Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-1715

Foundation design to minimise construction & programme risks: Collaborating to deal with soft soils, variable deep rockhead & Coal

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ABSTRACT: Cementation Skanska Limited was responsible for the design and installation of 1432 bored piles to support a new energy infrastructure project in Glasgow, inclusive of a 12m deep bunker structure in soft Alluvium up to 14m thick. Bearing pile lengths extended up to 32m, socketed into the Limestone Coal Formation. Moreover, irregular rock head level, composition, and strength, posed significant challenges for pile design and validation. Commercial pressures required early collaboration between consulting engineers RPS and specialist piling contractor Cementation Skanska Limited at concept design stage. By combining RPS's design remit with Cementation Skanska Limited's proven expertise in complex ground conditions, the team was able to identify and mitigate construction and geotechnical risks from the outset. This close collaboration steered RPS away from less viable designs, keeping the project focused on buildable, cost-effective solutions. The deep combi-wall basement served as a clear example of this approach in practice. Instrumented sacrificial pile load tests were installed ahead of the main works, providing validation of the proposed piling solution before full implementation. The ground model was simplified to manage complexity, and a design validation matrix was implemented to reduce risks to site operations, providing a feasible and timely solution for the client. Piles were designed to a specified toe level, but during construction, engineers closely monitored and characterised the ground. If piles met refusal, the design validation matrix allowed site operatives to assess the as-built pile resistance in real-time. As more piles were constructed, a rockhead level map was developed to scrutinize site findings and enable engineers to predict rockhead levels and composition. The benefits included a reduction in overall pile lengths, concrete volume, and the programme duration.

KEYWORDS: Limestone Coal Formation, combi-wall, Alluvium, design validation, sacrificial test piles, risk management

1 INTRODUCTION

The South Clyde Energy Centre (SCEC) in North Cardonald, Glasgow, is being developed to support Scotland's 2025 ban on landfilling biodegradable municipal waste. The facility will process up to 350,000 tonnes of non-recyclable waste annually, generating 45 MW of low-carbon electricity, enough to power 70,000 homes. The site of the facility is shown in Figure 1, during piling operations.

The main structures, including the tipping hall, boiler room, and flue gas treatment area, will be supported on 600–900 mm diameter continuous flight auger (CFA) and rotary bored piles (RBP). The waste bunker comprises a 23 m × 32 m × 12 m deep excavation constructed using a combi-wall with 1350 mm bored piles inside 1462 mm circular hollow sections (CHS), with sheet pile infill. Temporary tower cranes were supported on permanent works bearing piles.

Cementation Skanska Limited (CSL) were approached with an invitation to provide initial budget costs in November 2022 prior to commencement of detailed design for the piling works. The client (Fortum) already had a commitment to commission the plant by the end of 2024 and this meant they needed to select a delivery partner for the piling works early, as their programme would not accommodate development of the detailed design and then a traditional tender and procurement route. The principal contractor, Afry, facilitated discussions allowing CSL to be selected as a delivery partner and integration into the consulting engineers' (RPS) design team early in the process. This helped mitigate design and operational risk and shortened the design and lead-in programme.

Figure 1. SCEC construction site during piling operations.

