
- **Escrow 3** – Compressor House: This area contained pumps and a Compressor House. Free product comprising oil was noted in the only borehole drilled in that area.
- Site reconnaissance was conducted to identify and inspect existing monitoring wells installed during previous investigations:
  - A total of 16 existing wells were located at Escrow Site 1. 13 wells could be opened and monitored. Free product (<1cm) was found in 1 well (AAM-REC-11). Hydrocarbon odours were noted in 2 wells (AAM-REC-03 and AAM-REC-06).
  - 1 existing well (GABH-G-01) was located and monitored at Escrow Site 2. Diesel odours were noted.
  - 2 existing wells (APC-02 shallow and deep) were located and monitored at Escrow Site 3. Free product (>50cm) was found in the deep well.
- Summarizing about the proposed site investigations at each Escrow site in Table 3.22.

Table 3.22: Summary of Proposed Site Investigations at Escrow Sites

Escrow Site	Rotary Boreholes	Window Sample Boreholes	Soil Samples	Groundwater Samples
1 - Rectifier Yard	20	30	55	36
2 - Vehicle Refuelling Area	15	29	55	36
3 - Compressor House	15	22	60	37

3.16 2017 06 Escrow Site 1 (Rectifier Yard) Anglesey Aluminium Holyhead, Delineation Investigation and Risk Assessment, LK Consult (LKC)

- The ground conditions in the Rectifier Yard comprised made ground underlain by superficial deposits of sandy clay, silt or silty sand.
- The superficial deposits were underlain by weathered schist, generally recovered as sandy clayey gravel.
- The schist bedrock was highly micaceous. The shallow bedrock was moderately to highly fractured, while the deeper bedrock showed lesser fracturing.
- Shallow boreholes generally refused on the weathered schist or schist bedrock.
- Evidence of hydrocarbon contamination was noted in made ground and superficial deposits in several locations, based on odours and staining.
- No contamination was noted in schist bedrock cores, indicating limited downward migration.
- The contamination was associated with shallower groundwater within the made ground and superficial deposits.
- The depth of made ground, superficial deposits and weathered schist was variable but generally deeper in the central part of the Rectifier Yard.
- The evidence of hydrocarbon contamination in Table 3.23.

Table 3.23: Summary of Evidence of Hydrocarbon Contamination in the Rectifier Yard

Location	Depth (mbgl)	Evidence of Contamination
WS209	0.2-0.80	Moderate hydrocarbon odours in made ground
WS210	0.2-0.8	Weathered diesel odour in made ground
WS303	1.4-2.2	Weak hydrocarbon odour in natural gravel
WS306	0.4-0.6	Weak hydrocarbon odour in natural gravel
WS306A	1.3-4.0	Weak hydrocarbon odour in natural sand and gravel
WS307	1.0-1.9	Weak hydrocarbon odour in natural gravel
WS308	0.3-1.3	Strong hydrocarbon odour in weathered schist
WS313	1.5-1.7	Weak hydrocarbon odour in weathered schist
WS316	1.5-2.3	Weak hydrocarbon odour in natural sand
LKRBH17	2.0-3.0	Moderate hydrocarbon odours in weathered schist

- Two distinct groundwater bodies were encountered - a shallow zone in the made ground and superficial deposits, and a deeper zone in the schist bedrock.
- Groundwater strikes were recorded during drilling. Monitoring wells were installed and sampled to characterise groundwater quality.
- In the superficial deposits, groundwater was encountered between 0.2-2.3m depth. Hydrocarbon contamination was noted in several of these wells.

- In the bedrock, groundwater was encountered between 1.0-9.0m depth. No hydrocarbon contamination was noted in the bedrock wells.
- The pH of the groundwater ranged from 5.86-7.57 in the bedrock and 5.99-7.57 in the superficial deposits.
- Electrical conductivity ranged from 190-940 µS/cm in the bedrock and 180-560 µS/cm in the superficial deposits.
- The results indicate the hydrocarbon groundwater plume is limited to the shallow superficial deposits and has not significantly migrated to the bedrock aquifer.
- Hydrocarbon contamination was delineated in soils within the known impacted area, mostly in made ground and superficial deposits down to 4m depth.
- The highest total TPH concentration was 15,000 mg/kg at location WS303 between 0.4-1.0m depth.
- TPH chromatograms indicated the contamination was diesel/weathered diesel or oil.
- No contamination was found in schist bedrock cores, indicating limited downward migration.
- One location outside the known area (WS209) had TPH of 1,200 mg/kg in made ground, likely from a surface spillage.
- No concentrations in soil exceeded human health criteria for commercial land use.
- Testing across the wider Rectifier Yard found no further contamination hotspots.
- Maximum concentrations of contaminants were below the relevant commercial Generic Assessment Criteria (GAC).
- The investigation enabled successful delineation of the known hydrocarbon contamination at the Rectifier Yard to the shallow soils and groundwater within the superficial deposits, with no significant downward or lateral migration detected. Although risks were assessed to be low, remediation is proposed to achieve betterment of the shallow groundwater quality under the ALARP principle. The recommended strategy involves free product recovery, chemical oxidation of groundwater, potable water supply checks, and validation sampling to demonstrate remediation goals have been met. A Verification Report will confirm the works have been satisfactorily completed as outlined. This will allow the proposed renewable energy development to proceed and satisfy planning requirements. Overall, the investigation provided sufficient characterisation of the contamination to inform a suitable remedial approach focused on improving the shallow groundwater conditions to an acceptable level.

3.17 2017 06 Escrow Site 1 (Rectifier Yard) Anglesey Aluminium Holyhead, Remediation Strategy, LK Consult (LKC)

- Depth profiling found TPH concentrations up to over 1000 mg/kg in the 2.75 to 3.35m depth interval.
- Hydraulic conductivity testing gave values averaging 20.6 m/day in the less silty gravels and 1.83 m/day in the more clayey/silty gravels.
- Transmissivity from a falling head test was 447 m^2/day.
- A step drawdown test achieved a maximum drawdown of 2.06m at the pumping well. Drawdown decreased rapidly with distance, dropping to less than half at 1.3m away.
- Oxidant half-lives were estimated at 4.4-5.8 hours for peroxide and 70-177 days for persulfate.
- The total oxidant requirement (TOR) was calculated as 12.63 g/L for the site based on contaminant and matrix demand.
- Peroxide oxidation achieved near complete destruction of TPH within 24 hours at both 2% and 5% application rates.
- Persulfate oxidation resulted in slower TPH degradation and groundwater rebound effects over time as the oxidant was depleted.
- Trace metal leaching was significant with low pH persulfate but not an issue for near neutral peroxide reactions.
- A 2% peroxide solution was optimal for site conditions, requiring about 160 kg injectant for a 10m treatment radius.
- Post-treatment TPH concentrations were reduced to below detection limits in peroxide reactors, while persulfate reactors had residual TPH present.
- The numerical results strongly support the use of a staged 2% peroxide oxidation approach for effective treatment of the TPH contaminated groundwater at this site. The peroxide provides rapid destruction kinetics and minimal trace metal leaching issues.

3.18 2017 06 Escrow Site 2 (Vehicle Refuelling Area) Anglesey Aluminium Holyhead, Delineation Investigation and Qualitative Risk Assessment, LK Consult (LKC)

- The area is underlain by weathered schist bedrock, encountered at depths of 0.3-3.0 m below ground level (mbgl).
- Shallow boreholes and trial pits hit refusal on the dense weathered schist or competent schist.
- Deepest weathered schist was found in the east by the former fuel pumps, likely due to former below ground tanks.
- The schist is highly micaceous. Shallow schist is moderately fractured, with less fracturing at depth. Fractures are mainly sub-horizontal.
- Thickness of weathered schist is relatively thin compared to other areas of the site, likely due to the shallow bedrock depth.
- Historical removal of weathered schist occurred prior to construction to create a development platform.
- Hydrocarbon odours and staining were noted in 10 of 59 exploratory locations, mainly in the weathered schist layer from 0.2-3 mbgl.
- Only one schist core at 1.3-2 mbgl had diesel odour. Sub-horizontal fractures and low permeability likely limited vertical migration.
- No significant hydrocarbon contamination was found in other schist cores.
- The evidence of hydrocarbon contamination in Table 3.24.

Table 3.24: Summary of Evidence of Hydrocarbon Contamination in the Vehicle Refuelling Area

Boreholes	Depth (mbgl)	Description
WS407	0.3-1.7	Moderate hydrocarbon odour in made ground
WS416	0.21-0.3	Moderate hydrocarbon odour in weathered schist
WS417	0.21-0.4	Moderate hydrocarbon odour in weathered schist
WS419	0.3-0.48	Slight hydrocarbon odour in weathered schist
WS420	0.1-0.58	Slight oily odour in weathered schist
RBH409	0.2-1.0	Moderate diesel odour in weathered schist
RBH414a	0.2-0.4	Faint diesel odour in weathered schist
RBH417	0.2-1.0	Moderate diesel odour in weathered schist
RBH420 & RBH420a	0.2-2.0	Moderate diesel odour in weathered schist and underlying bedrock
RBH422a	1.0-3.0	Moderate diesel odour in weathered schist

- The delineation investigation analysed 51 soil samples from the Vehicle Refuelling Area. 22 of these samples had detectable concentrations of total petroleum hydrocarbons (TPH). The maximum TPH concentration found was 12,000 mg/kg in sample WS407, which was identified as diesel range organics. Generally, soil samples collected from below 1m did not have detectable

TPH, indicating that significant vertical migration of hydrocarbons has not occurred. 25 soil samples were tested for volatile organic compounds (VOCs), but only a single sample had concentrations above detection limits. Additionally, only one schist bedrock core, from 1.3-2m depth, exhibited a diesel odour but no staining or free product.

- Of the 11 superficial groundwater samples collected and analysed, 3 had detectable concentrations of TPH. The maximum TPH concentration was 120 mg/L, recorded in monitoring well RBH423a. TPH chromatograms indicated the contamination was weathered diesel. No free hydrocarbon product was identified during drilling activities or subsequent groundwater monitoring and sampling. With regards to VOCs, only 2 locations had detectable concentrations, mainly benzene derivatives, with higher levels present at RBH423a.
- Analysis of 15 groundwater samples from wells installed into the underlying schist bedrock found TPH concentrations above detection limits in 2 locations. RBH423 had the maximum TPH concentration at 2 mg/L. The presence of limited localised hydrocarbon contamination in the bedrock groundwater at RBH423 is likely due to more highly fractured and weathered schist providing a pathway. However, nearby bedrock groundwater samples did not detect TPH, indicating restricted lateral migration. No free product was identified during groundwater monitoring.
- Depth Profiling
  - Frozen soil cores collected to determine vertical distribution of contamination.
  - Maximum TPH concentration of 7,000+ mg/kg found at 0.55m depth in gravel layers.
  - Indicates contamination concentrated in shallow subsurface.
- Hydraulic Testing
  - Limited groundwater availability affected testing.
  - Falling head test abandoned as no observed hydraulic head decline.
  - Highly permeable aquifer but limited flow in shallow groundwater.
- Treatability Trials
  - Lab tests assessed chemical oxidant performance.
  - Peroxide most effective for TPH destruction and induced rapid NAPL dissolution.
  - Persulfate slower kinetics and possible rebound of TPH concentrations.
  - Peroxide provides circumneutral pH, unlike persulfate acidification.
- Depth profiling and lab testing found shallow TPH contamination that can be effectively treated with peroxide ISCO, which showed strong oxidation performance without secondary impacts. Peroxide is recommended for sequential injections to achieve 2% concentration in groundwater.
- The recommended treatment method is in-situ chemical oxidation (ISCO) using peroxide based on pilot study results. No free product recovery is needed as no free product was identified. Peroxide will be injected via new and existing wells to oxidize contaminants. Multiple oxidant applications are expected for required results. The treatment area was defined in the pilot study, with concentric

injections from perimeter inward to control migration. Approximately 30 new injection wells are needed across the treatment area. Oxidant will be stored and mixed onsite with required services.

- ISCO application is estimated at 1-2 months followed by 12 months of validation monitoring and monthly groundwater sampling to show contaminant reduction and no free product. Monthly updates on oxidation progress will be provided to stakeholders.
- An environmental permit is required for the remediation works. The treatment plant will occupy approximately 10x20m, with location dependent on-site operations. Treatment areas will be fully bunded for equipment access. No waste streams will be generated apart from basic consumables.

3.19 2017 06 Escrow Site 2 (Vehicle Refuelling Area) Anglesey Aluminium Holyhead, Remediation Strategy LK Consult (LKC)

- Depth profiling indicated that the peak hydrocarbon contamination was located at approximately 0.5 m below ground in the gravel fractions.
- Limited shallow groundwater volumes significantly affected undertaking hydraulic testing. A falling head test had to be abandoned due to no observed decline in hydraulic head.
- Chemical oxidation trials showed that a peroxide-based oxidant was highly effective at treating dissolved phase hydrocarbon impacts.
- The high permeability of the subsurface should allow efficient delivery and contact between the oxidant and contaminants.
- Multiple oxidant applications would likely be needed, starting from the edge of the plume and moving inwards to prevent uncontrolled displacement of contaminants.
- Neither dual vacuum phase extraction nor free product recovery were deemed necessary.
- The report recommended in-situ chemical oxidation using peroxide as the optimum treatment method for this site. It has rapid kinetics and can enhance dissolution and destruction of hydrocarbons.
- Sequential applications of 2% peroxide are recommended, targeting the most impacted areas based on monitoring residual TPH.
- Peroxide provides circumneutral pH conditions, limiting metal mobilisation that can occur with other oxidants like persulfate.
- Field injection trials prior to full scale work are recommended to confirm subsurface oxidant reactivity matches the lab trials.

3.20 2017 06 Escrow Site 3 (Compressor House) Anglesey Aluminium Holyhead, Delineation Investigation and Qualitative Risk Assessment, LK Consult (LKC)

- The site geology consists of schist bedrock overlain by variable thicknesses of unconsolidated superficial deposits.
- The schist is a medium grained metamorphic rock. It forms an impermeable base to the aquifer systems.
- The superficial deposits comprise horizons of gravel, sand and silt. They form a relatively high permeability aquifer system.
- In the Rectifier area, superficial deposits are up to 0.8m thick. In the Compressor area, they reach thicknesses of 2.3 - 5.8m.
- The schist bedrock was encountered at depths of 5 – 7m in boreholes. It showed moderate to high fracture density in the upper weathered zone, decreasing with depth.
- Groundwater flow is primarily through fractures in the schist. Weathered schist likely has higher conductivity than deeper unweather rock.
- The cross section shows thinner superficial deposits under buildings in the west (<1m) and thicker deposits in undeveloped land in the east (>1m). Weathered schist depth was greater in the west (circa 7mbgl).
- The evidence of hydrocarbon contamination in Table 3.25.

Table 3.25: Summary of Evidence of Hydrocarbon Contamination in the Compressor House

Boreholes	Depth (mbgl)	Evidence of Contamination
WS501	0.6-1.8	Strong diesel odour in made ground and natural sand, becoming slight from 1.8 mbgl
WS518	1.6-3.8	Moderate diesel odour in natural sand
RBH501	3.5-5.5	Diesel odour in weathered schist
RBH502	0.0-4.0	Slight weathered diesel odour in made ground and weathered schist
RBH516	2.5-5.5	Diesel odour in weathered schist
RBH517A	4.5-5.5	Diesel odour in weathered schist
RBH523	3.5-7.0	Diesel odour in weathered schist and schist bedrock
RBH524	4.0-6.0	Diesel odour in weathered schist
RBH527A	3.5-5.5	Diesel odour in weathered schist
RBH528A	4.0-5.5	Slight diesel odour in weathered schist
RBH529A	3.5-6.0	Slight diesel odours becoming moderate by 4.5 mbgl in weathered schist
RBH530A	4.0-6.5	Moderate diesel odour in weathered schist
RBH531A	3.5-4.5	Moderate diesel odour in weathered schist
RBH535A	3.5-4.5	Moderate diesel odour in weathered schist

- 4 soil samples showed TPH contamination relating to lubricating oil, with the highest concentration at 4,100 mg/kg in WS507.

- Lubricating oil contamination was localised around APC02, reflecting immobility.
- 12 samples had TPH relating to diesel, with the highest at 4,400 mg/kg in WS501.
- Diesel contamination extended below 2 mbgl, to 4.5 mbgl in RBH517A in the weathered schist.
- The vertical distribution indicates association with shallow groundwater.
- Only 2 samples had TPH from lubricating oil, at APC02 and new well WS508 nearby. Concentrations were low (0.6-2.8 mg/l).
- Free product (lubricating oil) was present at APC02. The source seems limited but not exhausted.
- 13 samples had TPH relating to diesel, with a maximum of 710 mg/l in RBH527A.
- Diesel plume was delineated covering the Compressor House area and east across North Road.
- The source is likely former mobile diesel compressors east of the Compressor House.
- Local groundwater flow seems influenced by sub-surface structures.
- 7 of 16 samples had low TPH levels from 0.1 to 4.6 mg/l, the bedrock not being significantly impacted.
- Lateral migration in bedrock is limited by less fracturing.
- Depth profiling showed peak hydrocarbon contamination from 3.0-3.85 mbgl, with concentrations up to 2,500 mg/kg associated with lower permeability horizons.
- More permeable gravels had TPH levels around 800 mg/kg, indicating they transmit groundwater but do not store significant contamination.
- Hydraulic testing at 3 locations showed variable conductivity ranging from 0.1 to 291 m/day, likely due to less permeable bands or sub-surface structures identified.
- High permeability of 291 m/day near APC02 indicates pumped groundwater extraction could effectively recover free product and dissolved hydrocarbons.
- Chemical oxidation trials demonstrated a peroxide-based oxidant achieved up to 100% destruction of TPH contaminants via redox reactions and free radical generation.
- Persulfate systems showed only 50-60% contaminant destruction at both high (2x TOR) and low (TOR) tested concentrations.
- Persulfate also caused groundwater acidification to pH 1.4 and mobilised up to 4,200 µg/L of dissolved metals.
- The aquifer's high permeability of up to 291 m/day allows efficient delivery of oxidants for ISCO treatment.
- The addition of specific data from the pilot testing provides quantified evidence that free product recovery combined with chemical oxidation using peroxide would be an effective remediation approach for rapidly treating contaminants at this site.

3.21 2017 08 Escrow Site 3 (Compressor House) Anglesey Aluminium Holyhead, Remediation Strategy, LK Consult (LKC)

- Previous investigations identified two sources of groundwater contamination - a localised plume of lubricating oil around monitoring well APC02, and a larger plume of diesel contamination extending east of the Compressor House.
- Additional investigation is proposed below an oil/water separator near APC02 to confirm it as the source of the lubricating oil.
- Proposed remediation methods:
- Dual Phase Vacuum Extraction (DPVE) to recover free product lubricating oil around APC02. This will operate for up to 6 months.
- In-Situ Chemical Oxidation (ISCO) using oxidants to treat dissolved phase diesel contamination across a wider area. Multiple oxidant applications anticipated over 1-2 months.
- Validation will involve 12 months of groundwater monitoring to demonstrate betterment and reduction of contaminant concentrations.
- A Verification Report will summarise the remediation works and results of validation monitoring to demonstrate to stakeholders that the remediation strategy has been implemented and betterment achieved.
- The overall objectives are to remove measurable free product and achieve significant reduction in dissolved contaminant concentrations in groundwater to meet the ALARP (As Low As Reasonably Practicable) principle.

3.22 2021 09 Former Anglesey Aluminium Site, Holyhead, Historical Review of the Site Investigations Undertaken in the Area of the Proposed Plastics Depolymerisation Unit (PDU), LK Consult (LKC)

- The PDU area refers to the proposed plastics depolymerisation unit building and associated infrastructure in the southern part of the former Anglesey Aluminium site. The PDU building is an existing structure that will convert plastic to pellets, syn-gas, syn-oil and syn-char. New infrastructure will surround the PDU building.
- For the historical site review, the PDU area was divided into:
  - PDU Building.
  - Area A (Southeast) - former vehicle yard.
  - Area B (Northwest) - former storage.
  - Area C (North) - former product storage.
- The review focuses on contamination in these four zones to establish a baseline before development.
- Baseline Contamination Levels in Table 3.26.

