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Back-analysis of Static Load Tests on Bored Piles Founded in Glacial Soils and Weak Rocks in Northern Ireland

Les analyses des charges d'épreuves sur pieux forés en sols glaciales et en roches faibles en Irlande de Nord

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ABSTRACT: The A6 Randalstown to Castledawson Dualling project comprises two separate sections of main transport corridor realignment near the northen end of Lough Neagh, the largest lake in all of Northern Ireland and Ireland. The banded nature of basalt from lava flows and ensuing weathering actions during glacial periods meant that core recovery and strength can vary significantly, with UCS test data ranging from 2MPa (extremely weak) on the lower bound up to occasionally strong to very strong (maximum 110MPa). Overlying lacustrine, laminated and glacial soil sequences, to depths between 10m and 50m were variable and also had the potential to induce additional negative skin friction loading affecting the piles. Socketing of piles in bedrock was necessary at several structures to allow for the appropriate design factors in accordance with Eurocode 7. The ability of Continuous Flight Augers (CFA) to penetrate sufficiently into harder basalt rock materials pose significant construction risks so the requirement for rotary bored equipment can add considerable additional expense to construction of structural foundations. In some cases, instrumented test data allowed determination of resistance mobilised in the overlying soils. The paper will explore the range of soil/rock strengths and stiffnesses derived from investigations and correlate these with the resistances proven and inferred from static load tests.

RÉSUMÉ: Le project d'A6 de Randalstown jusqu'á Castledawson y compris deux routes a deux séparées prés du lac Lough Neagh, qui est le lac le plus grand en Irlande de Nord (présque 392 km^2). On a besoin des pieux fondés en roches pour obtenir les coefficients d'EC7. La structure bandée des roches „basalt" et les actions abrasifs pendant la durée glaciale sont réduit tous les deux la résistance compressif uniaxiale et la la qualité des carotes rétrouvé pendant les études. Les séquences des sols a profonds entre 10m et 50m sont variables, dont que les actions des digues peuvent augmenter en friction négatif de côté sur les pieux. La capacité d'équipement CFA pour forer la roche dure est limitée par rapport l'équipement plus chère pieu foré rotatif. Les instruments installées en avant des charges préliminaires peuvent expliquer les résistances résultants dans les sols. La variation des résistances et rigidités sont corrélé aux renseignements y engistrés.

Keywords: Ground investigations, Pile foundations, Load testing, Rotary piling (bored/CFA)

1 INTRODUCTION

The A6 Randalstown to Castledawson improvement scheme in Northern Ireland involves the design and construction of 14 km of new dual carriageway and associated primary structures through an area with complex geological conditions. Many of the structures in Zone 1 of the scheme required deep foundations in order to satisfy load carrying and movement criteria. Arup and Roughan O'Donovan (AROD JV) as designers for the Graham Farrans construction joint venture have developed the detailed design for the scheme, including the geotechnical design for the earthworks and structures.

This paper outlines the ground conditions observed, the variety of pile types selected and the testing obtained on preliminary piles at several key structures. Fully instrumented pile load tests using strain gauges and extensometers were used in some cases and the findings from these tests enable further interpretation of load tests to compare with initial pile settlement calculation methods. Other practical considerations are discussed including penetration of CFA equipment in variable rocks.

2 STRATIGRAPHY ENCOUNTERED

Lough Neagh was formed by a massive tectonic subduction event resulting in a pull-apart basin, resulting in a large lake of almost 392 square kilmetres surface area, the northern tip of which is shown in Figure 1 (bottom right). Figure 2 shows where this project and the existing Toome Bypass crosses a constriction and other inferred geomorphological features. A parallel subsidiary valley infilled with other glacial deposits was also found at structure S04 north of River Moyola. Structure S11 is located right on the inferred western margin of the basin feature.

The ground investigations carried out for the scheme included standard boreholes along with Cone Penetration Testing (CPTu) in soft ground. Several borings experienced ongoing blowing sands and occasional loss of stability. CPTu testing could in hours penetrate to depths that took traditional boring methods almost a week due to the ongoing influx of sands. The ground conditions consist of peat, variable alluvial silts and sands, underlain by laminated glacio-lacustrine clays or a thin cover of glacial till.

