EDINBURGH ST JAMES – BEARING PILE DESIGN

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ABSTRACT Edinburgh St James (ESJ) is one of the largest and most significant regeneration projects currently underway in the United Kingdom (UK). A retail-led, mixed-use development which includes a retail, leisure, hotel and residential offering, ESJ is currently being constructed in the heart of Edinburgh, Scotland. A case study from the Edinburgh St James project is presented detailing the design of the bearing piles on the project. The case study is centred on the pile rock socket shaft resistance and how this was the critical parameter for the pile design. The case study presents the results from a soft-toe pile test and shows how the test can be used to design more efficiently, to the benefit of the project. By undertaking a soft-toe pile test, the project team was able to realise significant improvements in cost, programme and carbon, ultimately creating greater certainty of delivery.

1. Introduction

Edinburgh St James is a 1.7 million ft^2 retail-led, mixed use development in Edinburgh, Scotland, scheduled to open in 2020. Laing O'Rourke (LOR) became the Principal Contractor in October 2016, following a period of enabling works. The piling package was delivered by Expanded Geotechnical (a Laing O'Rourke company).

LOR worked directly with the client Nuveen from 2014 providing early advice to develop a plan for the project that would be deliverable within the required timeframe and budget. The client's consulting engineer was ARUP, novated to LOR after project award.

A case study is presented showing how pile testing can be used to optimise the foundation design in order to reduce cost, time and embodied carbon on a project.

2. Site Location

The site is located close to the junction between Princes Street and Leith Street in the city centre of Edinburgh. The site is bounded by St James' Place and Little King Street to the north, Leith Street to the east, Leith Street and James Craig Walk to the south and James Craig Walk and Elder Street to the west.

The topography of the site is generally sloping down from southwest to northeast. The initial ground levels varied from approximately 70m AOD at James Craig Walk in the south west to 58m AOD on Leith Street in the east/north-east. Level platforms were created within the footprint of the site to enable pile construction.

3. Geology

The basic succession of strata can be summarised as follows:

- Variable Made Ground
- Glacial Till – Very Stiff CLAY with cobbles and boulders in the matrix with very occasional sand lenses
- Variable rock mass consisting of Gullane Formation and Craigleith Sandstone with intermittent Tholeiitic dyke intrusions from the Edinburgh dyke swarm.

The variable mass included mudstone, sandstone, conglomerate with possible igneous intrusions. Igneous intrusions were not encountered during the investigation but were known to be present on the site based on the local geology. In a similar manner to the topography, the rock level also sloped generally in the direction of the ground down Leith St, but at a steeper angle.

4. Pile Design

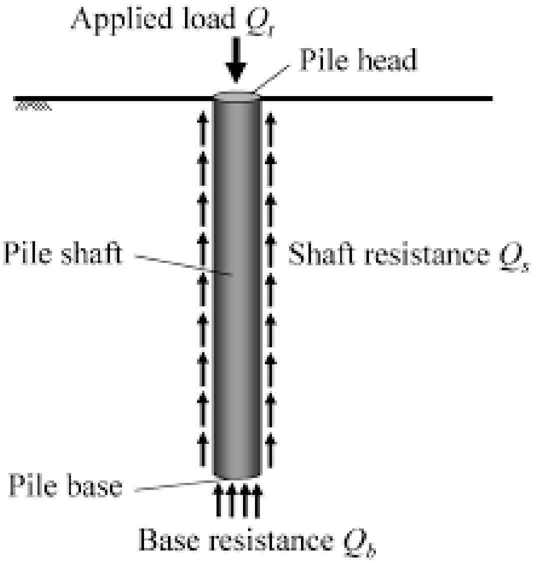
Prior to mobilisation, the major project risk identified for the piling was drill rig refusal on hard rock preventing pile construction. Hard rock drilling was perceived to be a higher risk to the contractor when compared to the risk of poor pile performance due to any unsatisfactory ground. The risk of poor pile performance was perceived to be low given the strength of the Glacial Till and the competence of the rock mass generally. The pile drilling technique and tools were specially designed to handle this complex hard rock drilling environment.

The critical parameter for the bearing pile design was the ultimate rock socket shaft resistance. Given that 70% of the individual task time was devoted to drilling the rock socket, this parameter determined the overall pile length and the piling programme. The ultimate rock socket shaft resistance was determined using:

- A number of different theoretical design methods based on the rock mass characteristics provided in the Site Investigation (SI); and
- Available load test data.

Figure 1 shows a pile as constructed in the ground with the base and shaft resistance clearly marked.