Table 3.26: Summary of Baseline Contamination Levels

Area	Material	Key Contaminants	Maximum Levels Detected
PDU Area	Soil	TPH	4.1 mg/kg
		PAHs	<0.01 mg/kg
	Groundwater	TPH	<10 µg/L
		VOCs	Not detected
Area A	Soil	TPH	8,140 mg/kg (pre-remediation)
		PAHs	3.5 mg/kg (pre-remediation)
	Groundwater	TPH	21,000 µg/L (pre-remediation)
		VOCs	600 µg/L (pre-remediation)
Area B	Soil	TPH	<10-12,000 mg/kg
	Groundwater	TPH	<10 µg/L
Area C	Soil	TPH	160 mg/kg
		PAHs	<0.01 mg/kg
	Groundwater	TPH	<10 µg/L
		VOCs	<0.01 µg/L

- Based on the comprehensive review of available historical data and site investigations, residual contamination in the vicinity of the proposed PDU area appears low.

- The main area of more significant contamination was in Area A (the former vehicle yard) where diesel impacts to soil and groundwater were identified. However, this area underwent remediation by chemical oxidation in 2017 to address the hydrocarbon contamination.
- Within the PDU building footprint itself, previous assessments encountered some weathered bedrock and groundwater, but limited sampling did not reveal substantial contamination.
- To the northwest (Area B) and north (Area C), assessments found some made ground, weathered rock and shallow groundwater. But again, contamination levels from sampling were not significant.
- The primary potential risks are from vapour inhalation and groundwater migration. However, given the industrial history and uses, the remediation undertaken, and the hardstanding coverage proposed, these risks are considered minimal.
- Therefore, based on the available historical data, it is concluded that no further intrusive investigation work is recommended in relation to the planning conditions for the PDU area. The key potential contaminant linkages have already been adequately characterised to inform a robust conceptual site model.
- In summary, the conclusion is that the PDU area has low residual contamination based on the historical site assessments reviewed and poses minimal risk for the proposed development.

4. Tier 3 summaries

The Tier 3 Summary chapter briefly summarises 15 peripheral reports dated 2009-2021 that provide relevant contextual information but do not contain new borehole investigation or geotechnical data for the site. The Tier 3 reports offer useful wider framing through overviews of matters like ecology, and mitigation recommendations without introducing substantial new subsurface technical information. While not directly advancing the site characterisation, the Tier 3 summaries concisely cover salient points from these reports to contribute to the overall understanding of the former Anglesey Aluminium site.

4.1 2009 08 Section 14, Summary of Mitigation and Monitoring

- The proposed renewable energy plant will have a generating capacity of 90 megawatts.
- Air quality mitigation includes using selective non-catalytic reduction (SNCR) to ensure NOx emissions meet regulatory limits, and bag filters to keep particulate emissions below 20mg/Nm3.
- A stack height of 60m is proposed along with adequate flue gas temperature and velocity for dispersion.
- Around 1.1 hectares of broadleaved woodland and 0.2 hectares of marshy grassland will be lost due to the development.
- Compensatory planting of native woodland species over 1.5 hectares is proposed to offset the woodland loss. Marshy grassland will be recreated on at least 0.2 hectares.
- An otter survey will be conducted prior to construction to avoid impacts. Protected wildlife corridors will be maintained.
- Lighting will be designed to minimize disturbance to bats and birds. Nest boxes will be installed for birds.
- Archaeological surveys found no buried archaeology or direct impacts on cultural heritage at the site.
- The development is expected to create 500-600 jobs during construction and 40 permanent jobs when operational.

4.2 2014 03 Anglesey Aluminium Metal Ltd, Site Wide Groundwater Monitoring September 2013-February 2014, Golder Associates Ltd

- Groundwater elevation ranged from 1.40 m AOD to 8.19 m AOD.
- The minimum elevation was recorded in boreholes near the coast in the northwest of the site, while the maximum elevation was recorded further inland to the east.
- The groundwater contours suggest a general flow direction to the northwest, towards the coast. This is consistent with previous monitoring.
- There were no significant changes in groundwater flow direction during the monitoring period.
- Some local disturbances to contours were noted in the Pitch Tanks area, likely due to the higher density of monitoring wells compared to other parts of the site.
- Groundwater elevations were generally consistent between monitoring rounds, with no major fluctuations observed.
- In summary, the groundwater elevations indicate a consistent north-westerly flow direction, with no significant changes over the 6-month monitoring period. The elevations ranged from around sea level to over 8 m AOD, with local variations related to site conditions and well density.

4.3 2014 07 Anglesey Aluminium Metal Site, Ecological Surveys Position Statement, Ramboll UK Ltd

- The statement provides a good overview of the previous ecological surveys conducted in 2009 and the recent preliminary walkover in 2014. It summarises the key protected species findings from 2009, including common lizards, badgers, water voles, newts, bats, and birds.
- The 2014 walkover checked the hardstanding area, reedbed, woodland, and water features. No major new findings were made, though some limitations were noted due to sub-optimal survey timing for species like newts.
- Based on the review, the statement recommends conducting updated detailed surveys for great crested newts, badgers, water voles, otters, reptiles, bats, and birds prior to construction.
- Consultation with Natural Resources Wales and the Council's Biodiversity Officer on the survey scope is also advised.

4.4 2014 10 Reptile Survey, Anglesey Aluminium, Kestrel Environmental Services

- The survey was conducted by Kestrel Environmental Services on behalf of Ramboll UK Ltd in October 2014 at the Anglesey Aluminium site in Holyhead, Anglesey.
- The survey methodology involved placing 30 felt refuges ("tins") and checking them over 6 planned surveys. Only 4 surveys were conducted due to weather.
- No reptiles were observed during the surveys.
- One common toad (Bufo bufo) was captured - 1 male and 1 female adult, and 1 juvenile.
- Weather conditions during surveys were documented, including temperature, wind speed and direction, and general conditions.
- The report concludes that insufficient survey effort occurred to confidently state absence of reptiles on site, due to the late season start and weather issues.
- It recommends further survey work in spring 2015 with additional refuges.
- It also recommends consideration of mitigation for common toads during any site clearance, as they are a priority BAP species.

4.5 2014 12 Anglesey Aluminium Metal Site, Ecological Constraints Note, Ramboll UK Ltd

- Additional ecological surveys are needed at the Anglesey Aluminium Metal site before construction can begin, including for reptiles, great crested newts, water voles, and breeding birds. Surveys should be timed appropriately for each species.
- There is a need to consult with Natural Resources Wales and the Isle of Anglesey Council's Biodiversity Officer on survey scope and requirements, especially for the proposed Eco Park. Recommended surveys for the Eco Park include bats and birds.
- Vegetation clearance should be timed to avoid impacts to protected species like great crested newts and breeding birds. Measures may be needed to obtain licenses for habitat management if great crested newts are present.
- The development of the Renewable Energy Plant and Eco Park need to be coordinated, especially regarding drainage, ecological enhancement, etc. Early engagement with planners and stakeholders is recommended.
- An Ecology and Habitat Mitigation Strategy will need to be prepared to discharge planning conditions, outlining surveys, mitigation, and enhancement. A partial discharge may be needed for early works.
- Key ecological constraints on site design include maintaining buffer zones from badger setts, woodlands, and potential great crested newt habitat areas.

4.6 2015 06 Anglesey Aluminium Site, Preliminary Ecological Walkover, Ramboll UK Ltd

- A preliminary ecological walkover was conducted at the proposed biomass power station site to identify needed surveys to update the baseline and support planning conditions.
- No great crested newts were found, but additional surveys are recommended for Water Feature 1 and the ornamental pond during March-June.
- Evidence of badger activity was found. Detailed surveys are recommended to map setts and activity.
- No definitive signs of water vole or otter were found, but additional surveys are recommended in the reedbed area.
- Potential reptile habitat was identified. Presence/absence surveys are recommended.
- No notable bird species were observed, but need for breeding bird surveys should be discussed with NRW.
- Bat activity surveys may be needed for trees to be removed.
- Meeting with NRW and Isle of Anglesey Council Biodiversity Officer is recommended to confirm survey scope and needs.
- After survey scope confirmation, updated schedule and mitigation plans can be developed as needed.
- Continued engagement with regulators throughout the survey and planning process will be important.

4.7 2015 07 Anglesey Aluminium Renewable Energy Plant Great Crested Newt EDNA Testing Report, Ramboll UK Ltd

- The report was prepared by Ramboll UK for Orthios Limited in July 2015. It presents the results of environmental DNA (eDNA) testing for great crested newts at the Anglesey Aluminium Renewable Energy Plant site in Holyhead, North Wales.
- Habitat Suitability Index (HSI) assessments were conducted on 5 water features at the site. Water Feature 1 was rated as 'average' suitability for breeding great crested newts, while the other features were 'poor'.
- eDNA testing was carried out on Water Feature 1 in June 2015. 20 water samples were taken and analysed by the ADAS laboratory.
- The eDNA test results were negative, showing no evidence of great crested newt DNA in Water Feature 1.
- Based on the HSI and eDNA findings, it is concluded that great crested newts are absent from the site. No specific mitigation measures are required for this species.
- The report recommends discussing the findings with Natural Resources Wales and Isle of Anglesey Council and considering ecological design inputs to retain/improve water features at the site.
- In summary, the eDNA testing showed no great crested newts present in the key water feature surveyed. This allows the project to proceed without impacts to this protected species. Recommendations are made to consult with regulators and improve biodiversity value of water bodies.

4.8 2015 08 Anglesey Aluminium Renewable Energy Plant Reptile Survey Report, Ramboll UK Ltd

- The survey was conducted by Kestrel Environmental Services on behalf of Ramboll UK Ltd in two phases: October 2014 and April-June 2015.
- The survey followed standard guidelines, using artificial refugia ("tins") to check for the presence of reptiles.
- Only one reptile species, the common lizard, was recorded during the surveys. No other reptile species were found.
- Common toads were also recorded incidentally during the reptile surveys.
- The common lizard population was classified as "low" based on the peak number of adults observed.
- Suitable reptile habitat was identified north of the proposed construction footprint, within the consented site boundary, to serve as a receptor site.
- A mitigation strategy is recommended, likely consisting of targeted trapping and translocation of common lizards and toads prior to construction.
- The mitigation aims to remove common lizards and toads from the construction footprint and prevent impacts during vegetation clearance and habitat removal.
- Habitat manipulation will also be undertaken to discourage reptiles from the construction area after translocation.

4.9 2017 03 Anglesey, Laboratory Bench Scale Chemical Oxidation Study, CE Geochem Ltd

- Peroxide systems showed up to 99% destruction of TPH across all fractions within 24-48 hrs. For example, 5% peroxide reduced C10-C12 TPH concentrations in the Rectifier reactors from 83 µg/L to 0.1 µg/L in 24 hrs.
- Persulfate alone resulted in slower TPH destruction, with only 50-60% reductions in TPH over 672 hrs. Persulfate also showed TPH rebound up to 190 µg/L for C10-C12 as oxidant was depleted.
- Peroxide activated persulfate showed 75-85% TPH destruction over 672 hrs, outperforming persulfate alone but not as effective as peroxide systems.
- Peroxide reaction with the GC aquifer matrix produced an increase in dissolved C10-C12 TPH from 4.4 to 250 µg/L after 6 hrs, indicating enhanced NAPL dissolution. Concentrations then declined to 3 µg/L by 96 hrs as peroxide degraded TPH.
- Acidic pH as low as 1.4 was observed for peroxide-activated persulfate, resulting in up to 4100 µg/L copper and 910,000 µg/L iron being mobilised from the aquifer matrix.
- Peroxide half-lives ranged from 4.4 to 5.8 hrs, compared to 1693-4252 hrs for persulfate.
- 2D simulations predicted peroxide would achieve a 45m radius of influence in the Compressor area after 6 hrs, with 2% concentrations remaining.
- Recommendations:
    - 2% peroxide is optimal for TPH destruction, NAPL dissolution, rapid kinetics within residence times, and avoiding metal mobilisation.
    - Stepwise injection approach proposed with 2% peroxide to optimise delivery to high TPH areas. Residual TPH to be monitored to guide injections.
    - Field injections recommended before full deployment to validate 2% subsurface peroxide persistence for 6-24 hrs seen in laboratory study.

4.10 2017 04 Remediation Pilot Study, Escrow Areas Anglesey Aluminium Penrhos Works Holyhead, Geo2 Remediation Limited

- Depth profiling showed peak contamination levels of over 1,000 mg/kg in the Rectifier Yard, over 7,000 mg/kg in the Garage, and over 2,500 mg/kg in the Compressor House.
- Hydraulic testing showed:
  - Average hydraulic conductivity of 10.18 m/day in the Rectifier Yard, with higher values of 20.63 m/day in less silty gravels and lower values around 1.83 m/day in more silty areas.
  - Transmissivity of 447.27 m^2/day in the Rectifier Yard.
  - Hydraulic conductivity averaging 179.99 m/day in the Compressor House, with values of 25.98 m/day and 291.33 m^2/day in different areas.
  - Transmissivity of 133.34 m^2/day in the Compressor House.
- Chemical oxidation testing found 2% peroxide oxidant achieved NAPL dissolution and prevented rebound within 96 hours.
- Recommendations include 2% peroxide oxidant application for all areas, with stepped application in key zones.
- Providing these quantitative results from the depth profiling, hydraulic testing, and chemical oxidation trials helps give more specific evidence to support the summarised findings and recommendations.

4.11 2018 02 Escrow Site 3 (Compressor House) Former Anglesey Aluminium Holyhead, Groundwater Risk Assessment Consult (LKC)

- The report is a groundwater risk assessment for a Compressor House area at a former aluminium works site in Holyhead, Anglesey. It was conducted by LK Consult Ltd for Orthios Eco Parks Ltd.
- The site contains a large diesel plume mainly in the superficial deposits, with minimal impact to the bedrock. Traces of diesel free product were found in one location.
- The report aims to model hydrocarbon concentrations in the bedrock aquifer to determine if remediation is needed at the site.
- Concentrations were compared to generic assessment criteria like WHO guidelines. Some aliphatic and aromatic hydrocarbon fractions exceeded the conservative criteria.
- A detailed quantitative risk assessment was done using the EA Remedial Targets methodology to generate site-specific target levels (SSTLs).
- Using conservative assumptions, the modelling indicates hydrocarbon concentrations at the site should not impact groundwater compliance points 50m or 400m downgradient.
- The modelled attenuation indicates contaminant concentrations at 50m distance should be below the generated SSTLs.
- At 400m distance to the Irish Sea, the model predicts all hydrocarbon fractions will meet SSTLs within 28m distance from source.
- The conclusion is remediation is not required, but enhancements will be done under the ALARP principle to improve groundwater and soil conditions.
- Proposed remediation methods include dual phase vapour extraction and chemical oxidation of diesel and oil contaminants based on successful pilot tests.

4.12 2020 07 Orthios, Former Anglesey Aluminium, Pernhos-Borehole Decommissioning Report, LK Consult (LKC)

- LK Consult Ltd was commissioned by Orthios Eco Parks Ltd to decommission boreholes at the former Anglesey Aluminium site in Penrhos, Holyhead.
- A total of 66 boreholes were proposed for decommissioning in the "planning boundary" area and 21 more across the wider site.
- Borehole decommissioning took place between May 25-29, 2020.
- The work followed Environment Agency guidance on decommissioning redundant boreholes. The process involved removing well heads, backfilling with gravel and bentonite, capping with concrete, etc.
- 65 out of 66 boreholes were decommissioned in the planning boundary, with 1 not located (LKRBH29). Total meterage decommissioned was 503m.
- 17 out of 21 boreholes were decommissioned in the operational site area. 4 were not located. An additional unknown borehole was found and decommissioned. Total meterage was 120m.
- Further decommissioning work is expected in March/April 2021 after regulatory approval.

4.13 2021 03 Orthios, Former Anglesey Aluminium, Pernhos-Post Remediation Monitoring Report (Escrow Site 1-Rectifier Yard Area), LK Consult (LKC)

- The post-remediation groundwater monitoring was conducted over a 12-month period at the former Anglesey Aluminium site in Holyhead, UK. The site was divided into 3 escrow areas that were investigated and remediated separately:
- Escrow Area 1 - Rectifier Yard
  - 7 monitoring wells sampled quarterly.
  - Remediation completed in Feb 2020 using excavation and chemical oxidation.
  - TPH concentrations reduced by 100% by end of 12-month monitoring.
  - No evidence of free product or plume migration.
- TPH concentrations in groundwater declined significantly after remediation, with all samples <10 µg/L TPH by the end of 12 months.
- Remediation achieved removal of free product and betterment of groundwater quality.
- No evidence of plume migration or significant rebound during monitoring period.
- Results indicate remediation aims were successfully met in Escrow Area 1.

The reports demonstrate the effectiveness of the remediation works and post-remediation monitoring in reducing petroleum hydrocarbon concentrations in groundwater to below target levels within 12 months. No significant risks from hydrocarbons remaining in groundwater were identified.

4.14 2021 03 Orthios, Former Anglesey Aluminium, Pernhos -Post Remediation Monitoring Report (Escrow Site 2-Garage Area), LK Consult (LKC)

- This is a groundwater monitoring report by LK Consult for the former Anglesey Aluminium site in Holyhead, UK. It covers Escrow Site 2, which is the former vehicle refuelling area (Garage).
- Groundwater was monitored over 12 months after remediation of diesel contamination in this area. Samples were taken from 6 monitoring wells installed in the superficial deposits and bedrock.
- Prior to remediation, concentrations of total petroleum hydrocarbons (TPH) up to 120,000 μg/L were detected, indicating diesel fuel contamination.
- Remediation involved removing underground fuel storage tanks and impacted soils. Oxygen release compound was also applied to enhance biodegradation of TPH.
- Post-remediation, TPH concentrations in groundwater decreased significantly in both the superficial deposits and bedrock. By the end of the 12-month monitoring period, TPH was reduced by 100% to below laboratory detection limits in all wells.
- The monitoring results demonstrate that the remedial objectives were achieved. Diesel impact has been eliminated and groundwater quality improved to below applicable standards.
- There is no evidence of remaining fuel contamination or that contamination has migrated beyond the original plume area. Natural attenuation processes have further reduced any residual dissolved hydrocarbon concentrations.
- In conclusion, the remediation was successful in removing the source area, eliminating fuel impacts to groundwater, and achieving site closure objectives agreed with regulators. Monitored natural attenuation has proven effective at addressing any remaining low-level dissolved phase impacts.