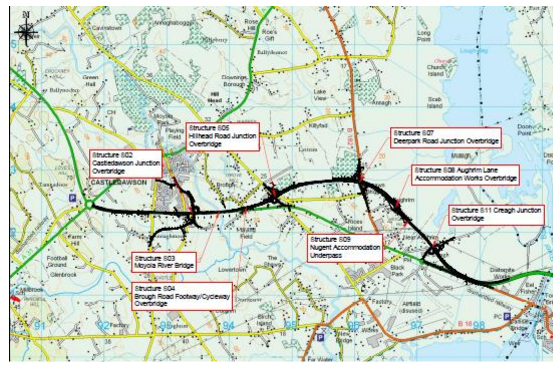

Figure 1: A6 Zone 1 and Lough Neagh / Lower River Bann (©Crown Copyright)

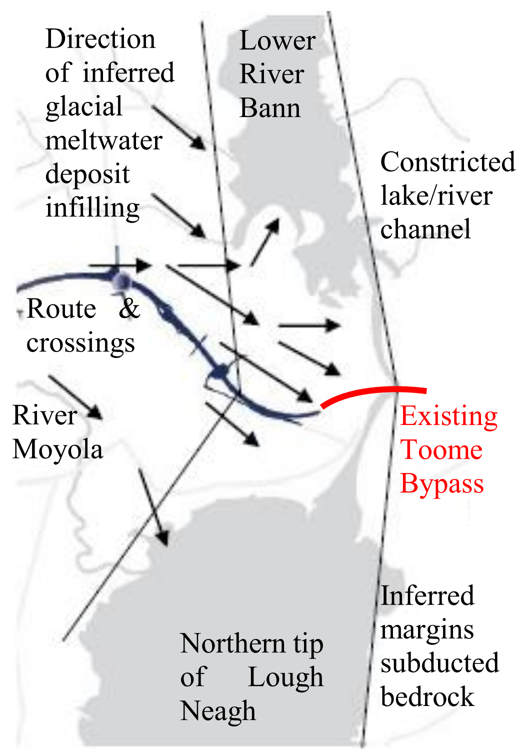

Figure 2: Inferred glacial depositions and influence of lake basin on bedrock

The most characteristic soil conditions at S11 and in the areas closest to the lake/river are discussed in more detail. Organic and peat soils were present in the upper 2m, occasionally deeper. High groundwater levels were also present, with the route traversing areas vulnerable to flooding. The majority of soils encountered had varying silt and sand content, with typical deposits comprising loose to medium dense silty to very silty fine to medium sand with occasional fine to medium gravel being present and in general these were uniformly graded.

The CPTu corrected cone resistance at S11 is presented in Figure 3 showing local pockets of dense sand and the underlying low to medium strength glacio-lacustrine deposits.

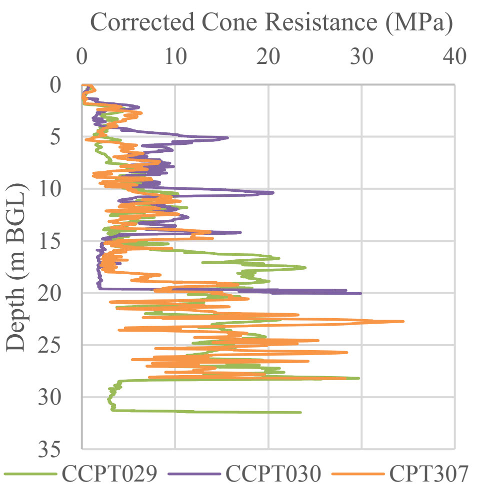

Figure 3: CPTu cone resistance near S11

The majority of the Standard Penetration Testing (SPT) test results in alluvial sands were in the range of N=10 to N=20, whether uncorrected or corrected for energy (N_{60}) and groundwater levels in accordance with BS EN 1997-2 Annex F Table F.1. The vast majority of these deposits were therefore considered to medium dense with Density Index (I_D) of 35% to 65%. Annex F Table F.3 provides example correlations of density to friction angle based on the uniformity and grading of deposits, suggesting effective friction angle of between 34 degrees and 41 degrees which is supported by parameters derived from the CPTu data. Lower SPT N-values were frequently observed in blowing sands. CPTu testing can give more reliable SPT results than SPT equipment can due to such disturbances (Robertson & Cabal, 2015). Characteristic values of SPT N-values from the CPTu plots given in Figure 3 average at N= 14 to 23 to 14.9m BGL.

Higher silty contents tended to contain moisture contents in excess of 25%, similar to alluvial deposits observed elsewhere in the Republic of Ireland near the River Shannon (Buggy & Peters, 2007). The shear strengths for the cohesive soils were derived from combinations of high quality triaxial tests undertaken on carefully preserved soil cores and/or derived using what would considered to be conservative correlations from in-situ testing including SPT and CPTu testing (Long, 2018).