2 GROUND MODEL

2.1 *Geology*

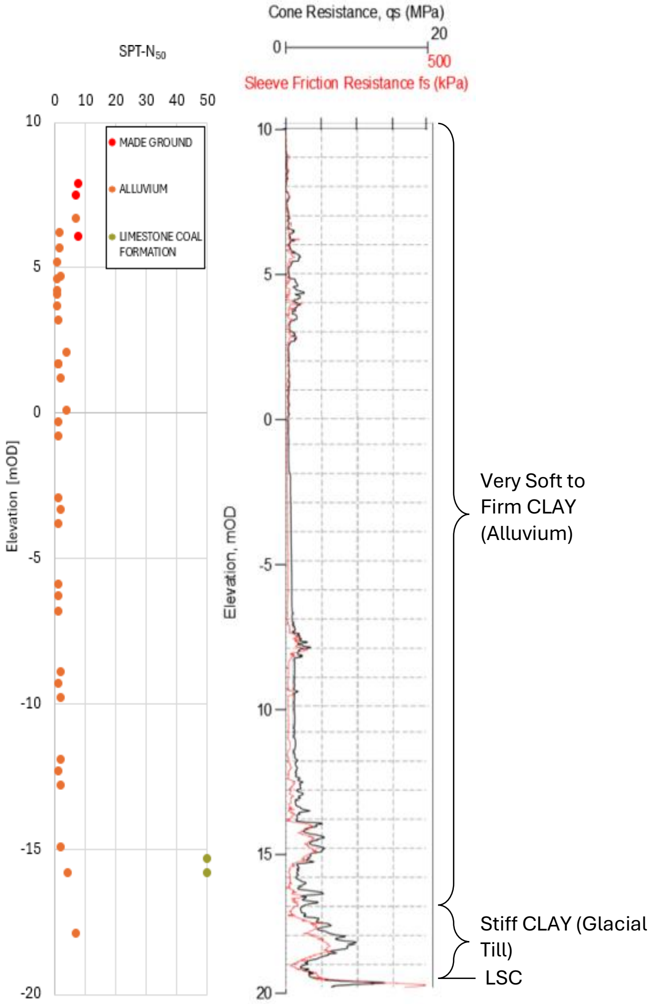
The site is underlain by bedrock of the Namurian-age Limestone Coal Formation (LSC) within the Clackmannan Group recording transitions from mixed carbonate-deltaic to fluviodeltaic environments driven by repeated marine transgressions and deltaic progradation. The LSC is defined by upward-coarsening cycles of sandstone, siltstone, and mudstone, typically capped by seatearth and coal. Post-depositional tectonic activity has resulted in folding and faulting across the region.

Overlying the LSC is a complex succession of Quaternary superficial deposits comprising Glacial Till. These are overlain by further fluvial deposits, recent Alluvium, and anthropogenic fill, producing a heterogeneous ground profile of very soft soils above variably composed, undulating and weathered bedrock.

2.2 *Site Investigation*

Several intrusive site investigations were conducted across the site. Exploratory boreholes identified a thin layer of made ground overlying fluvial and alluvial deposits, generally comprising very soft to soft brownish grey slightly sandy clay, with Standard Penetration Tests (SPT) N_{50} values ranging from 1 to 7 to depths of up to 22.0m (Figure 2a). Evidence of soft ground was further supported by subsequent Cone Penetration Tests (CPTs), which revealed low cone and sleeve resistances (Figure 2b). The CPTs also indicated that the alluvial material consisted of mixed particle sizes, necessitating evaluation under both undrained and drained conditions.

In some locations, Glacial Till was encountered beneath these deposits, typically described as firm to very stiff slightly sandy slightly gravelly clay. This was underlain by bedrock of the LSC, consisting of extremely weak to moderately weak mudstone or weak to strong sandstone with interbedded coal seams. Rockhead composition varied spatially across the site.



a) b)

Figure 2. a) Site-wide SPT N50 vs. elevation chart, b) E.g. CPT vs. Elevation chart.

2.3 *Ground Risk*

The ground risks associated with a piled foundation and temporary works are summarised in Figure 3, below:

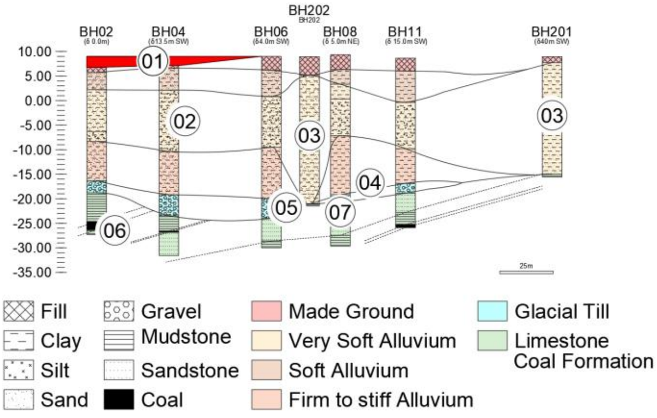


Figure 3. Simplified ground model of SCEC identifying ground risk elements.