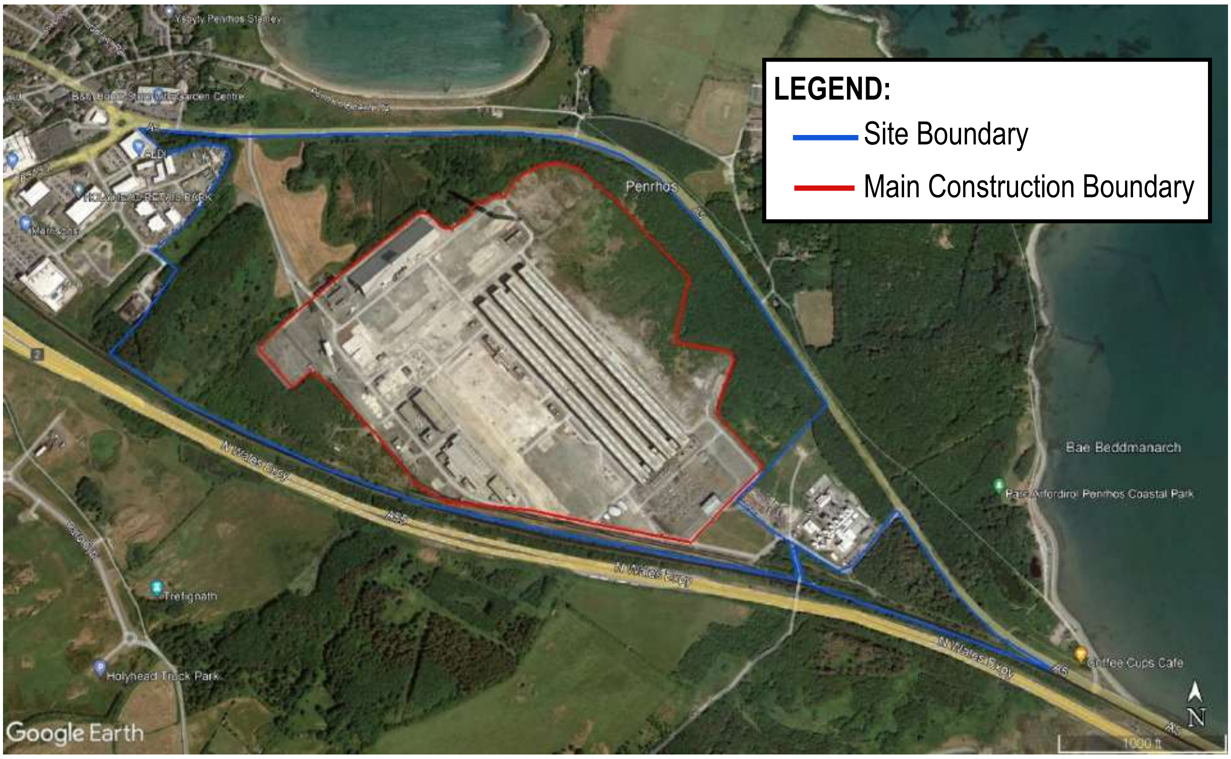
4.15 2021 03 Orthios, Former Anglesey Aluminium, Pernhos-Post Remediation Monitoring Report (Escrow Site 3-Compressor House Area), LK Consult (LKC)

- This report details the post-remediation groundwater monitoring results for Escrow Site 3 (Compressor House area) at the former Anglesey Aluminium site in Holyhead, UK.
- Prior investigations identified diesel and oil impacts to groundwater in the superficial and bedrock deposits. Remediation was completed in February 2020.
- 12 monthly post-remediation groundwater monitoring events were conducted to confirm remediation success and meet agreed remedial aims of free product removal and groundwater quality improvement.
- 8 monitoring wells (4 superficial, 4 bedrock) were sampled for TPH and BTEX.
- Pre-remediation, TPH in superficial deposits ranged from 1,900 ug/L to 710,000 ug/L. In bedrock, 260,000 ug/L to 710,000 ug/L.
- At the end of remediation, TPH reduced to 303-806 ug/L in superficial deposits and 189-1,159 ug/L in bedrock.
- In the first post-remediation visit, TPH was 140-360 ug/L in superficial deposits and non-detect to 180 ug/L in bedrock.
- By the second visit one month later, TPH was non-detect in all wells.
- Over the next 4 visits, all wells remained non-detect for TPH except two detections of 86-190 ug/L in superficial deposits.
- From visit 7 onwards (Sep 2020), TPH has remained non-detect in all wells for the duration of the 12-month monitoring period.
- No evidence of significant rebound or off-site contaminant migration.
- Post-remediation monitoring indicates remedial aims have been met - 100% reduction of TPH in groundwater and no ongoing free product.
- In summary, the post-remediation monitoring demonstrates effective remediation of the diesel and oil impacts previously identified at the Compressor House area. The agreed remedial objectives have been achieved.

5. Gap analysis in coverage

The gap analysis chapter examines portions of the 38-ha former Anglesey Aluminium site lacking adequate subsurface data based on historical investigations. The site is divided into the well-characterised main construction area and the sparsely assessed outside areas. While the main construction area containing key industrial facilities has been extensively investigated with multiple boreholes providing crucial geotechnical data, the outside areas like surrounding fields have sparse borehole coverage resulting in significant data gaps. Additional targeted site assessments and geotechnical testing are required in these poorly characterised outside areas to properly delineate conditions and support informed management decisions for potential redevelopment.

Figure 5.1: The picture shows approximate boundary of main construction and Former Anglesey Aluminium Plant.



The former Anglesey Aluminium site covers approximately 38 hectares. The comprehensive historical site investigations, spanning from the 1990s to recent years, provide extensive subsurface data on ground conditions, contamination, geotechnical properties, and hydrogeology across the majority of the site area.

However, there are some localised gaps where subsurface information is lacking based on the borehole and trial pit locations documented in the available investigation reports. These gaps represent opportunities for additional characterisation to supplement the robust existing dataset.

As shown in Figure 5.1, the former site can be divided into two primary areas – the main construction area red line) and the outside areas (blue line).

The main construction area includes the key industrial facilities like potlines, rectifier yard, pitch tanks, etc. This zone has been extensively investigated with multiple boreholes documented in the Tier 1 reports. The locations of these Tier 1 boreholes in the main construction area are shown in the drawings in Appendix A. These Tier 1 boreholes contained geotechnical testing like SPTs, vane shear, permeability, CBR, consolidation, classification, strength testing, etc. There is good geotechnical coverage across the main construction area based on the Tier 1 summary section.

The outside areas include the zones outside the core industrial facilities, such as the reclamation yard, surrounding fields, wasteland, wooded areas, and the conveyor route. These areas have sparse subsurface investigation coverage in many locations based on the available historical data. The locations of boreholes documented in the outside areas are shown in the drawings in Appendix B.

Specifically, the northern outside area lacks borehole data aside from along the conveyor belt route. The eastern zone also has major gaps, with insufficient geotechnical characterisation. The southeast corner has no boreholes documented. The western outside area has limited data, with a cluster of boreholes near the cooling towers but gaps elsewhere.

In summary, while the main construction area has robust subsurface datasets, significant gaps exist in the outside work areas that warrant additional investigation to properly characterise ground conditions, geotechnical properties, contamination, hydrogeology, and other parameters to support informed site management decisions and future redevelopment.

6. Overall summaries

This chapter consolidates the key findings from the extensive historical site investigations to establish a high-level understanding of present subsurface conditions across the former Anglesey Aluminium site. The overall summary synthesizes the critical results and interpretations from decades of assessments into a robust conceptual characterisation of the 38ha site's geology, hydrogeology, contamination distribution, geotechnical properties, data gaps, and potential risks requiring management. The aim is to compile the salient technical information from the comprehensive Tier 1, Tier 2 and Tier 3 reports into an integrated summary conveying the current status of the site and critical considerations for future redevelopment.

The former Anglesey Aluminium site is a 38-hectare brownfield site located near Holyhead in northwest Wales. The site operated as an aluminium smelter from 1970 to 2009. Since closure, the site has undergone extensive intrusive investigations to characterise ground conditions and contamination ahead of potential redevelopment.

Site Geology and Hydrogeology: The subsurface conditions are highly complex, comprising made ground, natural superficial deposits, weathered bedrock, and intact bedrock. Made ground thickness reaches up to 9.5m in places. Natural deposits include glacial tills, sands, gravels, and clays. Bedrock is fractured schist with variable weathering. Groundwater occurs in perched lenses in the superficial deposits and deeper bedrock aquifers. Flow is predominantly horizontal to the northwest. Hydraulic conductivity spans orders of magnitude from 10^{-5} to 10^{-10} m/s reflecting the lithological variability.

Contamination Overview: Localised hydrocarbon and metals contamination has been identified in soils and groundwater related to historical operational activities. Key impacted zones include the Rectifier Yard, Vehicle Refuelling Area, Pitch Tanks, and Green Carbon Plant. Maximum concentrations found historically include TPH up to 8,140 mg/kg, PAHs up to 6,170 mg/kg, PCBs up to 356,170 5 mg/kg, and asbestos in made ground. Groundwater contained pockets of LNAPL and DNAPL. Remediation works have addressed the majority of the risks already.

Geotechnical Properties: Testing indicates highly variable geotechnical properties, with very soft compressible zones to very dense rock. SPT N values range from 8-100 blows. Shear strengths vary from 50 kPa in soils to 8 MPa in rock. Hydraulic conductivities span 10^{-5} to 10^{-10} m/s.

Conceptual Site Model: The accumulated data enables development of a robust conceptual site model integrating information on geology, hydrogeology, contamination distribution, geotechnical parameters, structures, and more. This is imperative to accurately characterise conditions and guide risk management decisions.

In summary, while the former industrial site is complex, the comprehensive historical assessments provide vital insights to inform future redevelopment. With appropriate additional characterisation and risk-based management, the site presents significant opportunities for beneficial reuse.

7. Recommendations

Conduct additional intrusive investigations including boreholes, trial pits, and piezometer installation. Focus on geotechnical testing in outside main construction areas where gaps exist.

Geotechnical testing should comprise SPT, vane shear, CBR, consolidation, classification, strength tests on samples. Piezometers will enable groundwater monitoring. Produce design stage ground investigation reports with geotechnical recommendations.

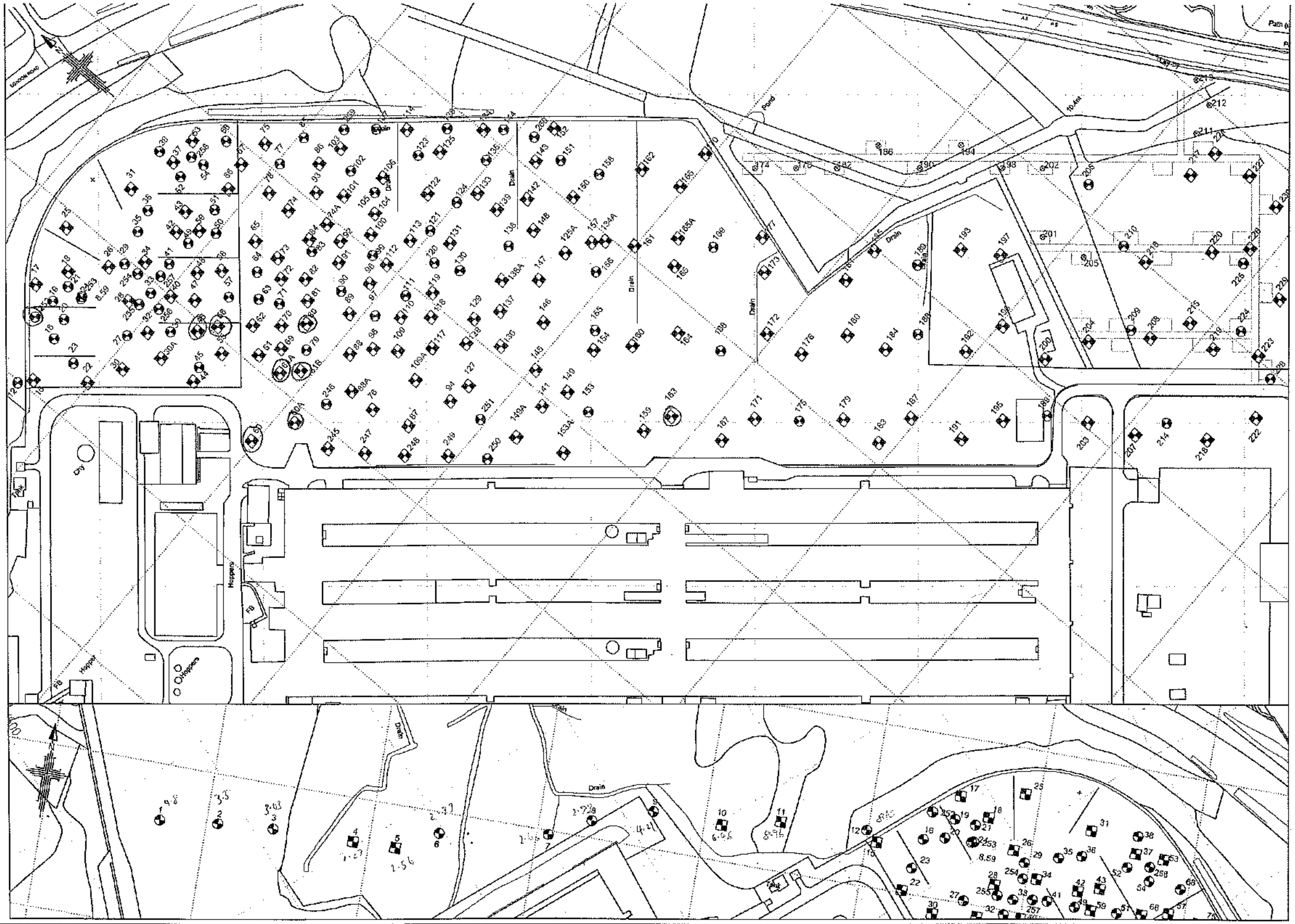
Further evaluate and supplement the groundwater monitoring network through new borehole installation and sampling. Monitoring can continue up to work development.

Adopt geotechnical design tailored to variable ground conditions across the site layout. Implement robust QA like foundation, earthworks, and contamination discovery protocol inspections.

Produce a materials management strategy maximising on-site reuse of excavated soils and rock where feasible. Manage unexpected contamination discoveries via a watching brief.

Develop management plans for matters like dust, ground gas, asbestos, waste soils, surface water, ecology protection through and after construction.

Appendix A: Drawings shows boreholes location of Tier 1 Summaries section.



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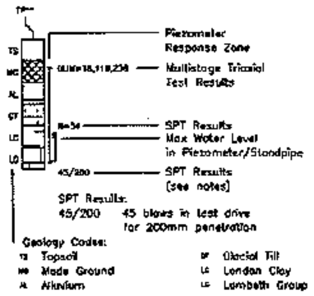

LEGEND TO SYMBOLS

- Borehole Location
- Window Sample Location
- Trial Pit Location
- Concrete Core
- Inspection Pit Location
- Water Sample Location

Surveyed 26th April 2010 (blue)
Surveyed 13th May 2010 (Brown)

Control & Datum Information

Grid Orientation: OSGB36 + OSTN02
Level Datum: OS Orthometric Hts

Rev	Description	By	Date
A	13 May - GI Survey	HSL	13.05.10

Surv.	Drawn	Date	Chkd	Date
NH	NH	26.04.10	MH	27.04.10

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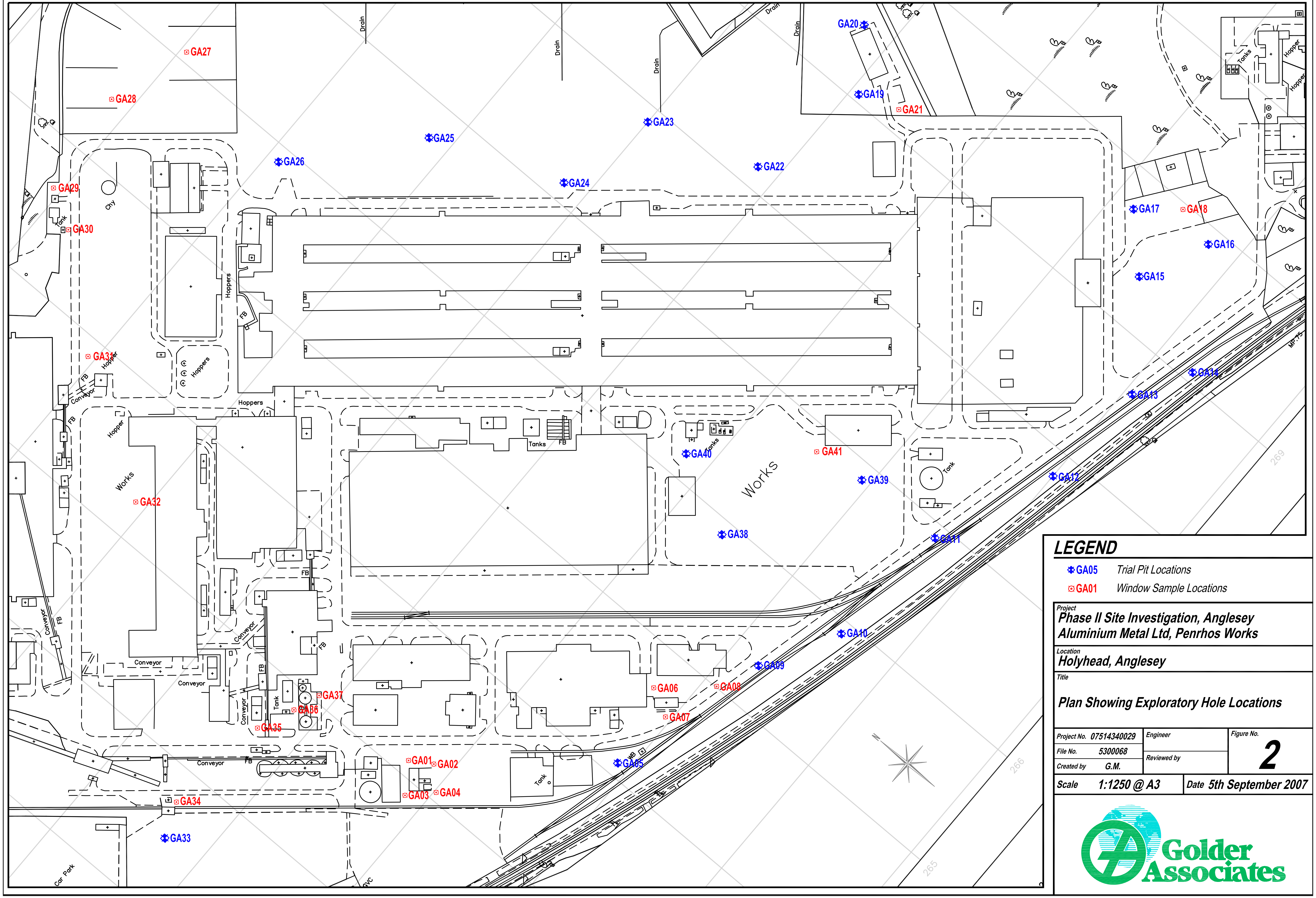
Title.	GI Location Survey
Site.	Anglesey Alliminium Penrhos Anglesey

COMPUTER GENERATED DRAWING - DO NOT ALTER

Dwg No. VIN_043_AA
Sheet No. 1

SCALE	1/1250	REV.	