The proportions of deposits relevant to each structure is presented in Figure 4 (omitting peat).

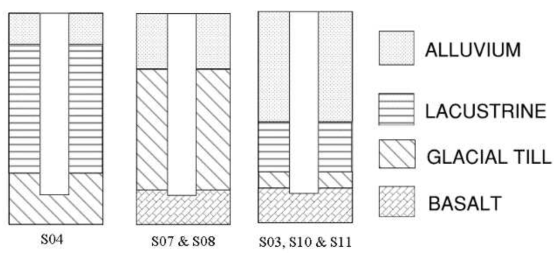

Figure 4: Indicative stratigraphy

Electrical resistivity and seismic refraction geophysical surveying at S11 did suggest the presence of firm to stiff or dense deposits even within the upper 5m however the intrusive investigations above suggested much lower resistance with the exception of CPTu testing.

Seismic refraction surveys at S03 and S07 indicated seismic velocities in uppermost levels of weathered basalt of between 2,400m/s and 2,600m/s, with several locations affected by either faulting, fracturing or potentially from shear force of glaciation on the top of rock. The interpreted fresh rock had seismic velocities of at least 3,500 m/s and up to 4,000 m/s. A similar range in conditions are expected at all of these piled structures based on review of rock core descriptions and core quality. At S11, the survey was limited due to the excessive depths to rock for the acquisition parameters.

The bedrock in the area is basalt which was encountered as fresh to highly weathered. Due to the banded and occasionally fractured basalt, the Rock Quality Designation (RQD) varied especially over the upper few metres into rockhead. RQD rarely exceeded 70% in the upper 3 to 4m, up to 7m at S07 (deeper fractured rock) but with notable exceptions as discussed below.

Figure 5 shows the relevant dataset for structures with rock strength plotted vs depth into rock (including weathered rock).

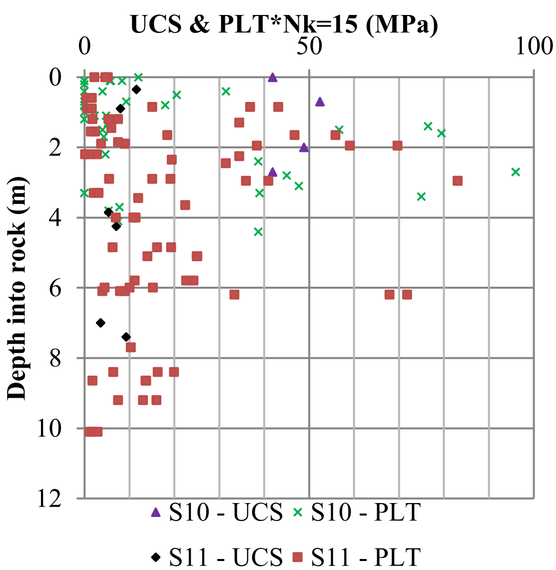

Figure 5. Rock strength vs. Depth into rock

Some of the better quality rock strength test data at S11 showed the basalt was very weak to medium strong rock (6.8MPa in weathered rock increasing to 47.0MPa) however lower strengths were frequently recorded via Is_{50} point load testing. Deformation testing on two strong fresh samples provided intact rock modulus of 14.9 and 17.3 GPa, and Poisson's Ratio, ν, of between 0.23 and 0.34, resulting in a Modulus Ratio of 368 and 392.

3 FOUNDATION DESIGN

3.1 Structural Pile Requirements

Typical load requirements for piled structures lead to arrangements with piles with rock sockets to provide from 1,500 kN to 2,350 kN however some negative skin friction was also anticipated.

Consultation with piling contractors had suggested that the length of embedment in competent basalt using typical CFA equipment would be a maximum of 0.5m and that there would be no guarantee of being able to progress through thick weathered rock where present. Such circumstances occurred at S03 which prompted the use of rotary bored piles. At S07, the geophysical survey indicated a zone of faulting or fracturing however intrusive boreholes indicated satisfactory conditions. Due to varying depths to rock and soil strata at S11 on the margins of the lake basin as described in Section 2, bored piles were also required for various reasons including the overall pile length.