Figure 1 - Pile Shaft and Base Resistance (Salgado *et al.*, 2011)



In order to account for geotechnical variability across the site, the design was based on the mudstone as it was the weakest rock encountered in the succession of strata. The sandstone and conglomerate recovered in the boreholes as well as the igneous intrusion, would provide even greater pile capacity when compared to the mudstone. Given the competence of the rock mass, the rock socket shaft resistance was considered reliable and was used in the design.

The preconstruction design was undertaken prior to the completion of the second phase of the Site Investigation (SI) and anchor load testing. Theoretical design approaches based on the available SI at the time indicated the rock socket shaft resistance to be around 1000kPa. Also, Boyd and Ozsoy (2013) carried out a soft toe pile test and measured 1300 kPa for a pile founded in a similar sedimentary rock mass in Glasgow. Yet, the preconstruction geotechnical interpretive report specified that the tender design was to be based on an ultimate rock socket shaft resistance of only 250 kPa for the mudstone. As the SI was incomplete, and there was still a risk that less competent rock could be identified. Further, there was no guarantee that third party checkers would ever approve a value greater than 250 kPa listed in the geotechnical report, until field evidence was available that would justify an increase in the maximum shaft resistance limit. Consequently, a value of 250kPa was used in the preconstruction phase design in order to mitigate the project budget risk as the foundation design was developed.

At the time, a blind comparison was done by the piling designer using the Sydney Rock classification method (Pells, Mostyn and Walker, 1998), based on the available initial SI information for the project. Using this method the mudstone was classified as a Class II Fractured Medium-Strong Shale with Unconfined Compressive Strength (UCS) of 7MPa. This classification resulted in a rock socket shaft resistance of 350 kPa for the weathered rock to 800 kPa in the general rock mass. Coincidently, the initial SI reported the minimum Unconfined Compressive Strength (UCS) value measured for the mudstone was ~7MPa.

As part of the second phase SI, an anchor was tested to failure in tension to assess the ultimate rock socket shaft resistance in the mudstone. The anchor was installed and then tested using a stressing jack with a calibrated hydraulic pressure gauge and dial gauges (See Figure 2). The anchor was pulled out of the ground whilst the applied tension load and anchor head deflection were measured in order to observe the insitu shaft resistance between the mudstone and the grouted anchor. When compared to the piles, the anchor load test was smaller in diameter and loaded in tension as opposed to compression. However, the test effectively measured the insitu ultimate rock socket shaft resistance as the anchor failed at the shear interface between the rock and the grout. The test was done in the Very Weak to Weak Mudstone and measured two results for ultimate rock socket shaft resistance of 1000 kPa and 1830 kPa. These values were obtained by measuring the failure load (kN) of the anchor and dividing by the area (m^2) of contact between the grout and the rock mass.

Figure 2 - Test Anchor (Bray, 2015)

After completion of the second phase of SI and anchor load testing, the geotechnical interpretive report was revised and the ultimate rock socket shaft resistance was increased to 520 kPa in the mudstone and this was considered to be a lower bound value for the rock mass. The design approach nominated in the interpretative report was to factor 520 kPa by half to 260 kPa and then verify the resulting pile resistance against the (unfactored) Serviceability Limit State (SLS) loads.

A project review of the geotechnical design and parameters for the project was undertaken by an external designer. This review proposed a similar ultimate rock socket shaft resistance of 520 kPa but reduced the applied factor of safety from 2 down to 1.2, again verified against the SLS. Further, they proposed a 10MPa contribution from the pile base to be used in the Eurocode Ultimate Limit State (ULS) calculations. The ULS design check was not critical, hence the design was insensitive to pile base resistance. This became the agreed design approach on the project. The only outstanding parameter to be agreed was the rock socket shaft resistance which became critical to the design.

The contractor's pile designer made their own determination of the ultimate rock socket shaft resistance and the different methods used are summarised as follows:

- R&J - Rosenburg & Journeaux (1976)

  Weathered Rock 375kPa

  Fresh Rock 1094kPa

- Horvarth (1978)

  Weathered Rock 330kPa

  Fresh Rock 933kPa

- H&K - Horvarth and Kenney (1980)

  Weathered Rock 250kPa

  Fresh Rock 707kPa

- M&W - Meigh & Wolski (1979)

  Weathered Rock 220kPa

  Fresh Rock 766kPa

- W&P - Williams and Pells (1981)

  Weathered Rock 338kPa

  Fresh Rock 967kPa

- R&A - Rowe & Armitage (1987)