Appendix B: Drawings shows boreholes location of Tier 2 Summaries section



LEGEND

GA05	Trial Pit Locations
GA01	Window Sample Locations

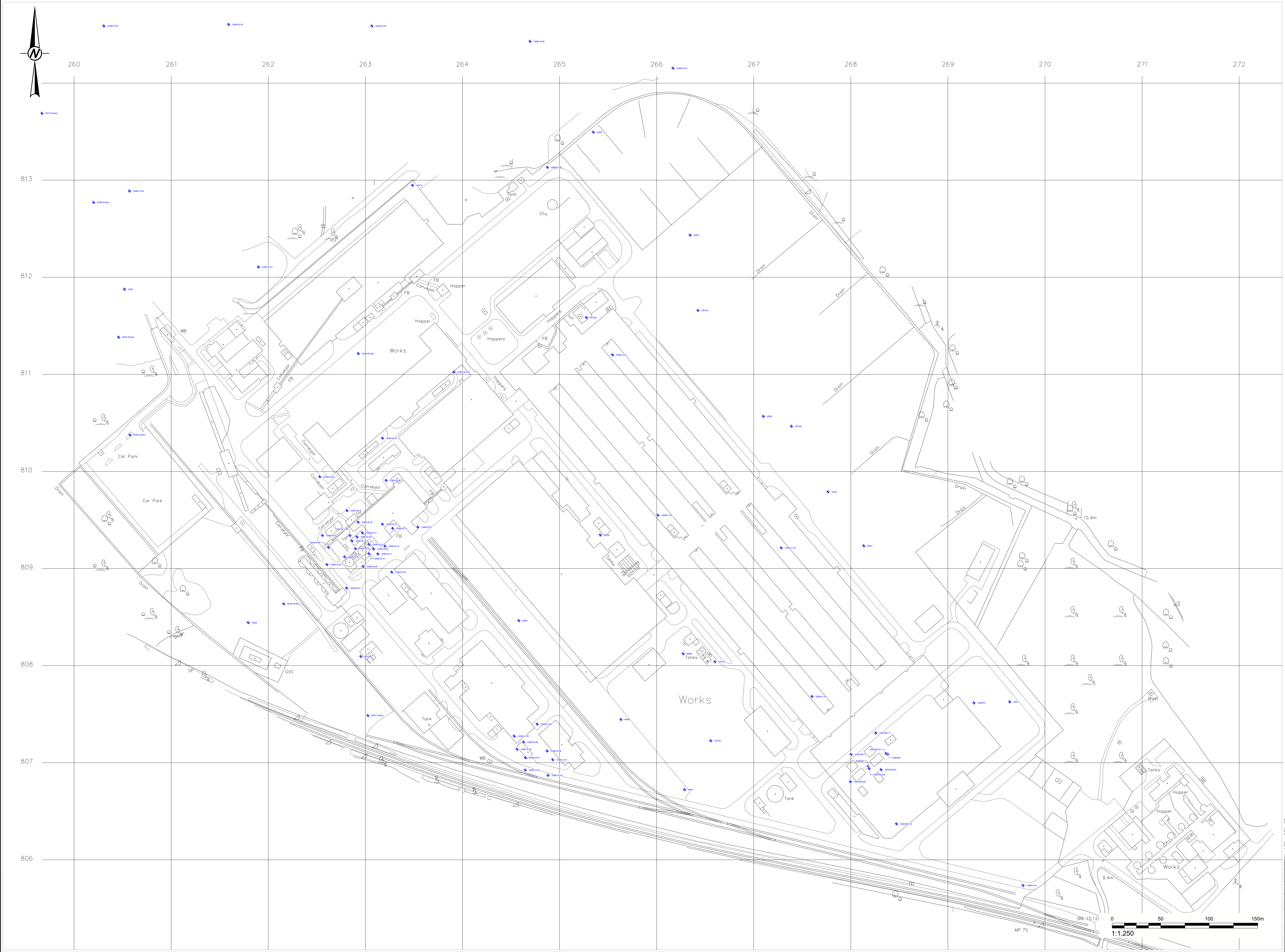
Project
Phase II Site Investigation, Anglesey
Aluminium Metal Ltd, Penrhos Works

Location
Holyhead, Anglesey

Title
Plan Showing Exploratory Hole Locations

Project No. 07514340029	Engineer		Figure No. 2
File No. 5300068			
Created by G.M.	Reviewed by		
Scale 1:1250 @ A3		Date 5th September 2007	

Golder Associates



LEGEND

GROUNDWATER MONITORING BOREHOLE
BOREHOLE INSTALLED FOR GROUNDWATER MONITORING. A DATE IN BRACKETS AFTER THE NAME INDICATES THAT THE BOREHOLE WAS COMPLETED AS PART OF AN EARLIER INVESTIGATION

NOTES
GABH-B-12 IS SCREENED APPROXIMATELY 10CM INTO OVERLYING MG

ALL LOCATIONS LISTED AS PREVIOUSLY COMPLETED AS PART OF AN EARLIER INVESTIGATION ARE APPROXIMATE LOCATIONS ONLY AND HAVE NOT BEEN SURVEYED IN.

REFERENCES
DRAWING BASED ON ORDNANCE SURVEY MAP PROVIDED BY: ANGLESEY ALUMINIUM

Rev	Description	PM	Review	Date

Client	ANGLESEY ALUMINIUM
Project	PITCH TANKS MEMO TO NRW
Title	BOREHOLE LOCATION PLAN

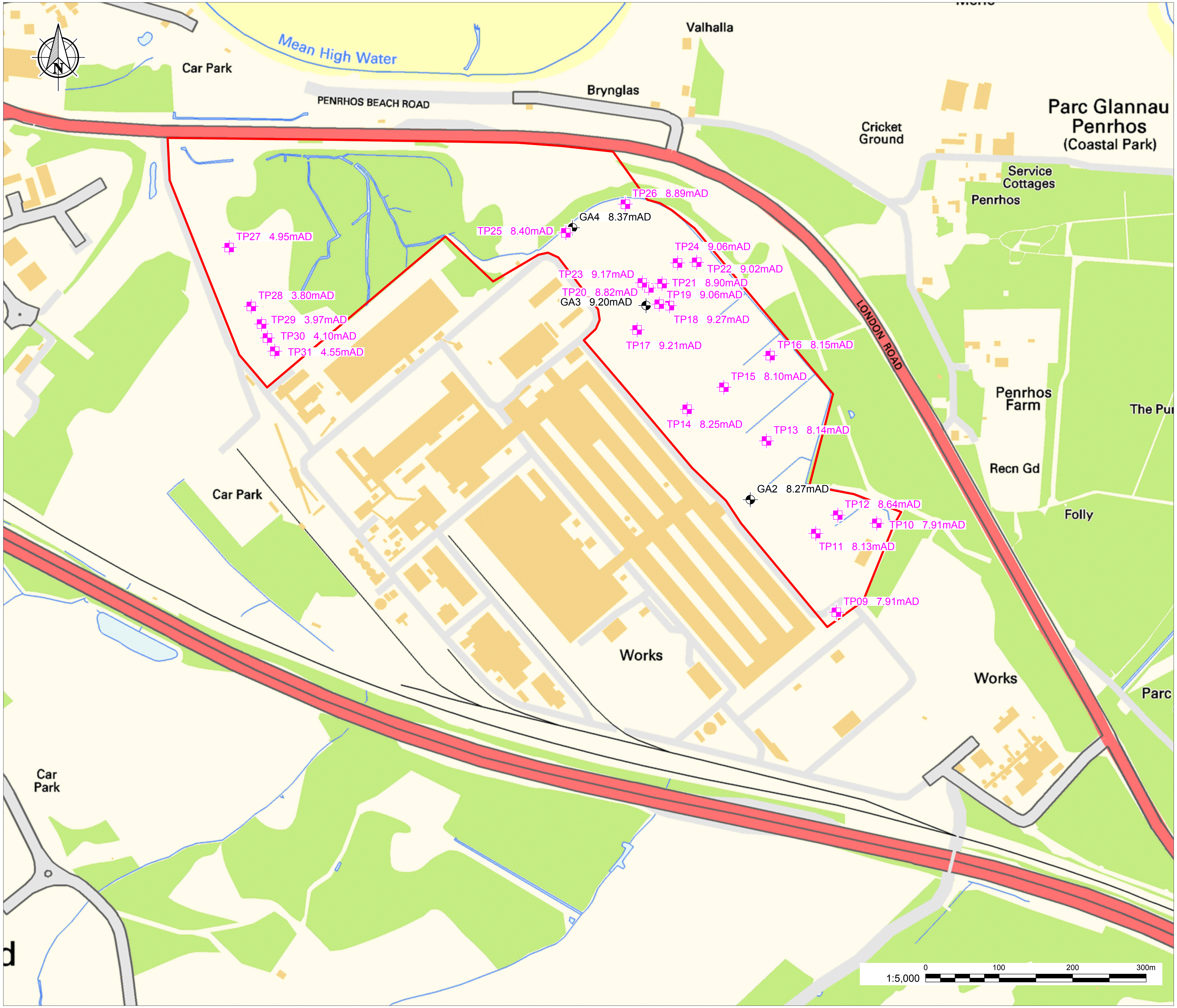
Created by	Requested by	Proj Manager	Reviewer	Date
JW	EDG	EDG	EDG	22/08/13

File No.	Project No.
585429	11514550169

Size	Scale	Status
A0	1:1,250	REPORT ISSUE

Drawing No.	Rev
1	-

GOLDER ASSOCIATES (UK) LTD
GOLDER HOUSE
TADCASTER ENTERPRISE PARK
STATION ROAD
TADCASTER
NORTH YORKSHIRE
LS24 9JF
+44 (0)1937 837 800



Legend

- North Road Area
- BH Borehole Locations
- TP Trial Pit Locations

Notes

Contains Ordnance Survey data © Crown copyright and database right 2011.

Rev	Description	PM	Review	Date

Client: Anglesey Aluminium Metal Ltd

Project: North Road Benchmark Investigation

Title: Investigation Locations

Created by	Requested by	Proj Manager	Reviewer	Date
TJ	HC	EDG	EDG	30/03/12

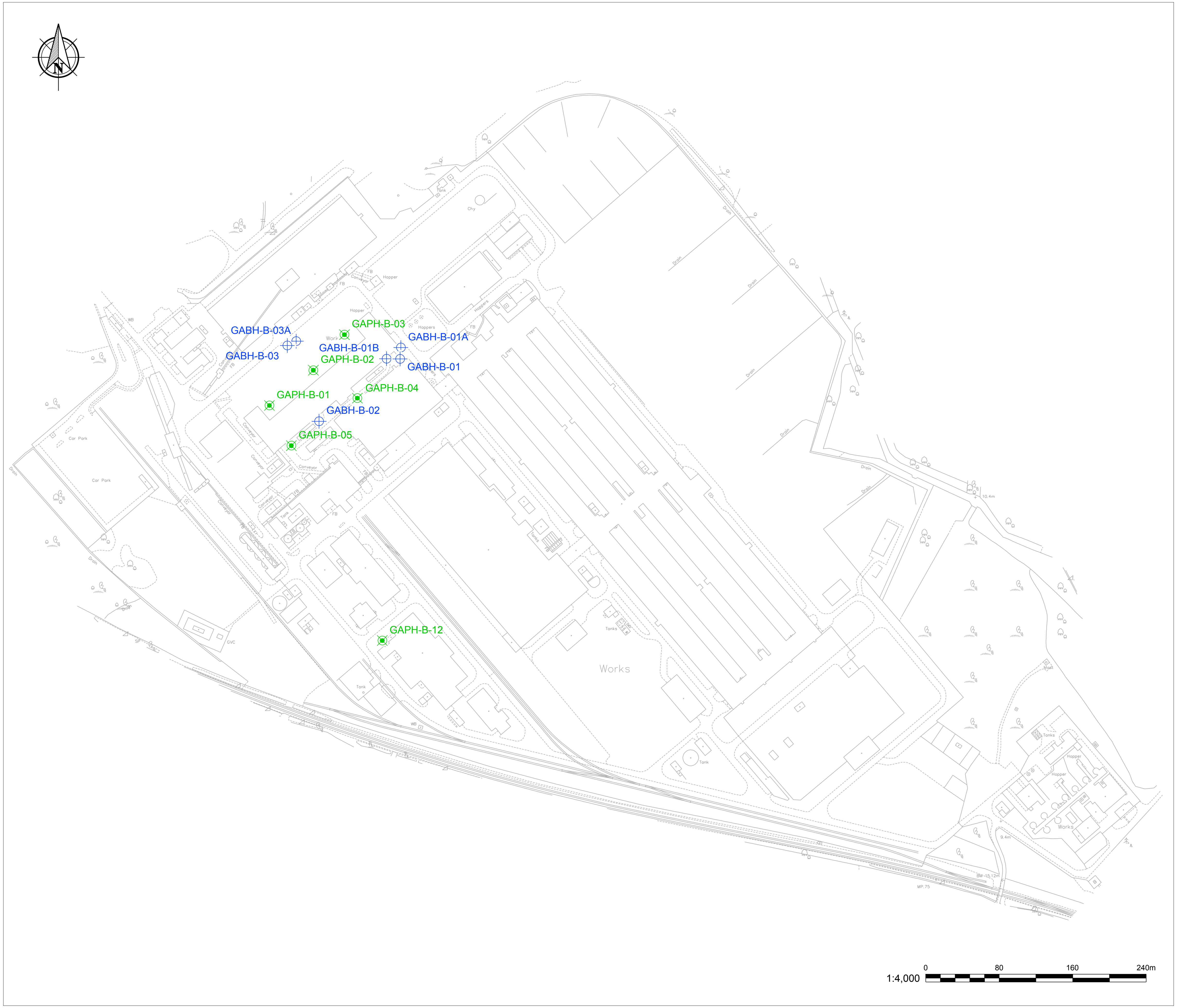
File No.	Project No.
585049	11514550169.503

Size	Scale	Status
A3	1:5,000	Report Issue

Drawing No.	Rev
3	-

Golder Associates

Golder House
Tadcaster Enterprise Park
Station Road
Tadcaster
North Yorkshire
LS24 9JF
+44 (0)1937 837 800



Legend

- Ground investigation borehole
- Ground investigation probe hole

Notes

Rev	Description	PM	Review	Date

Client	Anglesey Aluminium Metal Ltd
Project	Phase B Surrender Factual Report
Title	Borehole and Probe Hole Location Plan

Created by	Requested by	Proj Manager	Reviewer	Date
AD	CM	EDG	EDG	22/02/13

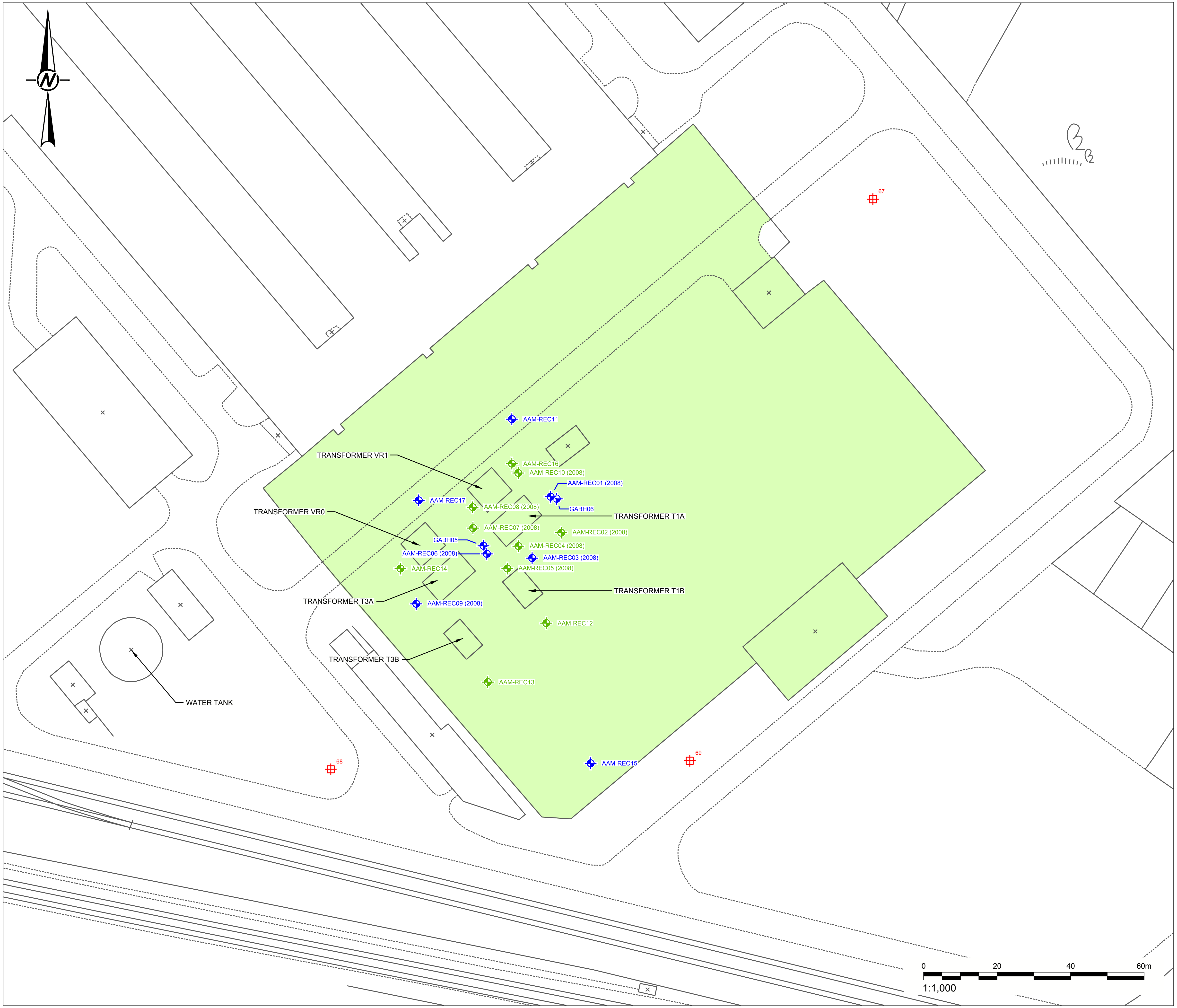
File No.	Project No.
5234613	11514550169.509

Size	Scale	Status
A3	1 : 4,000	Report Issue

Drawing No.	Rev
1	-

Golder Associates

1st Floor Carnmoney House
Edgewater Office Park
Edgewater Road
Belfast
BT3 9JQ
+44 (0)28 9078 7777



LEGEND

RECTIFIER YARD

GROUNDWATER MONITORING BOREHOLE
BOREHOLE INSTALLED FOR GROUNDWATER MONITORING.

BACKFILLED PROBES
PROBEHOLE DRILLED AND BACKFILLED.

APPROXIMATE LOCATION OF PERMIT APPLICATION IPPC ZONE 7 TRIAL PITS.

NOTES
BOREHOLES AND PROBEHOLES WITH (2008) IN THE LOCATION NAME WERE INSTALLED IN 2008 DURING OPERATIONS.