- 01 Negative shaft friction (NSF): upfilling of approximately 3 m may consolidate softer ground, causing downdrag and increasing the risk of pile overloading and settlement. The magnitude of NSF was assessed using methods in ICE MOGE, Burland et al. 2012.
- 02 Bore collapse/ no resistance: very soft Alluvium (Figure 4) may lead to bore collapse and no resistance. This limits the practical length of continuous flight auger (CFA) piles as the risk of instability increases if drilling on the Kelly bar extension is required for pile diameters above 600mm.
- 03 Contradicting borehole data: Some boreholes characterise all Alluvium as very soft and do not identify Glacial Till.
- 04 Inconsistent ground conditions: Glacial Till not always present.
- 05 Inconsistent rock level and composition: 5m variation in rockhead.
- 06 Coal: up to 3.0m thick unworked coal seam identified within the expected rock socket depths.
- 07 Strong Rock: Medium strong Sandstone identified making drilling conditions and duration more difficult and longer than expected, with potential risk of refusal above required pile toe levels.

Figure 4. Alluvial arisings from rotary bored piling rig.

2.4 *Design Model*

To mitigate the geotechnical risk, whilst accelerating the design process, CSL simplified the site into five ground model areas (Figure 5). Conservative assumptions were adopted, including selecting the deepest recorded rockhead depths from interpolated borehole data within each area. This approach allowed the piles to be designed to the minimum required length, avoiding reliance on a fixed rock socket length and the need for on-site decisions during installation which may have slowed down production.

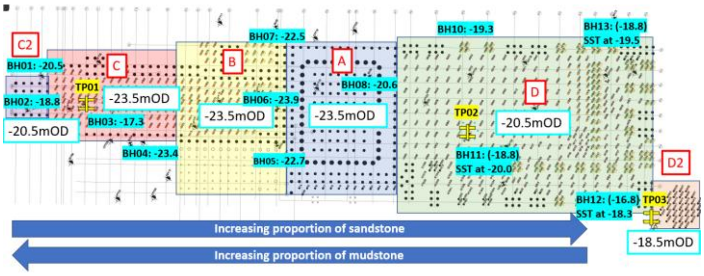


Figure 5. Simplified plan of ground model areas, design rockhead levels and test pile locations (all levels are mOD). Piles installed from a level of +8.7mOD.

Geotechnical design parameters were based on current site investigation data and in-house historical data (Table 1).

Table 1. Geotechnical pile design parameters.

Strata	**Internal Angle of Friction, ϕ'**	**Undrained cohesion, c_u**	**Ultimate Shaft Friction, q_s**	**Ultimate End Bearing, q_b**
	[°]	**[kPa]**	**[kPa]**	**[kPa]**
Made Ground	30	-	0	0
Alluvium – Upper & Middle	21-24	30-40	30 - 40	0
Alluvium – Lower	24	60	40	0
Glacial Till	32	150	75	0
Mudstone	32	300	150	1,000 - 3,000
Sandstone	-	-	275	12,500

CSL specified that no resistance was to be derived from coal seams and that piles were to be founded beneath any identified seams.

The groundwater regime was hydrostatic, with flow occurring through both the bedrock and superficial deposits. A groundwater level of 1.0 m below ground level was adopted based on monitoring data.

3 DESIGN DEVELOPMENT

Due to the extensive list of potential ground risks across the site, and as CSL were integrated early into the design team, CSL engaged with the consultant engineer (RPS) to help define an optimum and buildable foundation scheme. CSL produced technical notes throughout the design development phase to advise project stakeholders and allow informed decisions to be made, notably, advising on ranges for pile spring stiffnesses for incorporation into the structural model allowing derivation of pile loads and informing the final pile layout. There was a considerable focus required on the development of the retaining wall solution for the bunker, as this structure was integral to the foundation scheme with various slabs structurally connected to the bunker capping beam and pile caps in close proximity to the retaining wall, and had the potential to have the greatest programme impact on the project.

3.1 *Retaining Wall*

The initial proposal for the retaining wall was a reinforced concrete secant piled wall. However, due to the soft ground conditions and the requirement for no propping in the permanent condition within the bunker space, the induced bending moments of up to 16MNm per pile exceeded the capacity achievable with conventional reinforced concrete piles. To avoid complex pile reinforcement, CSL proposed a steel combi-wall which would achieve greater stiffnesses and increased structural capacity. However, it was recognized early on that the up to 20-week lead in time for the CHS elements of the combi wall would require accelerated concept and detailed design to ensure that the programmed critical path was not dependent on delivery of the CHS tubes to site. This was achieved successfully with all stakeholders informed and decisions based on reviewed risks, notably, with due consideration given to the specified loads potentially increasing.