3.2 Design Settlement Calculations

The settlement of the individual piles (ρ) can be estimated using this formula outlined in Tomlinson (2001):

$$\rho = \frac{(W_s+2W_b)L}{2E_pA_b} + \frac{\pi}{4} * \frac{W_b}{A_b} * \frac{B(1-\nu^2)I_p}{E_b} \qquad (1)$$

Where W_s (kN) is the load carried by the pile shaft, W_b (kN) load carried by the pile base, L (m) is the total length of the pile, A_s(m²) is the area of the shaft, E_p (GPa) is the deformation modulus of the pile material, A_b (m²) is the area of the base, ν is the Poisson's ratio of the material below the base of the pile (=0.5 for glacial till or =0.3 for basalt), I_p is the influence factor related to the ratio of L/R (=0.5), B (m) is the pile width and E_b (MPa) is the deformation modulus of the material below the base of the pile.

The original design calculations assumed that 30% of applied load would be carried in the shaft and 70% in the base in order to yield an upper bound estimate of pile settlement for piles terminating in the overburden. For piles terminating on rock the settlement calculation was carried out assuming 100% of the load is carried in the base, except for deep piles at S11.

4 PRELIMINARY PILING

4.1 *Preliminary Pile Construction*

At each structure, a preliminary pile was constructed using the same plant as proposed for the working piles. This involved Soilmec SR95 plant utilising casing 700mm diameter temporary casings and 600mm diameter rock augur in the case of rotary bored piles at S11 (SR70 at S03) or Soilmec SR75 plant equipped with CFA tools.

At structures S08 and S10 the range of RQD was observed to be higher than elsewhere casting doubts on the suitability of the CFA equipment. Test pile locations were typically selected expecting RQD of 20 to 30 and embedment into competent rock but other considerations on the statigraphy also influenced these choices.

Six preliminary piles were successfully installed and tested as listed in Table 1, using the same steel reinforcement cages, supplemented where required with additional strain gauge instrumentation spot welded at the relevant levels. For the rotary bored piles, instrument tubes to accommodate the proposed cross-hole sonic logging integrity testing were also provided. These were also suitable for installation of retrievable extensometers for the load test, following the procedure for an extended maintained load test as set out in the core Specification for Highway Works (2016).

Both S10 and S11 test piles were instrumented with a set of four vibrating wire arranged at quarter points on helical steel reinforcement strain gauges at four levels in the pile. The gauges were placed to correspond to stratigraphic interfaces and were included to provide data on the mobilised shaft resistance in each stratum.

Table 1. Preliminary / design pile criteria

Structure Pile Type & Base material	**L (m)**	**Dia. (mm)**	**SWL (kN)**	**DVL (kN)**	**Max Load (kN)**
S03 Bored into rock	17.7	700/ 600	2,025	2,225	5,260
S04 CFA glacial till	16.0	600	502	660	2,919
S07 CFA into rock	17.5	750	1,900	2,050	4,900
S08 CFA into rock	10.0	750	1,500	1,500	3,950
S10 CFA into rock	12.0	600	2,000	2,000	5,000
S11 Bored into rock	33.0	700/ 600	2,350	4,750	8,270

Allowances for twice the estimated negative skin friction were accounted for in selection of the Design Verification Load (DVL) compared to the Specified Working Load (SWL).

4.2 *Static Load Test Results*

Static load test data is presented in Figure 6, indicating the pile head load for the tests outlined in Section 4.1 Table 1 above.

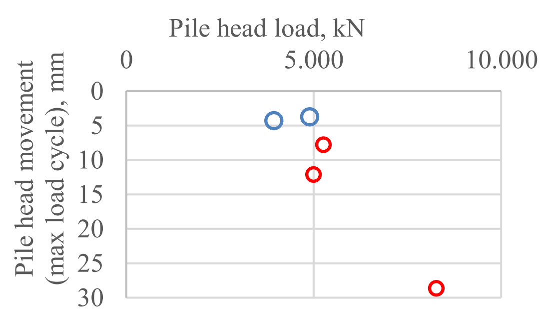

Figure 6.Pile head movements vs head load (blue-750mm in rock-CFA / red-600mm in rock)

5 ASSESSMENT

5.1 CFA Piling Penetration into Basalt

During design of the piles at the A6 R2C project, some difficulties in pile selection related to uncertainty in the capabilities of the CFA equipment to penetrate through both weathered and competent basalt bedrock comprising banded lava flows with highly variable strength, weathering and fracturing.

Casings and rotary boring techniques were used at structures where loose deposits susceptible to disturbance had been proven in investigations.

Embedment of greater than 0.5m in reasonably high RQD basalt was not assured based on preliminary enquiries. This guidance was supported by the records from preliminary trials at S08. The apparatus did achieve between 1.3 and 1.6m embedment through weathered rock and into higher quality materials.