REFERENCES
DRAWING BASED ON INFORMATION PROVIDED BY: ANGLESEY ALUMINIUM

Rev	Description	PM	Review	Date

Client: ANGLESEY ALUMINIUM

Project: PHASE C PERMIT SURRENDER REPORT

Title: RECTIFIER YARD LAYOUT AND INVESTIGATION LOCATION PLAN

Created by	Requested by	Proj Manager	Reviewer	Date
TJ	RV	EDG	EDG	06/01/14

File No.	Project No.
585451	11514550169

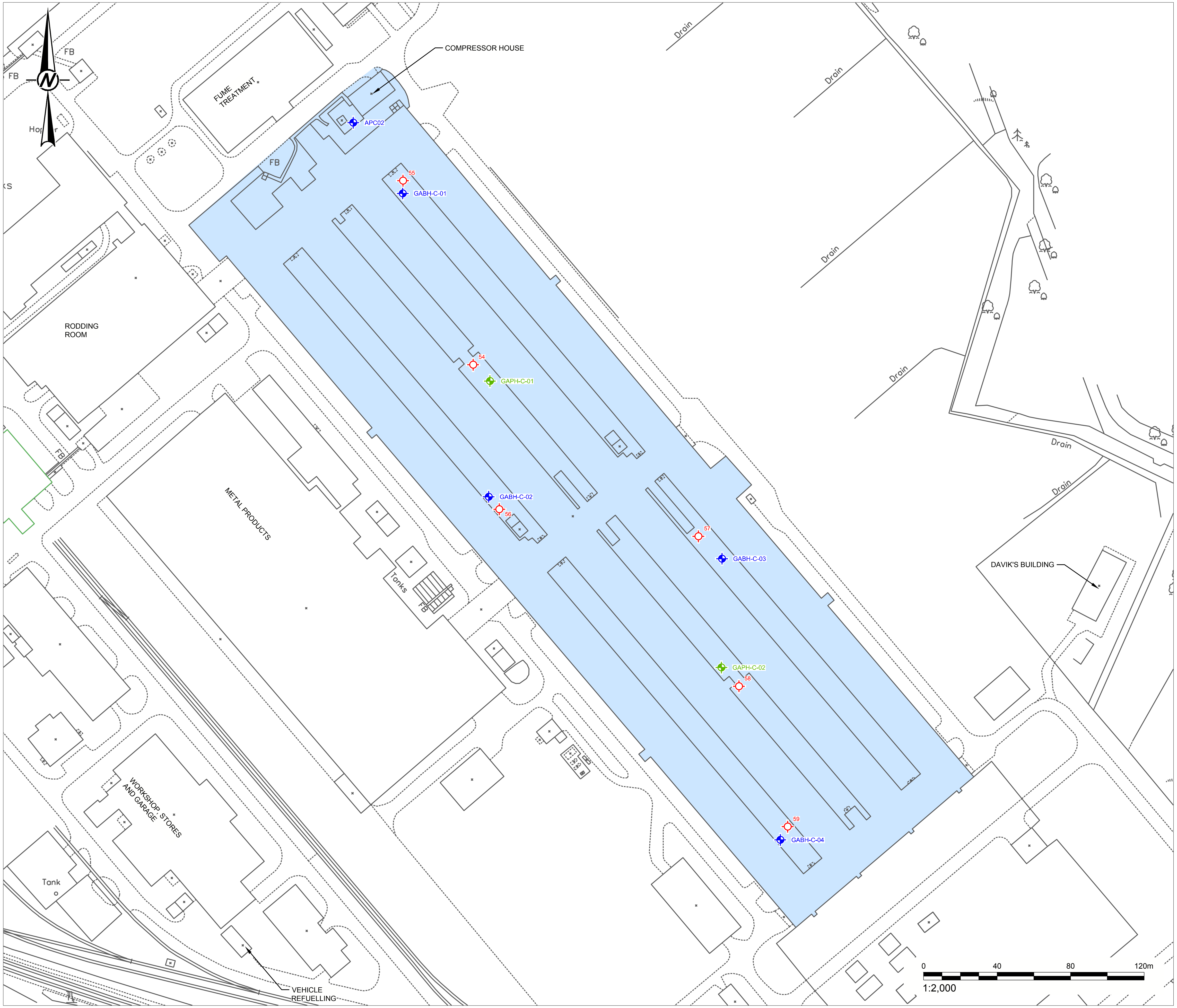
Size	Scale	Status
A3	1:1,000	REPORT ISSUE

Drawing No.	Rev
3	-

Golder Associates

GOLDER ASSOCIATES (UK) LTD
GOLDER HOUSE
TADCASTER ENTERPRISE PARK
STATION ROAD
TADCASTER
NORTH YORKSHIRE
LS24 9JF
+44 (0)1937 837 800

Plot Date: 8 January 2014 Time: 9:53 AM By: Jasinski, Tomasz Path: P:\2011\5_Mining\0000's\11514550169 AAM\450 - WIP\Graphics\Phase C Surrender Report - File Name: 3_585451_Rectifier Yard.dwg
Xref: XREF_OS Base.dwg; Golder_Logo A3.dwg;



LEGEND

- POTLINES
- GROUNDWATER MONITORING BOREHOLE – BOREHOLE INSTALLED FOR GROUNDWATER MONITORING.
- BACKFILLED PROBES – PROBEHOLE DRILLED AND BACKFILLED.
- APPROXIMATE LOCATION OF PERMIT APPLICATION IPPC ZONE 5 BOREHOLE.

REFERENCES
DRAWING BASED ON INFORMATION PROVIDED BY: ANGLESEY ALUMINIUM

Rev	Description	PM	Review	Date

Client: ANGLESEY ALUMINIUM

Project: PHASE C PERMIT SURRENDER REPORT

Title: POTLINES LAYOUT AND INVESTIGATION LOCATION PLAN

Created by	Requested by	Proj Manager	Reviewer	Date
TJ	RV	EDG	EDG	06/01/14

File No.	Project No.
585452	11514550169

Size	Scale	Status
A3	1:2,000	REPORT ISSUE

Drawing No.	Rev
4	-

Golder Associates
GOLDER ASSOCIATES (UK) LTD
GOLDER HOUSE
TADCASTER ENTERPRISE PARK
STATION ROAD
TADCASTER
NORTH YORKSHIRE
LS24 9JF
+44 (0)1937 837 800

Plot Date: 8 January 2014 Time: 9:54 AM By: Jasinski, Tomasz Path: P:\2011\5_Mining\0000's\11514550169 AAM\450 - WIP\Graphics\Phase C Surrender Report - File Name: 4_585452_Potlines.dwg
Xref: XREF_OS Base.dwg; Golder_Logo A3.dwg;

LEGEND

- REMELT AND PONDEROSA
- GROUNDWATER MONITORING BOREHOLE
  BOREHOLE INSTALLED FOR GROUNDWATER MONITORING.
- BACKFILLED PROBES
  PROBEHOLE DRILLED AND BACKFILLED.
- BACKFILLED TRIAL PITS
  TRIAL PIT EXCAVATED AND BACKFILLED.
- APPROXIMATE LOCATION OF PERMIT APPLICATION IPPC ZONE 4 TRIAL PIT.
- APPROXIMATE LOCATION OF PERMIT APPLICATION IPPC ZONE 4 BOREHOLE.
- APPROXIMATE LOCATION OF PERMIT APPLICATION IPPC ZONE 2 TRIAL PIT.

NOTES
PROBEHOLES AND TRIAL PITS WITH (2007) IN THE LOCATION NAME WERE INSTALLED IN 2007 DURING OPERATIONS.

REFERENCES
DRAWING BASED ON INFORMATION PROVIDED BY: ANGLESEY ALUMINIUM

Rev	Description	PM	Review	Date

Client: ANGLESEY ALUMINIUM

Project: PHASE C PERMIT SURRENDER REPORT

Title: REMELT AND PONDEROSA LAYOUT AND INVESTIGATION LOCATION PLAN

Created by	Requested by	Proj Manager	Reviewer	Date
TJ	RV	EDG	EDG	06/01/14

File No.	Project No.
585453	11514550169

Size	Scale	Status
A3	1:2,000	REPORT ISSUE

Drawing No.	Rev
5	-

Golder Associates

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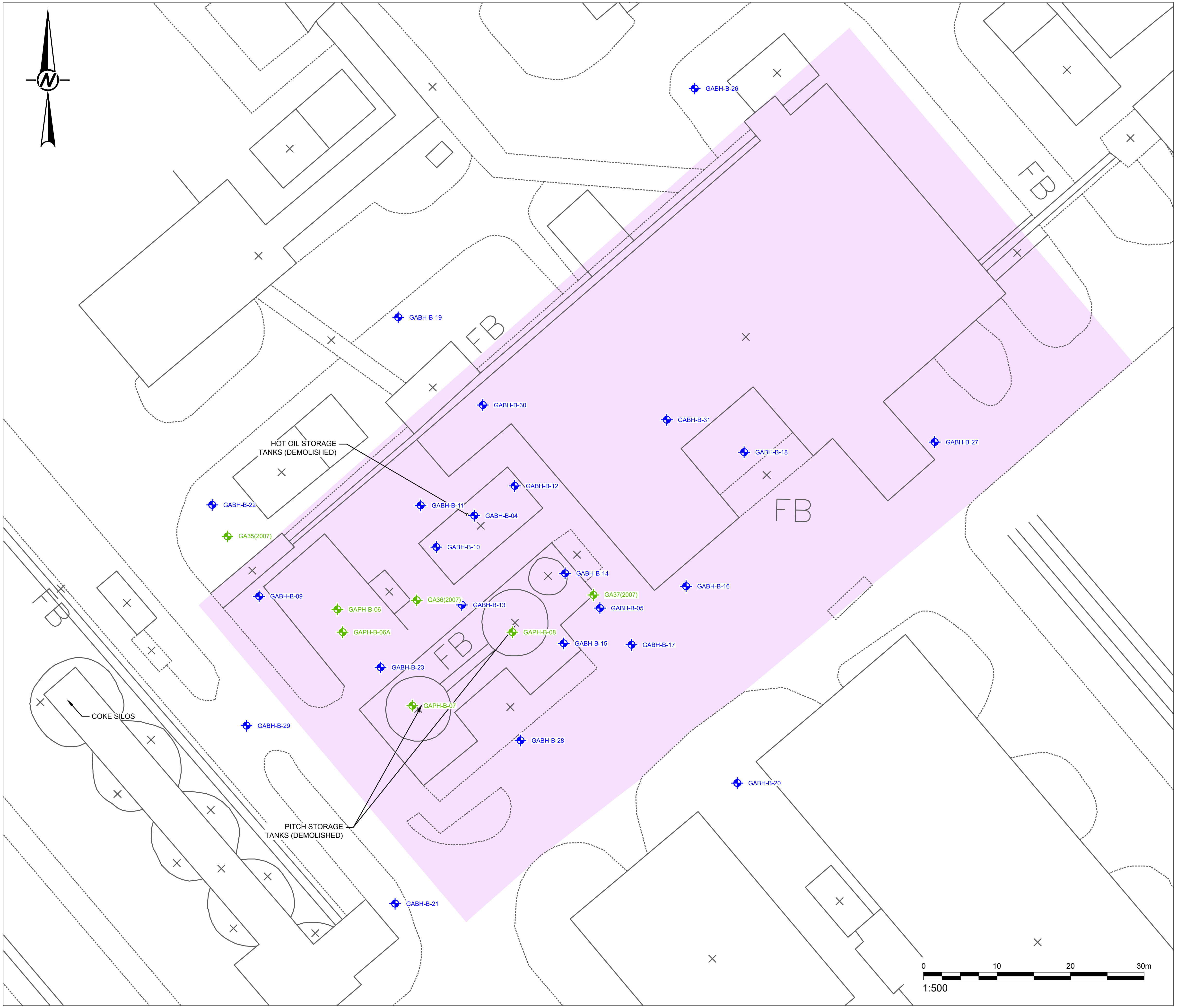
GOLDER HOUSE
TADCASTER ENTERPRISE PARK
STATION ROAD
TADCASTER
NORTH YORKSHIRE
LS24 9JF
+44 (0)1937 837 800



LEGEND

- PITCH TANKS AND GREEN CARBON
- GROUNDWATER MONITORING BOREHOLE
  BOREHOLE INSTALLED FOR GROUNDWATER MONITORING.
- BACKFILLED PROBES
  PROBEHOLE DRILLED AND BACKFILLED.

NOTES

PROBEHOLES WITH (2007) IN THE LOCATION NAME WERE INSTALLED IN 2007 DURING OPERATIONS.

REFERENCES

DRAWING BASED ON INFORMATION PROVIDED BY: ANGLESEY ALUMINIUM

Rev	Description	PM	Review	Date

Client: ANGLESEY ALUMINIUM

Project: PHASE C PERMIT SURRENDER REPORT

Title: PITCH TANKS AND GREEN CARBON LAYOUT AND INVESTIGATION LOCATION PLAN

Created by	Requested by	Proj Manager	Reviewer	Date
TJ	RV	EDG	EDG	06/01/14

File No.	Project No.
585454	11514550169

Size	Scale	Status
A3	1:500	REPORT ISSUE

Drawing No.	Rev
6	-

Golder Associates

GOLDER ASSOCIATES (UK) LTD
GOLDER HOUSE
TADCASTER ENTERPRISE PARK
STATION ROAD
TADCASTER
NORTH YORKSHIRE
LS24 9JF
+44 (0)1937 837 800

Plot Date: 8 January 2014 Time: 9:55 AM By: Jasinski, Tomasz Path: P:\2011\5_Mining\0000's\11514550169 AAM\450 - WIP\Graphics\Phase C Surrender Report - File Name: 6_585454_Pitch Tanks_Green Carbon.dwg
Xref: XREF_OS Base.dwg; Golder_Logo A3.dwg;

LEGEND

- SEWAGE TREATMENT PLANT
- GROUNDWATER MONITORING BOREHOLE
  BOREHOLE INSTALLED FOR GROUNDWATER MONITORING.
- BACKFILLED PROBES
  PROBEHOLE DRILLED AND BACKFILLED.

NOTES
PROBEHOLES WITH (2007) IN THE LOCATION NAME WERE INSTALLED IN 2007 DURING OPERATIONS.

REFERENCES
DRAWING BASED ON INFORMATION PROVIDED BY: ANGLESEY ALUMINIUM

Rev	Description	PM	Review	Date

Client: ANGLESEY ALUMINIUM

Project: PHASE C PERMIT SURRENDER REPORT

Title: SEWAGE TREATMENT PLANT LAYOUT AND INVESTIGATION LOCATION PLAN

Created by	Requested by	Proj Manager	Reviewer	Date
TJ	RV	EDG	EDG	06/01/14

File No.	Project No.
585455	11514550169

Size	Scale	Status
A3	1:1,000	REPORT ISSUE

Drawing No.	Rev
7	-

Golder Associates

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TADCASTER
NORTH YORKSHIRE
LS24 9JF
+44 (0)1937 837 800

Plot Date: 8 January 2014 Time: 9:55 AM By: Jasinski, Tomasz Path: P:\2011\5_Mining\0000's\11514550169 AAM\450 - WIP\Graphics\Phase C Surrender Report - File Name: 7_585455_Sewage Treatment Plant.dwg
Xref: XREF_OS Base.dwg; Golder_Logo A3.dwg;

KEY

Site Boundary

Dual Installation (Shallow Deep)

Elisting Monitoring Well

Window Sample Borehole (Shallow)

Trial Pits

NOTE: Subject to access depth to schist (especially Rectifier Yard)

Sampling Locations and features annotated by LK Consult Ltd are approlimate and are based upon observed measurements unless otherwise stated. Do not scale from this drawing and work from marked dimensions only. All dimensions and features should be confirmed on site by the Contractor. Where this drawing includes information provided to LK Consult Ltd by others, LK Consult Ltd gives no warranty, representation or assurance as to the accuracy of such information.

LK CONSULT LIMITED

Client: Orthios				
Site: Anglesey Aluminium, Holyhead				
Title: Proposed Sampling Location Plan				
Job No.: LKC 14 1181		Scale (See Scale Bar): 1:5000 A3	Figure: 5	Revision:
Drawn By: AC	Checked By: CH	Drawn: Feb 2016		

Eltract from: McCormick Architecture
Drawing Ref: A(100)04, RevC (25/02/15)

Appendix C – HBGS FAAP Geotechnical Ground Model and Design Parameters

Geotechnical Remediation Strategy

Technical Note: Geotechnical Ground Model and Design Parameters

HBGS

HB Geotechnical Services

Table of Revisions

Date	Revision No.	Comments
10 Nov 2023	-	First issue

Job Title	Geotechnical Remediation Strategy Geotechnical Ground Model and Design Parameters		
Client	Anglesey Land Holdings		
Location	Holyhead; Isle of Anglesey		
Date	10 Nov 2023	**Revision**	-
Prepared by	Nguyen Thi Mai Linh BSc MSc	**Signature**	
Reviewed by	Eur Ing Robert Hutchison BEng MSc MBA FGS M.ASCE CEng MICE	**Signature**	
Authorised by	Eur Ing Robert Hutchison BEng MSc MBA FGS M.ASCE CEng MICE	**Signature**	

HBGS

Contents

List of Notations

Symbol	Units	Meaning
Is50	MPa	Point load results were normalized for a 50mm diameter specimen.
NMC	%	Nature Moisture content
LL	%	Liquid Limit
PL	%	Plastic Limit
PI	%	Plasticity Index
CBR	%	California Bearing Ratio
k	m/s	Permeability
E_{intact}	MPa	Young's module of intact rock.
RQD	%	Rock Quality Designation
c	kPa	the cohesion component of shear strength
ø	°*(degree)*	The friction angle
q_c	MPa	The ultimate bearing capacity of the rock mass.
E_m	MPa	The deformation modulus of the rock mass.
n	Non-unit	Poisson ratio
γ_b	kPa	The bulk unit weight
USC	MPa	Unconfined Compression Test

1. Purpose of this Technical Note

1.1 Introduction

This technical note has been prepared to summarize the findings of historical ground investigations provide a geotechnical ground model and design parameter for the proposed rehabilitation of the former Holyhead aluminium smelting site in Wales.

The Holyhead site covers approximately 38 hectares and operated as an aluminium production facility from 1970 until closure in 2009. Since closure, the site has undergone extensive intrusive site assessments to characterize ground conditions and contamination ahead of potential redevelopment.

The purpose of this technical note is to:

i. Provide an overview of the site layout, history, and proposed rehabilitation works.

ii. Summarize the subsurface conditions across the site based on previous intrusive investigations.

iii. Provide a Geotechnical ground model.

iv. Present key geotechnical properties derived from historical testing data.

v. Identify potential geotechnical hazards that the ground model to given rise to.

This technical note consolidates the findings from decades of assessments into a robust ground model and associated geotechnical engineering parameters. It aims to transform the raw subsurface data into practical engineering recommendations to facilitate safe, efficient, and cost-effective design and construction for the proposed rehabilitation.

1.2 Structure of the Technical Note

This technical note has the following reporting structure:

i. Introduction;

ii. Historical Information and Relevant Extracts from Reports;

iii. Geological and Geotechnical Ground Model;

iv. Geotechnical Design Parameters;

v. Discussion and Suggestions

1.3 Limitations of the Technical Note

i. The factual data comes from investigations conducted between the 1990s to early 2010s. No new boreholes have been advanced in the last 5-10 years. Ground conditions may have changed, especially in shallow zones.

ii. The majority of geotechnical testing results were obtained during the 2010 site investigation campaign. Limited additional testing has been done since then to expand or refine the dataset.

iii. Made ground composition and properties are highly variable across the site. Generic parameters have been assigned, but these will need verification testing during rehabilitation.

iv. Superficial deposit thickness is known to vary significantly, with localized thick and thin zones. Depths require confirmation during construction.

v. The degree of schist weathering and fracture density cannot be accurately delineated from sparse borehole data. Additional probing may be needed, especially in foundation locations.

vi. Construction records of original plant earthworks are incomplete. Buried foundations and other obstructions may be uncovered during excavations.

vii. Groundwater monitoring data is limited. Water strikes during drilling provide generalized levels, but short-term and seasonal fluctuations are unknown.

viii. There are gaps in geotechnical coverage in the outer lying areas away from the main industrial plant. Additional testing requirements are expected.

ix. No geotechnical information could be found for some peripheral parts of the site. Significant risks may still exist in these uncharacterized areas.

x. While providing a strong baseline understanding, the limitations mean additional confirmatory testing will be prudent during rehabilitation to validate design assumptions and manage risks appropriately.

2. Historical Information and Relevant Extracts from Reports

2.1 Introduction

Summarizing the criteria used for including or excluding historical reports in the Holyhead ground model and geotechnical assessment:

i. The extensive historical site assessments include a mix of factual, interpretative, geotechnical, and environmental reports focused on both shallow and deep subsurface conditions. In developing the ground model and geotechnical parameters for the Anglesey Aluminium REP, Holyhead site, the criteria used for selecting the most applicable reports were:

ii. Inclusion - Reports containing substantial factual data or geotechnical interpretation related to deeper subsurface conditions below 10m depth. These provide the most relevant information on ground conditions and properties influencing foundation design and construction.

iii. Exclusion - Reports focused solely on geo-environmental aspects like contamination testing or human health risk assessments. These offer limited value for geotechnical engineering purposes.

iv. Exclusion - Reports where borehole locations lack coordinate information or detailed logs. Many historical assessments, especially early 1990s reports, do not provide borehole coordinates tied to a coordinate system or contain minimal borehole log details. The lack of precise borehole locations and thorough logging makes correlation across investigations difficult and introduces uncertainty in developing the ground model.

By applying these criteria, the reports most pertinent to developing a robust ground model, determining representative geotechnical parameters, and enabling fit-for-purpose foundation design were prioritized and summarized. Reports dealing exclusively with environmental testing were excluded as they provided limited technical merit for the geotechnical assessment and rehabilitation recommendations.