CSL worked collaboratively with RPS, discussing options and their own design constraints. CSL were then able to fully propose the alternative combi-wall through a technical note demonstrating the assumed construction sequence, including assumptions regarding prop system stiffnesses, capping beam stiffnesses and long-term loading from the base slab. This allowed any issues to be resolved early in the design phase prior to completion of the detailed design work and commencement of on-site operations.

The foundations around the bunker consisted of slabs and caps supported on bearing piles structurally connected to the proposed retaining wall capping beam. An iterative design process was therefore required to finalise the design proposals for the bunker as this directly impacted the adjacent piles and foundation solution. For example, the capping beam was required to act as a high-level prop which induced increased shear forces in the piles supporting the beam. These piles were also located within the active wedge of the retaining wall and therefore required longer reinforcement to resist increased bending moments requiring consideration of the best construction method and diameter for these piles.

Axial loads in excess of 8MN were specified on the structural elements of the combi wall, necessitating longer rock sockets than considered elsewhere on the project. This required CSL to assess the feasibility of installing these longer sockets more than 5m into potentially medium strong to strong sandstone. To ensure that this was achievable, CSL confirmed that they had the appropriate tooling available to achieve the design requirements, in the form of rock rollers.

The final design of the combi-wall was undertaken in Geosolve Wallap and is schematically shown in Figure 6. It required two levels of additional temporary propping, illustrated in Figure 7. The wall is propped in the permanent condition by the 1.5m deep capping beam and the 1.6m thick base slab, allowing the full bunker space to be utilised without intermediate propping. Pairs of ZZ28-700 sheet piles were utilised between the CHS piles.

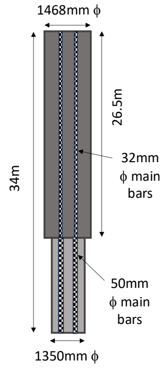


Figure 6. Combi-wall pile design.

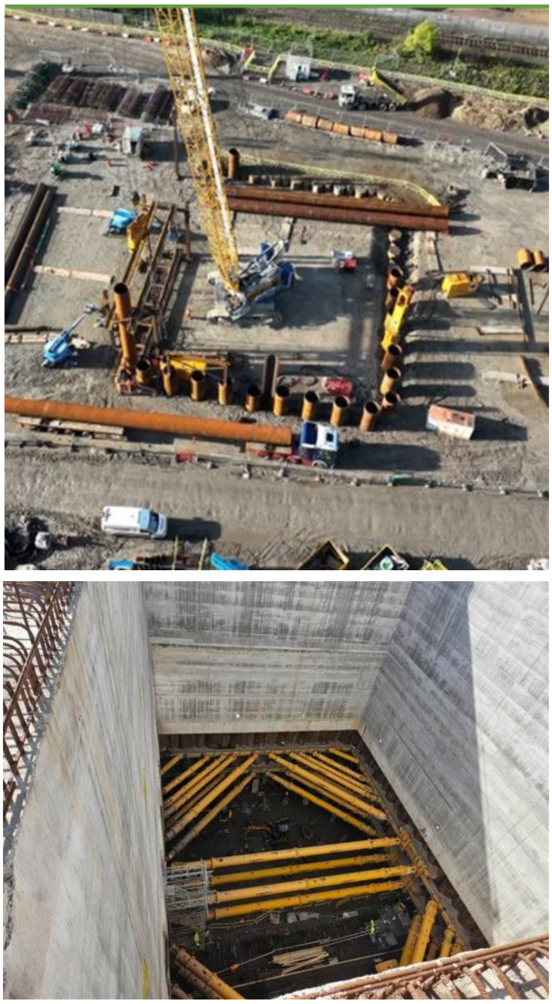


Figure 7. Combi-wall installation & temporary propping.

3.2 *Bearing Piles*

The design development process for the bearing piles involved highlighting the benefits as well as the drawbacks of CFA piles versus rotary bored piles, summarised in Table 2. The pile design was undertaken to the Eurocode requirements of BS EN 1992-1-1:2004 & BS EN 1997-1:2004.

Table 2. Comparison between CFA & RBP.