  Weathered Rock 450kPa

  Fresh Rock 1273kPa

- Anchor Load Test Result TA2

  Weathered Rock Not Tested

  Fresh Rock 1000kPa

After the final review of all of the second phase SI, the contractor's piling design was based on 1000kPa generally for the rock mass with a 0.5m weathered zone at the top of the rock established at 338kPa. The value of 1000 kPa determined by the anchor test data was used as it was considered to be the most reliable and fell approximately in the median of all the theoretical design approaches. Boyd and Ozsoy (2013) also concluded in their paper that the method described by Williams (1981) was the most accurate, which resulted in 967 kPa for the mudstone, again approximately confirming the proposed value of 1000kPa. When compared like for like, this was nearly double the proposed design value 520 kPa.

5. Soft Toe Preliminary Pile Test

To further verify the contractor's design, a soft toe pile test was undertaken to inform the final design approach adopted for this project. In order to account for geotechnical variability, the test pile was located close to a borehole that ensured that the rock socket was positioned in the mudstone. During construction, the pile arisings were logged to confirm the socket material was in fact mudstone. Figure 3 below shows the Bauer BG 42 piling rig that was used to construct the soft toe test pile and the four reaction anchor piles used as part of the test.

Instrumentation, consisting of 5 levels of vibrating wire strain gauges, was used in the test pile to assess the rock socket shaft resistance.

Figure 3 - Bauer BG42 Piling Machine with Segmental Casing Installing Test Pile and Anchors

Figure 4 is a photo taken during construction of the test pile showing the steel reinforcing cage with the white polystyrene void former visible at the base of the cage used to create the soft toe.

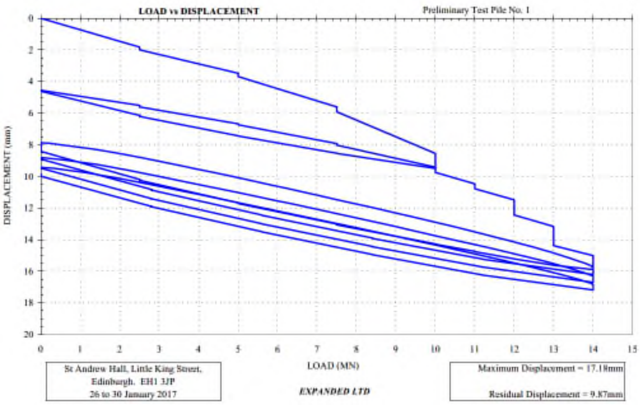
Figure 4 - Pile Cage with Soft Pile Toe Being Installed Test Pile Cage

Figure 5 below shows the 14MN pile testing frame that was used to test the pile.

Figure 5 - 14MN Pile Test Frame

The pile load test results are shown in figure 6 below.

Figure 6 – Pile Load Test Results



The test proved a rock socket shaft resistance in excess of 1000kPa in the mudstone. However, the 14MN test frame did not manage to fail in the rock socket so the pile had more reserve capacity. Applying the Sydney method to this rock mass would appear to be conservative based on both the anchor and pile load test results.

6. Conclusion

The soft toe pile test isolated the shaft from the base resistance and in this way measured the ultimate rock socket shaft resistance, albeit undermobilised. The results from the pile test measured a rock socket shaft resistance in excess of 1000kPa in the lowest strength rock (mudstone). This result was similar to other load test results nearby, was consistent with the anchor testing and fell close to the median of a range of commonly used theoretical approaches based on the site investigation.

Based on the soft toe pile test results the design was approved using an ultimate rock socket shaft resistance of 1000kPa. This significantly reduced the pile rock socket lengths across the project. Estimates of the reduction in rock socket drilling were approximately 40% and this equated to around 1850m^3 of concrete.

Buildings are currently responsible for 39% of global carbon emissions, as a result decarbonising the sector is one of the most cost effective ways to mitigate the worst effects of climate change (Adams, Burrows and Richardson, 2019). The resulting reduction of embodied carbon was 705,197kg based on the material alone using a figure of 0.159 kg of embodied carbon CO_2 per kg taken from the Inventory of Carbon and Energy (ICE) database for building materials (Jones, 2019).

The test also enabled an optimisation of the pile cap layout and a change from groups of smaller piles to one large single pile supporting each column. This reduced the number of bearing piles from 240no to 178no piles. The test also meant a reduction in the rock socket length of the restricted headroom piles which were installed using smaller rigs that drilled slower through the rock; realising significant improvements in cost, programme and carbon.

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