2.2 Information Reviewed and Taken into Inclusion.

The following reports were reviewed, with salient information being considered in the compilation of the ground model and design parameters:

i. 2010 07 Anglesey Aluminium REP, Ground Investigation Report, Mott MacDonald.

ii. 2010 07 Report on a Ground Investigation at Anglesey Aluminium REP, Holyhead (Volume One), Soil Engineering.

iii. 2010 07 Anglesey Aluminium REP, Geotechnical Design Report, Mott MacDonald.

iv. 2012 05 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase A, Golder Associates Ltd.

v. 2008 01 Phase II Environmental Site Investigation of Anglesey Aluminium Metal Ltd, Penrhos Works, Holyhead, Anglesey, Golder Associates Ltd.

vi. 2012 04 Anglesey Aluminium Metal Ltd, North Road Benchmark Investigation, Golder Associates Ltd.

vii. 2013 02 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase B, Golder Associates Ltd.

viii. 2013 06 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Garage Area, Golder Associates.

ix. 2014 01 Section 9 of the Site Condition Report for Environmental Permit BL1100IX, Anglesey Aluminium Metal Ltd Penrhos Works-Phase C, Golder Associates.

x. 2015 12 Anglesey Aluminium Metal Ltd, Factual Report on Further Intrusive, Investigation of Pitch Tanks and Anode Bake Area, Golder Associates Ltd.

2.3 Reports Reviewed and not taken into exclusion.

The following reports were available to be considered, if required, but were discounted due to the reasons in §2.1:

i. 1994 09 Anglesey Aluminium Metal Limited, Environmental Assessment, Groundwater and Land Contamination Survey Report, Wallace Evans Limited.

ii. 2001 09 Phase 1a Site Report for IPPC Application at the Anglesey Aluminium Facility, Holyhead, Anglesey, URS.

iii. 2013 05 Anglesey Aluminium Metal Limited and Anglesey Aluminium Metal Renewables, Conceptual Site Model, Golder Associates Ltd.

iv. 2013 08 Summary of Investigation in the Pitch Tanks Area, Anglesey Aluminium Metal Ltd, Pernhos Works, Holyhead, Golder Associates Ltd.

v. 2016 03 Renewable Energy Plant (REP) Former Anglesey Aluminium Works Site

vi. Investigation Brief, LK Consult (LKC),

vii. 2016 03 Anglesey Aluminium Holyhead, Phase 1 Preliminary Risk Assessment, LK Consult (LKC).

viii. 2016 04 Anglesey Aluminium Holyhead, Phase 2 Geo-Environmental Investigation and Risk Assessment, LK Consult (LKC).

ix. 2016 05 Anglesey Aluminium REP, Holyhead, Remediation Strategy, LK Consult (LKC).

x. 2016 05 Anglesey Aluminium REP, Holyhead, Escrow Sites Scope of Works, LK Consult (LKC).

xi. 2017 06 Escrow Site 1 (Rectifier Yard) Anglesey Aluminium Holyhead, Delineation Investigation and Risk Assessment, LK Consult (LKC).

xii. 2017 06 Escrow Site 1 (Rectifier Yard) Anglesey Aluminium Holyhead, Remediation Strategy, LK Consult (LKC).

xiii. 2017 06 Escrow Site 2 (Vehicle Refuelling Area) Anglesey Aluminium Holyhead, Delineation Investigation and Qualitative Risk Assessment, LK Consult (LKC).

xiv. 2017 06 Escrow Site 2 (Vehicle Refuelling Area) Anglesey Aluminium Holyhead, Remediation Strategy LK Consult (LKC).

xv. 2017 06 Escrow Site 3 (Compressor House) Anglesey Aluminium Holyhead, Delineation Investigation and Qualitative Risk Assessment, LK Consult (LKC).

xvi. 2017 08 Escrow Site 3 (Compressor House) Anglesey Aluminium Holyhead, Remediation Strategy, LK Consult (LKC).

xvii. 2021 09 Former Anglesey Aluminium Site, Holyhead, Historical Review of the Site Investigations Undertaken in the Area of the Proposed Plastics Depolymerisation Unit (PDU), LK Consult (LKC).

xviii. 2009 08 Section 14, Summary of Mitigation and Monitoring.

xix. 2014 03 Anglesey Aluminium Metal Ltd, Site Wide Groundwater Monitoring September 2013-February 2014, Golder Associates Ltd.

xx. 2014 07 Anglesey Aluminium Metal Site, Ecological Surveys Position Statement, Ramboll UK Ltd.

xxi. 2014 10 Reptile Survey, Anglesey Aluminium, Kestrel Environmental Services.

xxii. 2014 12 Anglesey Aluminium Metal Site, Ecological Constraints Note, Ramboll UK Ltd.

xxiii. 2015 06 Anglesey Aluminium Site, Preliminary Ecological Walkover, Ramboll UK Ltd.

xxiv. 2015 07 Anglesey Aluminium Renewable Energy Plant Great Crested Newt EDNA Testing Report, Ramboll UK Ltd.

xxv. 2015 08 Anglesey Aluminium Renewable Energy Plant Reptile Survey Report, Ramboll UK Ltd.

xxvi. 2017 03 Anglesey, Laboratory Bench Scale Chemical Oxidation Study, CE Geochem Ltd.

xxvii. 2017 04 Remediation Pilot Study, Escrow Areas Anglesey Aluminium Penrhos Works Holyhead, Geo2 Remediation Limited.

xxviii. 2018 02 Escrow Site 3 (Compressor House) Former Anglesey Aluminium Holyhead, Groundwater Risk Assessment Consult (LKC).

xxix. 2020 07 Orthios, Former Anglesey Aluminium, Pernhos-Borehole Decommissioning

xxx. Report, LK Consult (LKC).

xxxi. 2021 03 Orthios, Former Anglesey Aluminium, Pernhos-Post Remediation

xxxii. Monitoring Report (Escrow Site 1-Rectifier Yard Area), LK Consult (LKC).

xxxiii. 2021 03 Orthios, Former Anglesey Aluminium, Pernhos -Post Remediation Monitoring Report (Escrow Site 2-Garage Area), LK Consult (LKC).

xxxiv. 2021 03 Orthios, Former Anglesey Aluminium, Pernhos-Post Remediation Monitoring Report (Escrow Site 3-Compressor House Area), LK Consult (LKC).

3. Geological and Geotechnical Ground Model

3.1 Introduction

The the Anglesey Aluminum REP, Holyhead site comprises a complex sequence of man-made and natural geological units, reflecting its industrial history and modification through development. Historical borehole logs and trial pit observations spanning decades of investigation have been compiled to characterize the vertical and horizontal distribution of stratigraphic units across the 38-hectare site.

This section summarizes the key features of the interpreted ground model in terms of made ground, superficial natural deposits, bedrock geology, and hydrogeology The ground model forms the basis for engineering analysis, with geotechnical parameters assigned to each unit as described in subsequent sections of this report.

By consolidating the extensive site investigation data, a robust understanding of subsurface conditions can be developed. This provides the geological framework underpinning the geotechnical recommendations for the proposed rehabilitation.

3.2 Key Aspects of the Geological Model

i. Presence of made ground across most of the site related to previous demolition and construction activities. Made ground composition and properties are highly variable. Thickness reaches about larger 5m in localized areas.

ii. Distribution and depth of natural superficial deposits comprising glacial clays, silts, sands and gravels. Deposits are thinner or absent in developed areas where excavation occurred historically. Thickness varies from 0-15m.

iii. Geological structure and depth of the foliated schist bedrock. Fracture density decreases with depth. Weathering is present in the upper zones, with un-weathered rock at depth. Encountered from ground level to around 12m below surface.

iv. Groundwater table locations in both the superficial deposits and deeper schist aquifers. Measured levels indicate predominantly horizontal flow directions towards the northwest.

v. Changes in stratigraphy and material properties laterally across the site reflecting localized disturbance, infilling, excavation, and variable geology.

vi. Presence of demolition debris, concrete blocks, and other obstructions within made ground that could impact future construction.

vii. Differences between developed areas with thick made ground and more natural zones with remaining superficial deposits.

viii. Boundaries between stratigraphic units require generalization given sparse borehole data. Actual transitions may be more gradational.

ix. Extrapolation of conditions between borehole locations carries inherent uncertainty due to geological complexity.

3.3 Aspects of the Geological Model Which are Irrelevant.

Certain aspects of the geological model were considered extraneous details for the purposes of geotechnical assessment and developing engineering recommendations for the site rehabilitation. These unimportant aspects based on the historical data review include:

i. Intralayer lithological variations within made ground - The made ground composition is highly heterogeneous demolition rubble and fill of variable materials, grain sizes, and properties. Logging minor lithological differences within the made ground was deemed unnecessary.

ii. Detailed stratigraphic divisions of superficial deposits - The natural superficial deposits comprise interbedded sequences of glacial clays, silts, sands, and gravels. Delineating subtle lithological bands within these layers was not required.

iii. Detailed schist mineralogy - Notations on mineral constituents and textures in the schist did not contribute inform engineering characterization.

iv. Structural geology - Detailed fracture orientation measurements had minimal influence on geotechnical assessment.

By focusing on the key geological attributes required for developing engineering properties and rehabilitation recommendations, aspects peripheral to this objective could be excluded. This helped streamline the historical data into an optimal ground model providing the necessary inputs for geotechnical interpretation and design.

3.4 Strata Descriptions

Made Ground: Heterogeneous fill material comprising sandy clay, gravel, ash, metal, concrete, refractory brick, wood, plastic and other waste. Thickness varies across site from 1 to 5m. Anthropogenic inclusions common.

Superficial Deposits: Variable sequence of natural glacial and alluvial deposits including clay, silty clay, sand, gravel, cobbles. Thickness 1m to 15m. Gravelly sand and silty clay common

Schist Bedrock: Mica schist bedrock of Mona Complex, Precambrian to Cambrian in age. Micaceous with narrow quartz banding. Grey to bluish grey, medium grained metamorphic rock. Highly fractured near surface but fracture frequency reduces with depth.

3.5 Groundwater Levels

Groundwater was encountered at varying depths across the site, ranging from just below ground level to over 12m depth.

Continuous monitoring between May 13 to June 1, 2010, suggested relatively consistent groundwater levels, especially at deeper locations across the site.

Localized shallow groundwater pockets were present within the superficial geology, likely due to the presence of low permeability layers.

Standing water was observed at multiple locations during site investigations.

Groundwater levels exhibited tidal fluctuations in marshy low-lying areas along the conveyor route to the west.

Some hydraulic connections were evident between neighbouring boreholes up to 20m apart during drilling and testing.

The depth to groundwater measured 20 minutes after drilling showed a variable distribution reflecting localized conditions.

Deeper and more consistent groundwater was encountered above the rockhead in the reclamation yard area in the centre of the site.

The connection between groundwater at the site and nearby surface water bodies like the Irish Sea remained unconfirmed based on available data.

3.6 Summary of the Geological and Geotechnical Model

Figure 3.1 displays the distribution of boreholes and trial pits as extracted from the geotechnical and environmental report spanning the years 2010 to 2015.

The geological model has been interpreted to have the following attributes:

i. Figure 3.2 presents the isolated locations of Made Ground, along with their respective thickness ranging from approximately 1 to 5 meters.

ii. The horizon of Superficial Deposits is characterized by a variable sequence of natural glacial and alluvial deposits, which encompass materials such as clay, silty clay, sand, gravel, and cobbles. Figure 3.3 displays the common locations of gravelly sand and silty clay within this horizon.

iii. Figure 3.4 provides an overview of the Schist horizon, encompassing both Weathered Schist and Schist Bedrock. The Weathered Schist is characterized by highly fractured greenish grey mica schist with clay and quartz veins, while the Schist Bedrock consists of micaceous, medium-grained metamorphic rock with varying fracture density, reflecting its depth within the Precambrian to Cambrian Mona Complex.

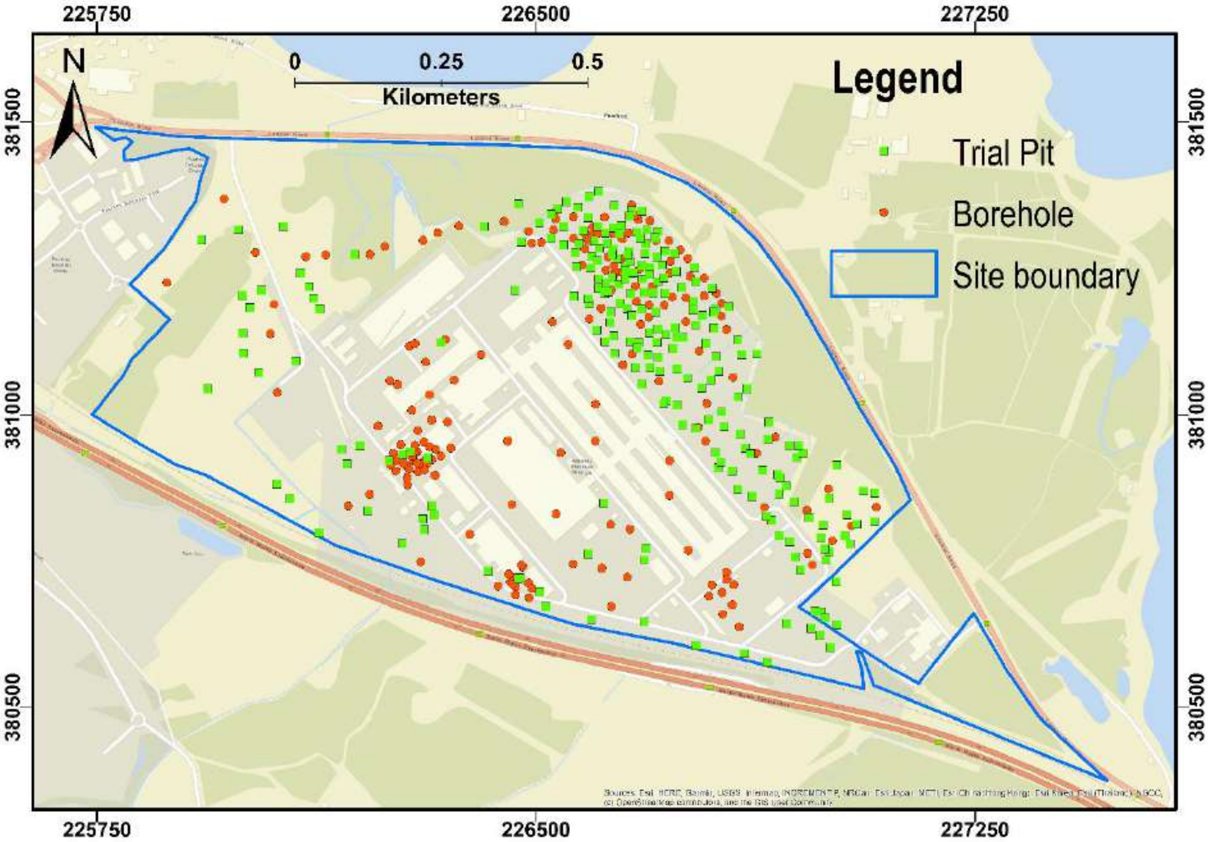

Figure 3.1: The map shows boreholes/trail pits locations (2010 – 2015) in Anglesey Aluminium REP, Holyhead.

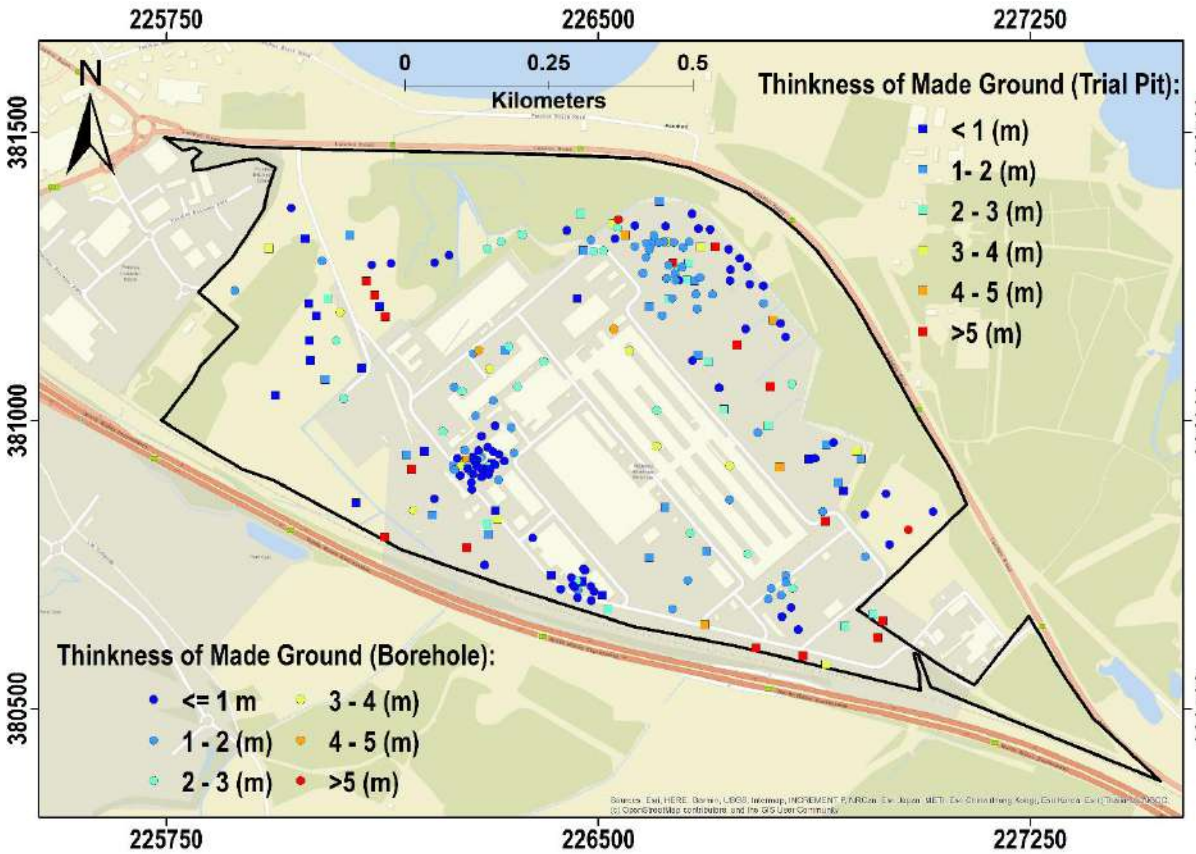

Figure 3.2: The map shows Borehole/Trial Pit Locations along with Made Ground thickness in each location.