CFA	RBP
Quicker - ~8no. per day per rig therefore cheaper.	Slower - ~1no. per day per rig and require support fluid and/or long temporary casing ~3.5x cost of CFA pile
Higher risk of refusal on strong rock	More options for progressing through stronger rock with alternative digging tools
Practical cage length limited by ability to plunge after concreting therefore limits tension and shear capacity – if cage exceeds 12m, central bar also specified up to 26m.	Cages are installed prior to concreting so tension & shear capacity not limited by cage installation
Identification of rockhead level difficult	Identification of rockhead level potentially difficult
Limitation on maximum pile length and pile diameter – 600mm & 750mm diameter installed, up to 35m long. Max 4m rock socket.	Greater length and diameter achievable – 750mm, 900mm & 1350mm diameter installed, up to 38m long. Max 5m design rock socket.
Must ensure careful installation, with controlled auger extraction and adequate concrete supply to maintain bore stability and review of concreting records after completion.	Using temporary or permanent casing in soft or loose ground reduces the risk of bore collapse.
Factor of safety >1.2 on ultimate shaft resistance required to mitigate uncertain end bearing conditions.	Increased certainty on end bearing conditions means reduced length of piles.
Considering the above	
Maximum serviceability loads: Compression: 1000 to 1300kN Tension: 200kN	Maximum serviceability loads: Compression: 1400 to 4700kN Tension: 700 to 1900kN

CSL outlined the risks, limitations, and capabilities of each piling technique in relation to the site's ground conditions, enabling RPS to adopt the optimum piling layout from the outset. This allowed the scope to be defined quickly and efficiently, thus allowing pile design, approvals and works to be undertake as early as possible, which was a benefit given the time constraints and commitment to the commissioning date.

There was an iterative process between the pile design and the structural model, with pile loads defined based on their modelled spring stiffness. CSL undertook settlement calculations and relayed these back to the consulting engineer to allow them to refine their model. These were later revised following on site pile load tests which recorded lower settlements than originally anticipated leading to alternative pile layouts and reduced number and diameter of piles.

Initially, CSL limited the maximum CFA pile diameter to 600mm due to bore instability through the soft Alluvium. In the spirit of continued collaboration CSL challenged its own decision by undergoing a 750mm CFA trial pile to determine its feasibility. After a successful trial, CSL then analysed 880/750 RB piles for eligibility for the conversion to 750mm CFA piles. 49no. RBP piles were subsequently switched to CFA saving ~£490,000 for the client.

3.3 *Sacrificial Test Piles*

CSL constructed 3no. CFA sacrificial test piles prior to commencement of the main work (Figure 8).

Figure 8. Sacrificial test pile setup with 6no. anchor piles.

These load tests were undertaken in order to:

- **Validate design parameters**: The majority of the material above rockhead on the project was described as soft to firm cohesive material in the boreholes. Although there was confidence in the design parameters utilised for the bedrock due to historic local experience, there were some questions over the parameters for the soft superficial deposits. This was further evident from the variations in descriptions between the boreholes and the CPTs undertaken at the same locations.

- **Validate design approach/strategy:** Pile installation requirements were presented as a pile length rather than a rock socket requirement. The design philosophy determined an appropriate bedrock level and design calculations were based on this, however CSL's project team were provided with a pile length rather than a minimum rock socket requirement. This was predominantly due to difficulties identifying rockhead at the interface between firm to stiff clay of the overlying superficial deposits and potentially extremely weak mudstone.
- **Confirm pile settlement performance**: The pile layout and loading were governed by a structural model that was heavily reliant on pile spring stiffnesses. Installation of sacrificial test piles meant that pile settlement performance could be reviewed before the project commenced. The specified settlement criterium was 1% of the pile diameter.
- **Early opportunity to test practical reinforcement installation lengths**: Due to tension loads, some of the permanent works piles had long reinforcement requirements. As part of the sacrificial test piles, 30m long 63.5mm diameter GEWI bar was successfully installed in the anchor piles.

All three sacrificial test piles were installed with vibrating wire strain gauges and distributed fiber optic sensors (DFOS) to record strain in real time along the depth of the pile during the pile load tests.

Following installation of the test piles, the installation logs were reviewed to determine whether rockhead could be identified from the rig instrumentation. Figure 9 shows the rotary torque installation logs for the test piles with the stratum descriptions from the adjacent boreholes added. All test piles were located within 1.5m of a borehole. There is no clear transition between the superficial deposits and Mudstone bedrock for TP01. There is a gradual increase in torque through the Glacial Till deposits on test piles TP02 & TP03, but the exact level of the interface into Sandstone is not clear. This confirmed that utilising the CFA installation logs as a design validation tool and relying on the site team to determine rockhead was not possible, therefore providing required pile lengths to the site team was the correct approach.