The trials showed that the upper weathered rock posed no barrier to getting into the rock with the CFA plant and it was necessary for better quality competent basalt to reach the limit of its capabilities.

A notable lesson is that some investigation boreholes were progressed with excessive chiselling and rotary open-hole drilling and missed coring some of the weathered rock immediately at rockhead. This limitation in information can lead to design decisions being made based on misleading assumptions.

5.2 Mobilised Pile Resistances

The instrumented pile at S10 showed very high mobilised shear stresses in the uppermost alluvium and could not be considered reliable, even if upper peat materials had been replaced locally for the adjacent earthworks prior to piling.

Assessment of S11 pile concrete modulus required several iterations to derive consistent data from the strain gauges also proved troublesome. Higher levels of resistance were mobilised in the deep alluvium soils than anticipated while negligible readings were obtained in the lowest gauge in weathered rock / top of competent rock (RQD>30). Extensometer readings were used to match the approximate modulus and load readings over these intervals.

The derived distributions of load along the length of the pile for each of the main load cycles is shown in Figure 7. The mobilised shaft resistance p_s (=W_s / A_s) over each zones gauges are plotted in Figure 8 vs. pile head movement normalised by the relevant pile diameter.

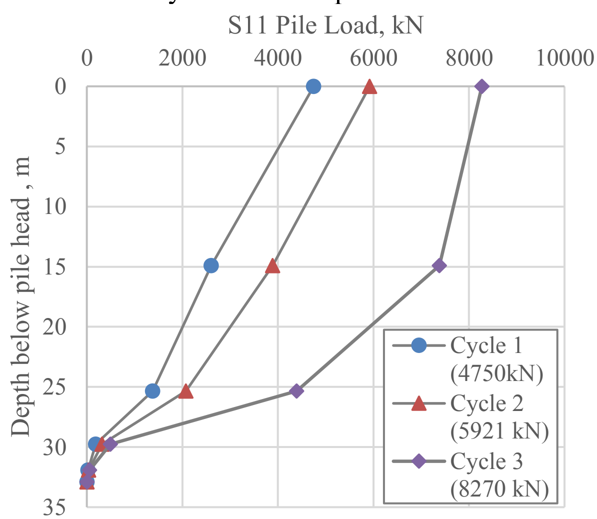

Figure 7: S11 Pile load vs depth per load cycle

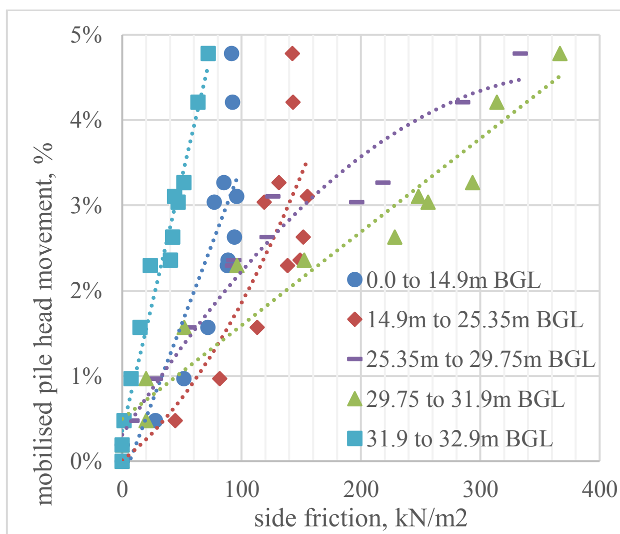

Figure 8. Mobilised shaft resistance (S11)

The key variant in determining these distributions was the assignment of the concrete modulus which is strain dependent and which is not consistent along the pile length (Fellenius, 2001).

The maximum reading in the gauges were of the order of 1000 microstrain ($\varepsilon \approx 0.1\%$ strain). The ratio of differences in strain to differences in load was analysed from the factual data suggesting the majority of concrete tangent modulus was in the range of 20 GPa to 55 GPa, although much higher values were also determined particularly where unloading cycles and low strain were observed. Strain softening was particularly evident during the maintained hold stages in the upper strain gauges.

Residual loading within the piles was also suspected however the strain readings at start of S11 test prior to any load application were considered negligible at between -0.7 and +0.2 microstrain averaged across all levels, with the vast majority being in compression. Concrete tangent modulus derived over the very first load increment to 100kN was indicated consistently at 90 GPa at the upper strain gauges increasing towards 850 GPa close to rockhead at 30m BGL.