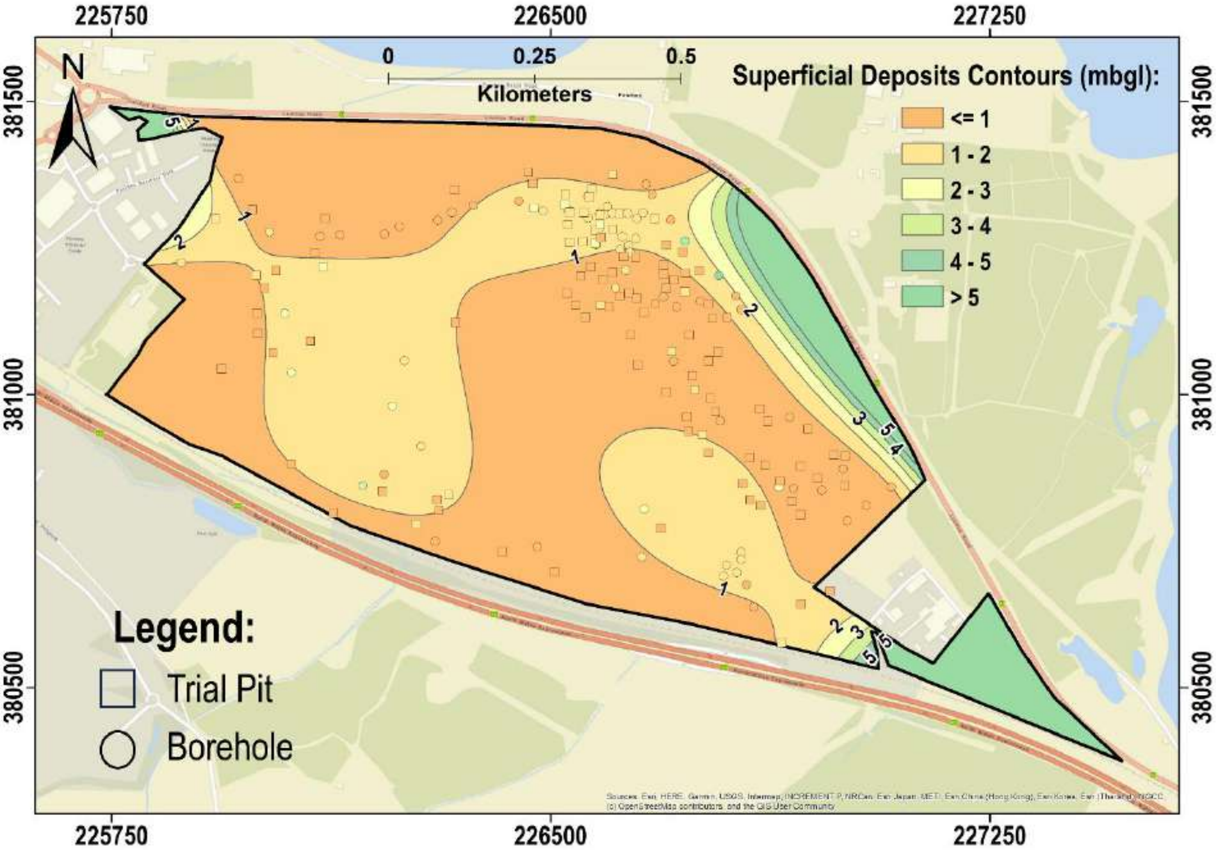

Figure 3.3: Upper horizon of Superficial Deposits and these observation through Boreholes/Trial Pits

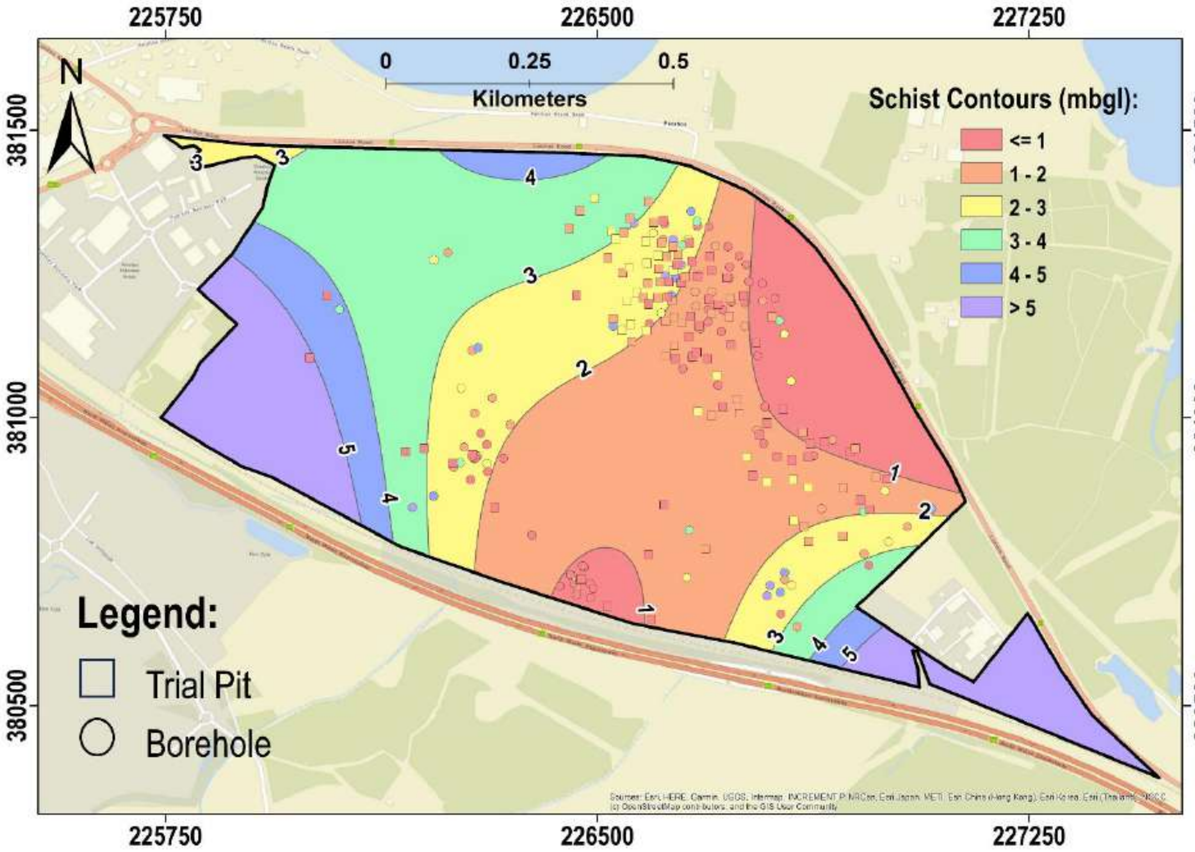

Figure 3.4: Upper horizon of Schist and these observation through Boreholes/Trial Pits

4. Geotechnical Parameters

4.1 Introduction

It's worth noting that most of the reports related to Anglesey Aluminium REP in Holyhead predominantly focus on environmental aspects, with geotechnical information being scarce. So, the valuable geotechnical insights are contained within the "Report on a Ground Investigation at Anglesey Aluminium REP, Holyhead (Volume One)" published by the Soil Engineering company in July 2010 and Anglesey Aluminium REP, Ground Investigation Report of Mott MacDonald company. The locations of boreholes from this critical report are illustrated in Figure 4.1. This section states these geotechnical design parameters for the Anglesey Aluminium REP, Holyhead.

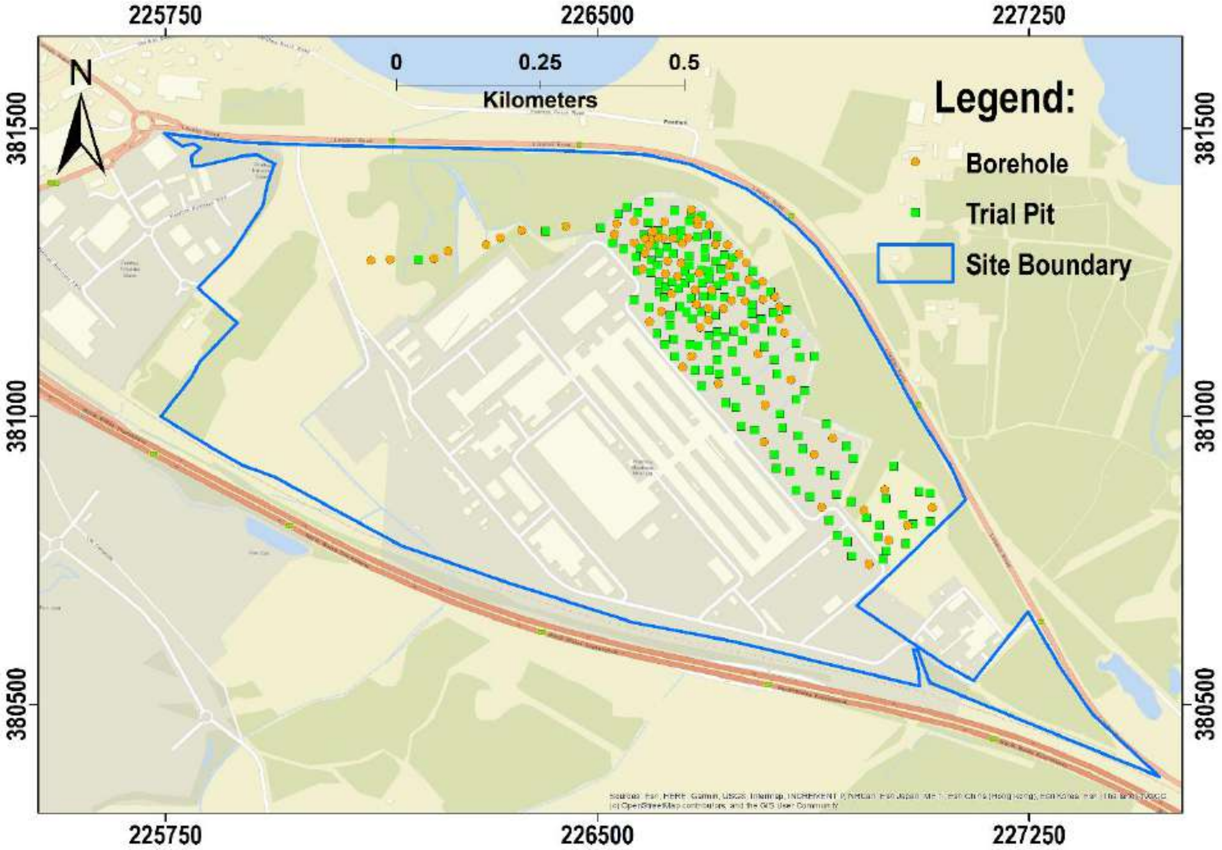

Figure 4.1: The map shows the locations of boreholes from the "Report on a Ground Investigation at Anglesey Aluminium REP, Holyhead (Volume One)" published by the Soil Engineering company in July 2010.

At present, the design parameters have been derived for the various strata, to be applied wherever that strata is on the site. It is not the purpose of this note to present and derive individual ground models and parameters for each and every type of analysis that has been (or needs to be) undertaken. That is managed within the relevant analyses themselves and reported elsewhere.

4.2 Summary Geotechnical Design Parameters

The geotechnical investigation reveals distinct strata with varying SPT counts with depth in Figure 4.2:

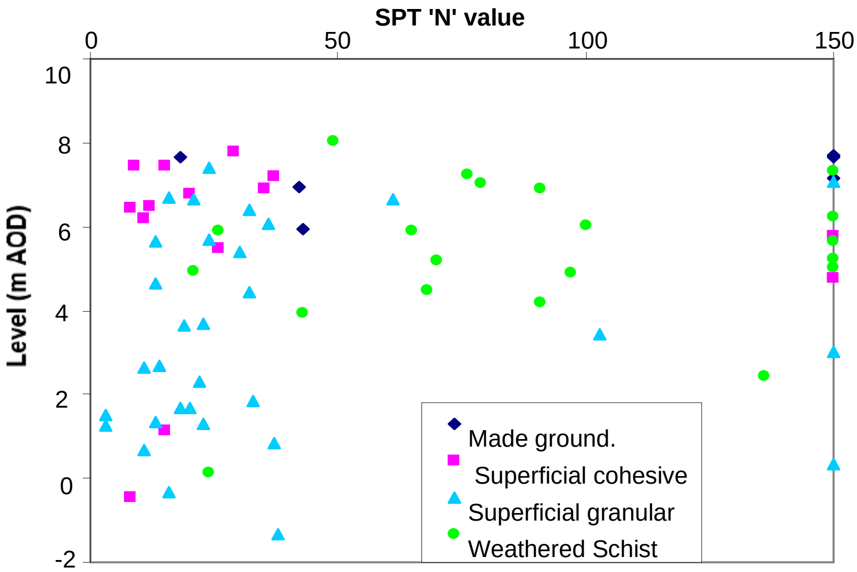

Figure 4.2: : SPT profile through made ground and drift deposits at Anglesey Aluminium REP

The conducted CBR tests, averaging a value of 11.5%.

The Rock Quality Designation (RQD) varies with depth and between boreholes, displaying little correlation, with an average RQD of 22% and a range of 15% to 30%, providing insights into the heterogeneous nature of rock quality in the studied geological area:

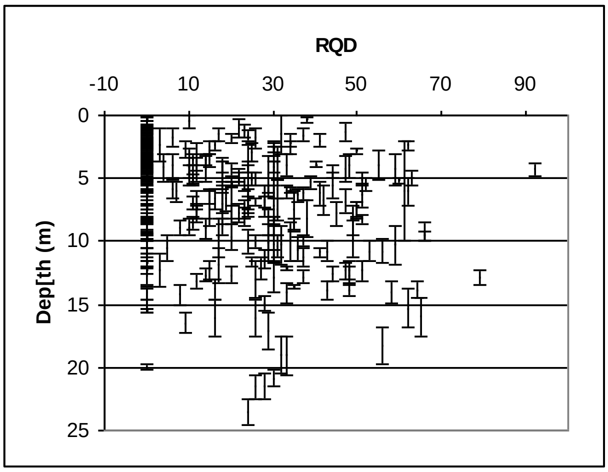

Figure 4.3: Variation of RQD with depth for the REP site

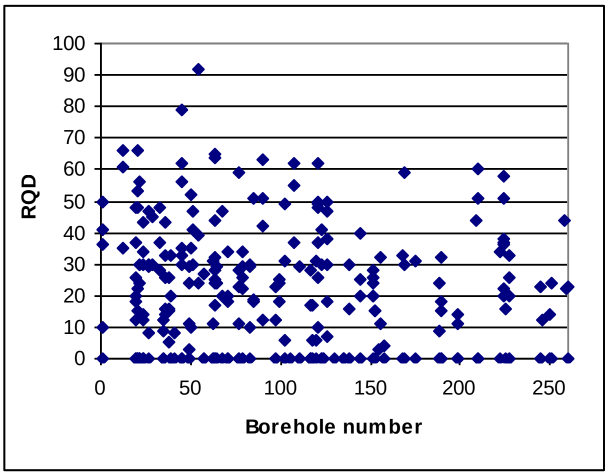

Figure 4.4: Variation of RQD across exploration locations

HBGS

The scatter plot in Figure 4.5 illustrates the distribution of UCS values with depth, showing the majority of results falling between 10 and 35 MPa, while the average UCS value was calculated at 30 MPa.

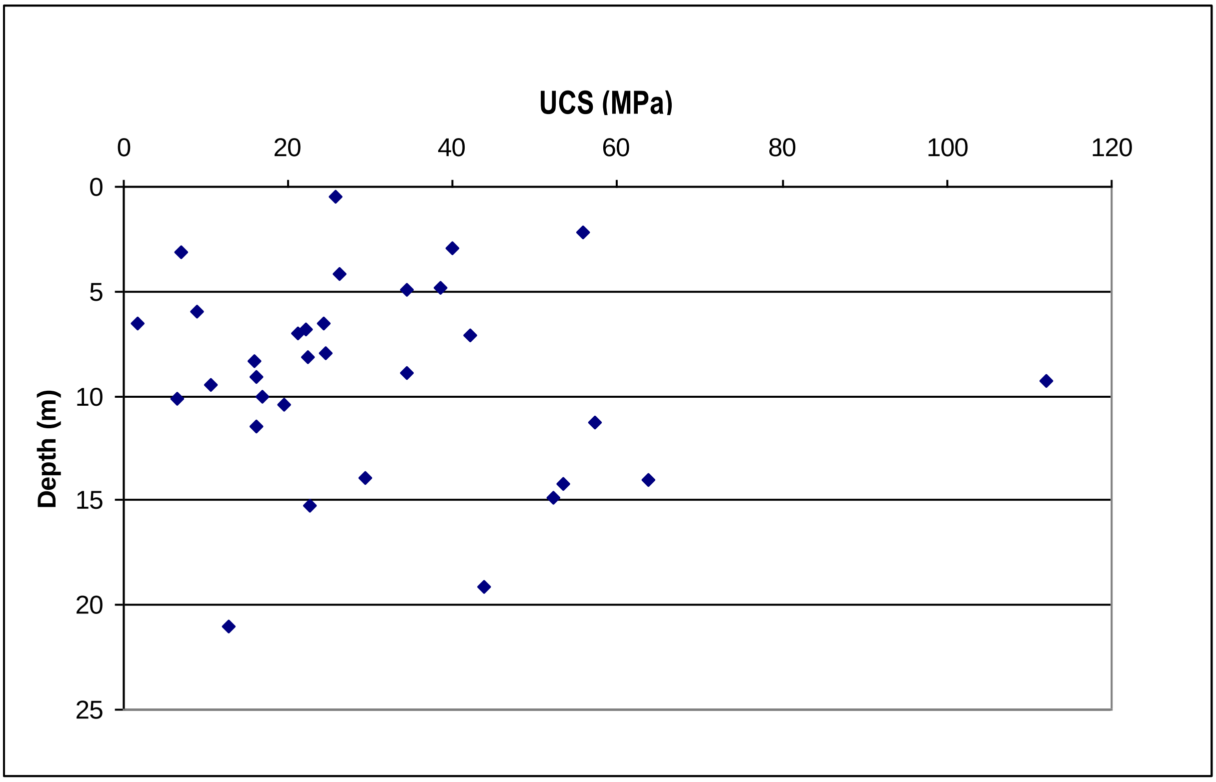

Figure 4.5: UCS results from samples obtained from across the entire REP site

This is a summary table and for detailed and/or individual design parameters for specific scenarios, reference should be made to sections §4.3 - §4.5. These sections also detail the derivation of the parameters.

Strata		γ_b kg/m^3	**c** kPa	**Ø** °	E_m GPa	**n**
Superficial deposits	Cohesive	2.0 – 2.3	c_u = 50.0	0	E_u = 0.012 E' = 0.009	0.1 (drained)
	Granular	2.0 – 2.3	-	34 – 42	E' = 0.031	0.2
Schist	**Weathered**	2660	-	40 - 43	E' = 0.185	0.2
	Non weathered	2660 + 4z	0.5 – 2.5	30	0.2 + 0.1z	0.4

4.3 Made Ground

The following table summarizes for Made Ground:

Parameter	Value
General Strata Profile: Upper Strata Lower Strata	0 mbgl 5 - 8 mbgl
Standard Penetration Test[1] (N)	**25 – 50**
Ø[2]	40 - 43
The deformation modulus of the rock mass[3] (E_m)	Gravel: 0.031 Schist: 0.185
Poisson ratio[4] (n)	0.2
California Bearing Ratio[5] (%)	9.2 - 16
The bulk unit weight[6] (γ_b)	Gravel:2.0 - 2.3 Schist: 2660

Note: The design parameters above are shown, in part, taking into account the parent materials and the host materials that the Made Ground has been derived from.

1 Values obtained from site investigation data directly.
2 Kulhawy and Goodman (1980 an 1987 after Tomlinson 2001)7 have shown that c and Ø parameters can be related to the RQD. Tomlinson Foundation Design and Construction, 7th Edition.
3 Em = j.E: The deformation modulus of the rock mass based on Tomlinson Foundation Design and Construction, 7th Edition.
4 Values obtained from site investigation data directly.
5 Values obtained from site investigation data directly.
6 Values obtained from site investigation data directly.