Following the pile load tests, the strain profile of the piles (Figure 10) showed that the local CPT logs appeared to be a better representation of the ground strength and strata encountered compared to the local rotary boreholes that were undertaken. This is likely due to the soft superficial deposits proving challenging to log and assign engineering descriptions to once they were no longer in-situ.

Review of the measured strain during the pile load tests showed that generally, the upper material which was logged as soft clays in the boreholes or low strength material in the CPT logs had measured shaft friction of between 20kPa and 40kPa, which was in line with the design parameters. However, there was a significant difference between the borehole descriptions, the CPT logs and the measured shaft friction for the lower Alluvium. These were described as firm clay in the boreholes but measured shaft friction suggested a stiff to very stiff material. In this instance the measured shaft friction was higher than the parameters for the design (see Table 3).

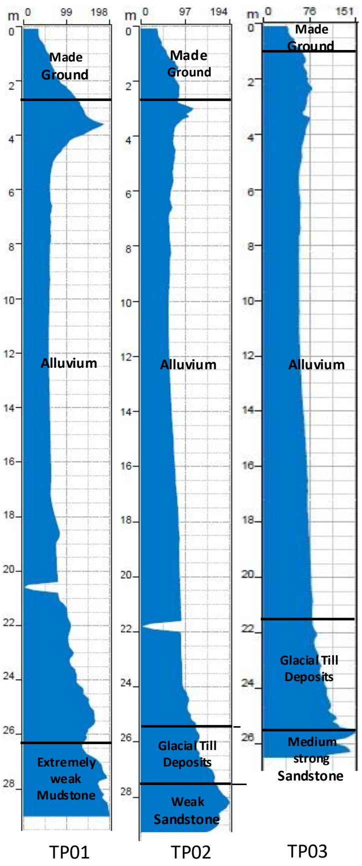


Figure 9. Rotary torque logs for sacrificial test piles.

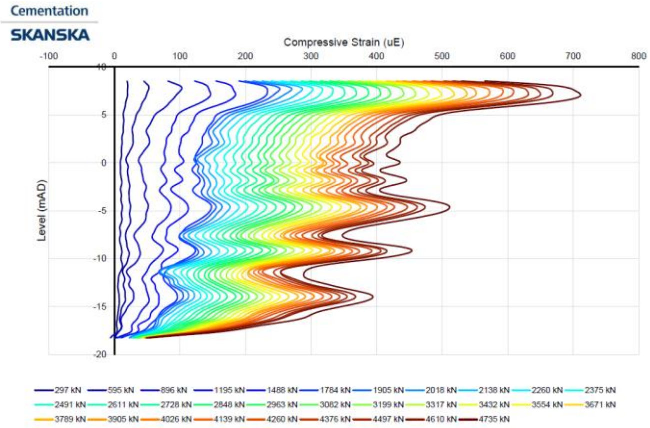


Figure 10. Sacrificial test pile 1 showing measured strain with depth per load step.

Table 3. Back analysed design parameters.

Strata	**Design strength, q_s (kPa)**	**Back analysed, q_s (kPa)**
Alluvium – upper and middle	30 to 40	20 to 40
Alluvium – bottom	40	100 to 140
Glacial Till	75	150

There are some limitations in terms of the instrumentation data from the sacrificial load tests. Piles were not tested to geotechnical "failure" therefore the measured strain in the piles, particularly for the lower section, does not represent ultimate shaft friction values, but the maximum measured values. The piles were also not loaded to a high enough load to mobilise the pile base, therefore there is very limited information on the strain and therefore the load measured at the pile toe. As a result, the design values used in the bedrock could not be compared and the values in the Glacial Till are likely to be higher than the design values based on the measured shaft friction against the settlement of the piles. However, the test piles were able to validate that the upper, softer material on the project broadly confirmed the design parameters taken and the lower Alluvium and Glacial Till had higher shaft friction values than were taken in the design. Due to program constraints and not fully validating all the design parameters taken, the sacrificial test piles were used to validate the design approach and parameters but could not be used to revisit the design and offer efficiency with higher shaft friction values.