4.4 Superficial deposits

The following table summarizes for Superficial deposits:

Parameter	Value
General Strata Profile: Upper Strata Lower Strata	 1 – 5 mbgl 5 - 15 mbgl
Standard Penetration Test[7] (N)	8 – 37
Natural Moisture Content [8](%)	28 - 113
Liquid Limit[9] (%)	44 - 128
Plastic Limit[10](%)	30 - 66
Plasticity Index[11]	6 - 62
California Bearing Ratio[12] (%)	9.2 - 16
The cohesion component of shear strength [13]*(KPa)*	50
The friction angle (Ø)[14]	$34^{o} – 42^{o}$
Permeability[15] (m/s)	10^{-6}

7 Values obtained from site investigation data directly.
8Values obtained from site investigation data directly.
9 Values obtained from site investigation data directly
10 Values obtained from site investigation data directly.
11 Values obtained from site investigation data directly.
12 Values obtained from site investigation data directly.
13 Kulhawy and Goodman (1980 an 1987 after Tomlinson 2001)7 have shown that c and Ø parameters can be related to the RQD. Tomlinson Foundation Design and Construction, 7th Edition
14 Kulhawy and Goodman (1980 an 1987 after Tomlinson 2001)7 have shown that c and Ø parameters can be related to the RQD. Tomlinson Foundation Design and Construction, 7th Edition.
15 Values obtained from site investigation data directly

4.5 Schist

The following table summarizes for Schist:

Parameter	Value
General Strata Profile: Upper Strata	1 – 5 mbgl
Standard Penetration Test[16] (N)	From 50 to larger 100
Natural Moisture Content [17](%)	0.1 – 0.7
Rock Quality Designation RQD[18] (%)	15 - 30
Point Load[19] (MPa)	0.1 – 0.8
Unconfined Compression Test [20](UCS)	10 – 35
The deformation modulus of the rock mass [21](E_m)	0.2 + 0.1z
The cohesion component of shear strength [22](KPa)	0.5 – 2.5
The friction angle [23](Ø)	30 º / Weather Schist: 40 º - 43 º
Poisson ratio[24] (n)	0.3 – 0.6
The bulk unit weight[25] (γ_b)	2660 + 4z (z = depth below surface in meters)
Young's module of intact rock [26](E_{intact})	1 - 15
The ultimate bearing capacity of the rock mass[27] (q_c)	1.65 – 8.25

16 Values obtained from site investigation data directly.
17 Values obtained from site investigation data directly.
18 Values obtained from site investigation data directly.
19 Values obtained from site investigation data directly.
20 Values obtained from site investigation data directly.]
21 Em = j.E: The deformation modulus of the rock mass based on Tomlinson Foundation Design and Construction, 7th Edition.
22 Kulhawy and Goodman (1980 an 1987 after Tomlinson 2001)7 have shown that c and Ø parameters can be related to the RQD. Tomlinson Foundation Design and Construction, 7th Edition
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24 Values obtained from site investigation data directly.
25 Values obtained from site investigation data directly.
26 Em = j.E The deformation modulus of the rock mass based on Tomlinson Foundation Design and Construction, 7th Edition.
27 Kulhawy and Goodman (1980 an 1987 after Tomlinson 2001)suggested the relationships qc is the ultimate bearing capacity of the rock mass and quc is the unconfined compressive strength of an intact specimen. Tomlinson Foundation Design and Construction, 7th Edition

5. Discussion and Suggestions

5.1 Discussion

The concentration of boreholes in the Northeast region of the Anglesey Aluminium REP project has left us with limited geotechnical information for the rest of the site, posing challenges to the overall geotechnical understanding.

Despite a scarcity of geotechnical information, two key reports, the "Report on a Ground Investigation at Anglesey Aluminium REP, Holyhead (Volume One)" by the Soil Engineering company and the "Anglesey Aluminium REP Ground Investigation Report" by Mott MacDonald, provide essential geotechnical insights.

One significant concern is that geotechnical data is predominantly confined to the Northeast area. Consequently, we must extract and compile geotechnical data from the available boreholes, which contain valuable information on each soil layer.

The absence of coordinates and coordinate systems in the LK reports for some boreholes is another challenge. This omission makes it difficult to integrate these boreholes into the geological model, impacting the completeness and accuracy of our geological analysis.

In summary, the concentration of boreholes in one region and the lack of coordinate information in certain reports are the main challenges affecting our geotechnical understanding. Addressing these issues is essential for a more comprehensive and reliable geological model.

5.2 Suggestions

To address the geotechnical challenges at the Anglesey Aluminium REP project, the following steps are recommended.

Firstly, there are a number of reports which do not have 3D co-ordinated data associated with them. Therefore, it is recommended that these boreholes are overlain on the site plan and 3D data be attached to them manually. While a laborious process this will add invaluable information to the overall ground model and mean that the boreholes can be taken into account. This will mean that a positive return on investment can be gained for these holes over/above them being undertaken again. After this stage it is recommended that the ground model be updated and re-evaluated.

Lastly, if required, then further intrusive locations can be undertaken. These could include thorough geotechnical tests, such as SPT, vane shear, CBR, consolidation, and classification tests. Furthermore, if required, install piezometers for groundwater monitoring. Incorporate environmental data where relevant and address site-specific exceptions. Finally, consider additional data collection in areas with limited information. These measures will enhance our understanding and support safe and efficient project design and construction.

Appendix D – Technical Note – Soakaways

Technical Note – Soakaways

To:	Anglesey Land Holdings Ltd
From:	Robert Hutchison - HBGS
CC:	
Date:	09 September 2024
Re:	Prosperity Parc, Holyhead – Soakaways

1.0 INTRODUCTION

This Technical Note has been prepared to make an assessment of the potential to utilise soakaways for the site drainage of surface water run-off at the proposed Prosperity Parc site near Holyhead on the Isle of Anglesey.

The site is located some 2kms to the south-east of Holyhead town centre directly to the north of the A55 North Wales Expressway. The A5 trunk road lies to the north and north-east of the site and Holyhead Retail Park lies to the west. The site formerly comprised Anglesey Aluminium Metal (AAM) Works, a large brownfield site which operated as an aluminium smelter from 1970 to 2009.

Since closure, the site has undergone extensive intrusive investigations to characterise both ground conditions and contamination ahead of potential redevelopment.

2.0 PROPOSED DEVELOPMENT

The proposed development comprises up to 238,000 sqm of employment development with date centre (B8), office, and research and develoment (B1) uses. The Illustrative Masterplan (**Figure 1**) is provided as an indicated for how the site could be brought forward but the detailed design will be through reserved matters.

The external areas of the site adjacent to the proposed structures are to comprise mainly asphalt or concrete surfaced hardstanding with on-plot landscaped areas. The margins of the site to the north-west and north-east of the development areas are proposed to comprise 'Retained and Enhanced Green Infrastructure' as shown on the Draft Parameters Plan presented as **Figure 2**.

3.0 GEOLOGICAL SETTING

3.1 Sources of Information

The following sources of information have been consulted as part of the limited desk study research undertaken for the current Technical Note:

- British Geological Survey (BGS) maps and publications for the site including 1:50,000 Series Anglesey Sheets 92 & 93 and parts of sheets 94,105 & 106. [Special Sheet] (Solid & Drift Editions, dated 1974 and 1980 respectively).
- British Geological Survey – www.bgs.ac.uk.
- Natural Resources Wales – www.naturalresourceswales.gov.uk.
- HBGS - Former Anglesey Aluminium Plant Geotechnical Remediation Strategy Volume 1 - Historical Review and Summary Report/002, dated 24 October 2023.
- HBGS - Geotechnical Remediation Strategy Technical Note: Geotechnical Ground Model and Design Parameters, dated 10 November 2023. Report reference, JMS/NMG/45184-Rp-001.

3.2 Geology

Artificial / Made Ground

The HBGS reports referenced above indicate most of the site to be underlain by Made Ground (Anthropogenic Ground) related to previous demolition and construction activities. The composition and properties of the Made Ground are highly variable and may be summarised as heterogeneous fill material comprising sandy clay, gravel, ash, metal, concrete, refractory brick, wood, plastic and other waste with frequent anthropogenic inclusions. The thickness of the Made Grund materials varies across site from 1m to 5m, being generally thicker around the margins of the site where soakaways would be more likely to be sited.

Superficial Deposits

The available geological mapping and the previous reports referenced above indicate the Made Ground to be underlain over the majority of the site Glacial Till deposits (Diamicton – formerly termed Boulder Clay) of Devensian age which are described as consisting mainly of 'clay and silty clay with rock fragments'. Limited areas in the centre of the site may be underlain by Glaciofluvial Deposits consisting of sand and gravel or Tidal Flat Deposits comprising of clay and silt. The thickness of the Superficial deposits ranges between 1m and 15m, being thinner or absent in developed areas where excavation occurred historically and where future development is proposed.

Bedrock

The Made Ground and Superficial deposits are indicated to be underlain by bedrock strata of the New Harbour Group part of the Mona Complex Precambrian to Cambrian age. The New Harbour Group bedrock is described as micaceous schist with narrow quartz banding comprising of grey to bluish grey, medium grained metamorphic rock. The rock may be highly fractured near surface but fracture frequency reduces with depth.

3.3 Hydrogeology

The Superficial deposits are predominantly classified as Unproductive Strata with small areas in the north-west of the site being classified as a Secondary A aquifer. The bedrock is classified as a Secondary B Aquifer.

Secondary A Aquifers are described as 'permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers.

Secondary B Aquifers are described as 'predominantly lower permeability layers which may store and yield limited amounts of groundwater due to localised features such as fissures, thin permeable horizons and weathering. These are generally the water-bearing parts of the former non-aquifers'.

3.4 Hydrology

Surface Waters – onsite and adjacent drains. The Irish Sea is located 100m to the north and 400m to the south of the site.

Flood Risk - the development site lies mainly within a Flood Zone 1 and therefore has a Low probability of flooding. The north-west of the site lies within a Flood Zone 3 (flooding from rivers or sea without defences).

3.5 Mining

Coal Mining

The site does not lie within an area requiring the commission of a Coal Mining Report in accordance with The Coal Authority and The Law Society publication 'Coal Mining and Brine Subsidence Claim Searches – Directory and Guidance', Sixth Edition (2006).

Reference to the Coal Authority Interactive map viewer does not identify the site as lying within a 'Development High Risk' area.

Non-Coal Mining

The site does not lie within a brine affected area.

4.0 PREVIOUS INTRUSIVE INVESTIGATIONS

A number of ground investigation reports have been undertaken on the site and the current Technical Note focusses on the information most relevant to the potential for the disposal of surface water and/or rainwater run-off via soakaway installations.

4.1 Ground Conditions

The extensive previous works at the site included significant numbers of window sampler, cable percussion and rotary boreholes and trial pits as shown in Figure 3.1 of the HBGS Geotechnical Remediation Strategy. The shallow boreholes (window sample or cable percussive boreholes) generally refused on dense weathered schist or the underlying schist bedrock. The ground conditions beneath the site generally comprised of Made Ground underlain by Glacial Till consisting of superficial sandy clay and/or clayey gravel with localised bands of sands and gravels. The Superficial Deposits were predominately underlain by weathered schist which in many locations was recovered as clayey gravels or cobbles of mica schist.

The Made Ground included frequent anthropogenic inclusions such as pottery, ash, metal and wood in most locations and visual and olfactory evidence of hydrocarbon contamination was noted at many locations. Subsequent laboratory has identified elevated concentrations of PAHs, pH, PCBs, metals and TPHs. Asbestos containing materials (ACMs were also identified.

The Superficial deposits comprised variable sequences of sands, clays, gravels and cobbles. These were occasionally absent in previously developed areas and extended to a maximum depth of 15m.

The schist bedrock was highly micaceous, and the shallow bedrock was predominately found to be highly fractured. The deeper schist bedrock showed a lesser degree of fracturing with predominately sub-horizontal fracturing noted.

Typical examples of the Made Ground are shown on the following Plates (nos. 1, 2, 3 & 4) below.

Plate 1

Plate 2

Plate 3

Plate 4

Typical examples of the Made Ground overlying Glacial Till deposits are shown on the following Plates (nos. 5 & 6) below.

Plate 5

Plate 6

A typical example of the Glacial Till is shown on Plate no. 7 below.

Plate 7

Typical examples of the Schist bedrock present near surface are shown on Plates (nos. 8 & 9) below.

Plate 8

Plate 9

4.2 Groundwater

Groundwater strikes were recorded during the previous investigations in both trial pits and boreholes and within the Made Ground, Superficial deposits and the underlying Schist bedrock. Groundwater was recorded at varying depths across the site, ranging from just below ground level to over 12m depth.

Frequent localised shallow groundwater pockets were present within the superficial geology, probably due to the presence of low permeability layers and standing water was observed at multiple locations during the site investigations.

Some hydraulic connections were evident between neighbouring boreholes up to 20m apart during drilling and testing. Steady state water depths recorded mainly in trial pits 20 minutes post-strike were generally at depths between ground level and 3m. Groundwater levels also exhibited tidal fluctuations in marshy low-lying areas in the western parts of the site.

The HBGS Geotechnical Remediation Strategy has assigned a permeability of 10^{-6} m/s to the Superficial deposits.

The results of contamination testing of groundwater samples identified limited contamination by metals, fluoride, PCBs and petroleum hydrocarbons, with occasional free product also being identified.

A typical example of the standing surface water is shown Plate no. 10 below.

Plate 10

Typical examples of shallow groundwater ingress into trial pits from the Made Ground are shown on the following Plates (nos. 11, 12 & 13) below.

Plate 11


Plate 12


Plate 13

A typical example of the rapid groundwater ingress from the Made Ground into a trial pit is shown on Plate no. 14 below.


Plate 14

A typical example of the groundwater ingress from the Glacial Till / Schist bedrock interface is shown on Plate no. 15 below.

Plate 15

A typical example of the groundwater ingress from the near surface Schist bedrock is shown on Plate no. 16 below.

Plate 16

5.0 DISCUSSION

BRE Digest 365 'Soakaway Design' states that soakaways have been a traditional way to dispose of stormwater from buildings and paved areas remote from a public sewer or watercourse and in recent years, soakaways have been used increasingly within urban, fully-sewered areas to limit the impact on discharge of new upstream building works and to avoid costs of sewer up-grading outside the curtilage of a particular development.

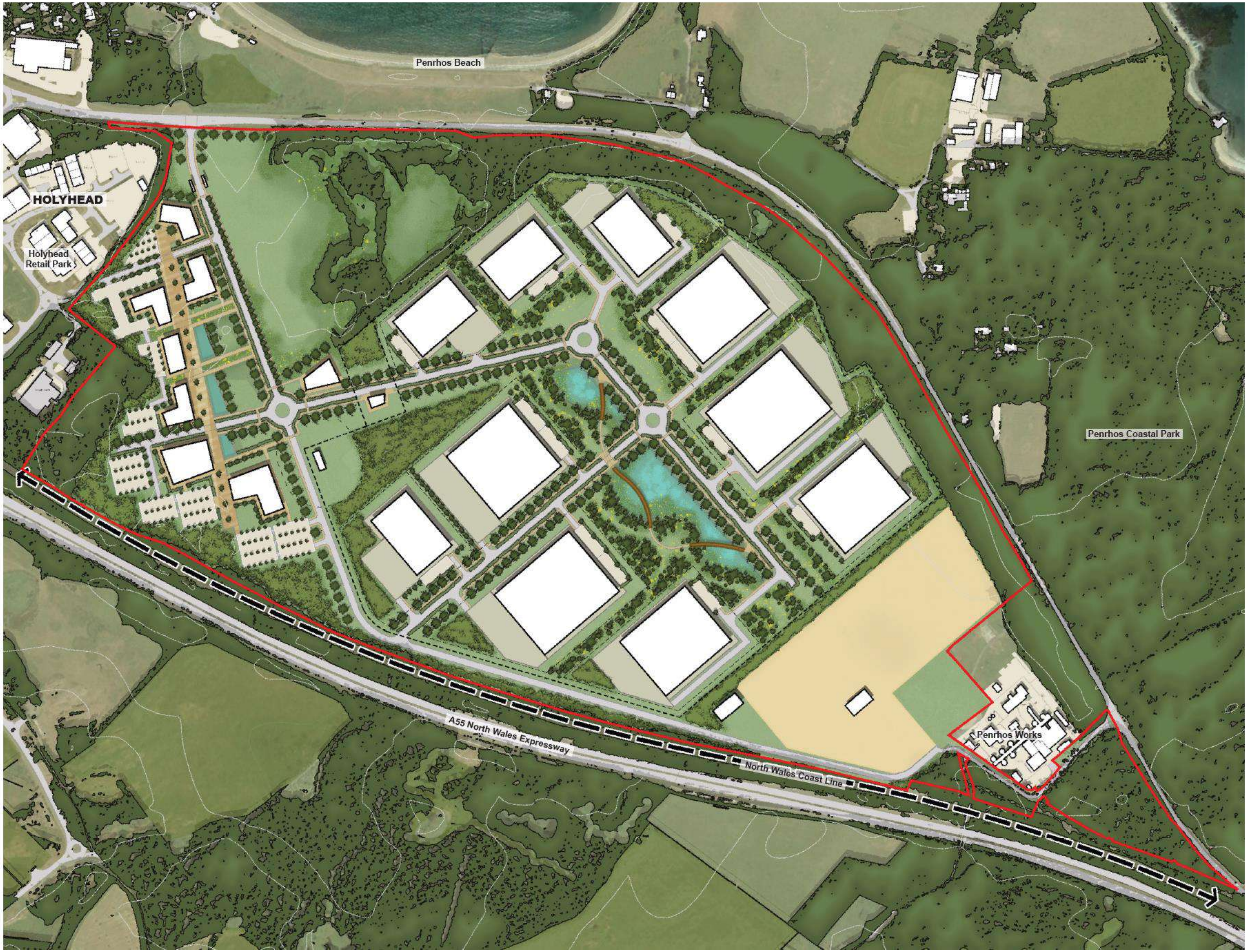
Both BRE Digest 365 and Environment Agency guidance preclude the installation of soakaways within Made Ground and Environment Agency guidance goes on to state that 'soakaways should also not be built in contaminated soil'. Although in the case of Prosperity Parc, the concentration of trial pits in particular in the north-eastern region of the proposed development site has left us with only limited information on ground conditions for the remainder of the site, the available information, mainly from boreholes, indicates ground conditions, as described in **Sections 3.0** and **4.0** (above), to be similar across the whole site.

Therefore, given the current presence of 1m to 5m of Made Ground across the site and the likely requirement to raise site levels to achieve final finished floor levels, most of the site would appear to be unsuitable for soakaways on this basis alone.

Furthermore, Environment Agency guidance states that soakaways cannot be installed ‘where the water table is less than 1 metre below the base of the soakaway’. This would apply whether the water table is perched or represents the local groundwater table. Steady state water depths were generally recorded at depths of between ground level and 3m, the likely depth range over which soakaways would be installed. Groundwater levels also exhibited tidal fluctuations which, given the current site ground levels of the order of between 7m and 8m above OD, potentially could lead to water ingress into soakaways at times of unusually high tides. Both of these issues further render soakaways inappropriate at the Prosperity Parc site.

In conclusion, HBGS consider that soakaway installation at the Prosperity Parc site is not viable and would recommend, where possible, disposal of surface water run-off to local watercourses and/or re-use of the existing site drainage system which is understood to discharge directly to Holyhead Bay.

Figure 1
Illustrative Masterplan



KEY

Application Site Boundary

Existing electric cables / substation to be retained and extended. BESS scheme to be developed in this area

POTENTIAL DEVELOPMENT

- Potential built development
- Potential loading yards & HGV parking
- Potential car parking
- Indicative primary route with pedestrian / cycle links
- Indicative emergency access road
- Potential electricity substations
- Potential Gatehouse for Main Site
- Potential Gatehouse / controlled access point for Data Centre
- Existing Tunnel Access building with 50m buffer zone
- Data Centre Campus fence lines Double fence line - 8m apart

GREEN INFRASTRUCTURE

- Indicative areas of existing vegetation
- Areas of existing vegetation to be retained - Establised landscape buffers
- Potential landscape buffers to reinforce existing vegetation
- Potential individual tree and hedgerow planting
- Potential SuDS Features including areas wetland habitat / wildflower grassland
- Potential Green Corridors - Wildlife corridors between plots

Figure 2

Draft Parameters Plan

Application Site Boundary:
87.92ha / 217.25ac

Development Zones / Built Infrastructure:
66.20ha / 163.58ac

Will contain on-plot and other landscaping and planting, habitat enhancement and creation, drainage and other infrastructure including vehicular, cycle and walking access

Up to 238,000 sqm Class B1 and B8 (data centres only), plus battery energy storage (unique use);

Finished Floor Levels: Similar to existing ground levels of approximately 5 to 10m AOD.

Retained & Enhanced Green Infrastructure:
21.72ha / 53.67ac

Retained existing access from the A5

Secondary / emergency site access (existing)

Railway site access (existing)

Indicative areas of Tree Preservation Orders (TPO)

MoD / RAF Consultation Zones on Heights

Building Heights

Zone A: Max height up to 18m to ridge excluding point features

Zone B: Max height up to 21m to ridge excluding point features