In addition to the sacrificial test piles, 11no working test piles were undertaken which were a mixture of 600mm and 750mm diameter. They were loaded to DVL + 50% Frep with maximum test loads between 1,500kN and 3,491kN.

Settlement predictions were undertaken based on Fleming's method (Fleming, 1992). This resulted in anticipated settlements of up to 4mm for 600mm diameter piles and between 4 & 7mm for 750mm diameter piles at serviceability limit state (SLS), which were compliant with the specification requirement of up to 1% of the pile diameter. The results summarized in Table 4 indicate actual settlements of approximately 50% of the predicted values.

Table 4. Summary of test pile results.

Test Pile Reference	**Settlement at DVL**	**Settlement at DVL + 50% Frep**	**Settlement at maximum test load**
TP01 (600 mm)	2.9mm	5.2mm	23.5mm
TP02 (600mm)	2.1mm	3.4mm	14.8mm
TP03 (600mm)	2.1mm	3.5mm	17.0mm
Working test piles (11no 750mm & 600mm)	Between 1.1mm and 3.3mm	Between 2.1mm and 6.6mm	-

3.4 *Design Validation Matrix*

To validate the design in conditions where the piling rig experienced early refusal before achieving the minimum design pile length, CSL developed a rock socket design matrix. CSL engineers monitored the arisings from rotary bored piles logging the rock socket length and composition. The as-built rock socket was evaluated against the ultimate limit state (ULS) and SLS loads and settlement criteria.

During the course of the project, CSL engineers prepared a rockhead level contour map (Figure 11) to predict both the depth and composition of the rockhead. This increase in rockhead certainty enabled the refinement of rockhead level design assumptions, allowing the design pile length of RBP and CFA piles to be reduced. The integration of the design validation matrix with the rockhead map resulted in substantial site efficiencies, including a total drilled length reduction of 2,241 m, a concrete volume saving of 908 m³ (3%), and a fuel saving of 9,000 litres. These measures shortened the rotary piling programme by 5.5 weeks delivering a carbon reduction of 319,571 kgCO₂e and generating £457,116 in piling cost savings, shared between the Client and CSL.

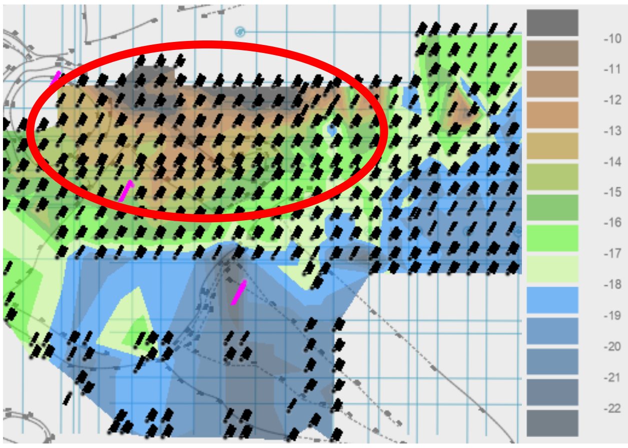


Figure 11. Rockhead-level contour map excerpt, highlighting an area with shallower than expected rockhead by up to 7.3m, in red.

4 CONCLUSIONS

Early contractor involvement allowed collaborative engagement between CSL and the scheme designer. This resulted in early agreement of design principles which in turn expedited the pile design approval process, thus ultimately aiding the overall project programme. It also enabled early identification that the bunker retaining wall required an alternative to the client's initial secant wall solution, i.e. a stiffer solution resulting in the adoption of a steel combi-wall.

The back analysis of the sacrificial test piles indicated that the design parameters within the superficial deposits were appropriate within the upper and middle deposits, but parameters at greater depth could have been increased. These unfortunately could not be adopted within the bearing pile design due to programme constraints.

The design validation matrix proved a useful tool to ensure piling works could progress when refusal occurred on stronger rock and resulted in significant savings, particularly on the project programme.

This project provided excellent opportunities for collaborative working between the client, principal contractor, scheme engineer and CSL to reduce the overall programme duration and to create savings for the client (Section 3). In hindsight, earlier involvement would have enabled further design optimisation rather than design validation, potentially resulting in further programme, cost and carbon savings.

5 ACKNOWLEDGEMENTS

The authors and contributors would like to thank their organisations for their support in preparing this paper, along with the following stakeholders: SPI, RPS, Afry, Fortum.

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