Another aspect that would have helped in analysing the data would have been more frequent extensometers as one of them appeared to exceed its maximum range at an early stage in the load test (above the crucial design loading).

The proportion of load reaching the competent rock (socket from 31.9m to 32.9m) was negligible compared to the estimated design resistance. The majority of load in the pile was transferred via the overlying soils and in the upper weathered / competent rock.

5.3 Soil-Pile Resistance

Calculation methods for estimating pile shaft resistance in coarse soils are based on effective stresses. The mobilised shaft resistance in the test piles was much higher using the coefficients given via BS 8004 Cl. 6.4.1.2.2 Table 8 and 9 for K_s (0.5 to 0.9 depending on soils/pile type) and δ (=1.0 for bored / CFA piles). The footnotes to Table 8 do state that these values can be superseded by local static pile load test data. It was necessary to ignore the water levels to achieve the same resistance derived from the testing for these same coefficients and $\phi' = 41°$.

BS EN 1997-2 also provides unit shaft resistance in coarse soils with little or no fines in Annex D Table D.4. The CPTu plots in Figure 3 had average cone resistance of $q_t = 4.5$ MPa to $q_t = 7.7$ MPa to 14.9m which would suggest $p_s = 40$ kPa/m to $p_s = 60$kPa/m, much less than the $p_s =75$ to $p_s = 80$kPa/m shown in Figure 8 above. BS EN 1997-1 Cl.4.3.4.2(1)P notes that '*if ultimate compressive resistance of piles is derived from CPT results, calculation rules based on locally established correlations between results of static load tests and CPT results shall be used*'.

5.4 Rock Socket Resistance / Settlements

The mobilised pile resistance within combinations of weathered and competent rock from 30m to 32m BGL at S11 were shown in Figure 8 to have approximately 240 kPa/m for 2% pile head movement.

Separately, the settlements of all piles listed in Table 1 founded in basalt were reviewed against the assessed range in characteristic rock mass modulus using Equation 1. Once pile elastic compression was deducted from the applied loads, the observed settlements were used as a means to establish base rock mass modulus for a typical applied base pressure. In the course of this exercise, it was clear that the majority of loading at several structures was almost entirely taken in shaft resistance along the length of the pile with only 10% to 20% being transmitted through into the rock. This generally suggested the rock mass modulus should be in the range of 100MPa to 200 MPa for W_B / A_B of 1 MPa to 4 MPa. Much higher stiffness is likely.

The load test data obtained for pile settlements compare very well with the ranges provided in CIRIA C181 Figure 3.5 which presents data from Canada on the smoothness of rock sockets in weak sandstone and mudstone rocks, as shown in Figure 9. This may be coincidental given the soil resistances described above.

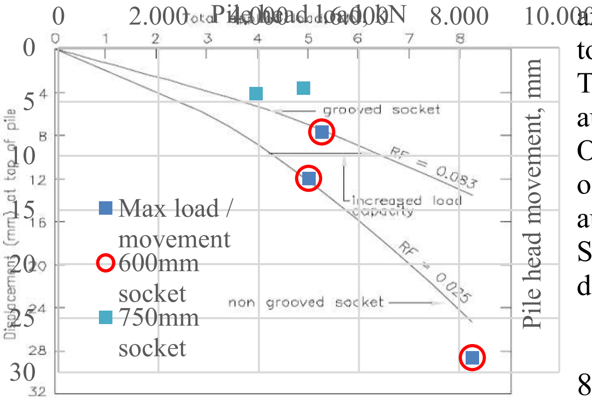

Figure 9. CIRIA C181 Fig 3.5 vs A6 load tests

6 CONCLUSIONS

Pile requirements and testing information has been presented for a project in a complex glacial setting. Preliminary static load testing CFA and rotary bored pile foundations has been beneficial.

CFA penetration in weathered and competent rocks have been proven to be effective for the loadings indicated. Rotary boring for greater embedment into basalt is advisable.

Mobilised shear strengths and normalised pile head resistance have been presented showing some unusual performance compared to the design expectations, exceeding the proportions derived from characteristic values arrived at when designing to conservative principles.

The design resistance estimated in accordance with Eurocode principles often erred on the side of caution. This was partly based on the poor borehole stability, low SPT N-values observed in blowing sands and low strength data in laminated soils susceptible to disturbance. The pile test programme proved otherwise with much greater loading being taken in shaft resistance throughout. It is notable that there was insufficient data from tests taken on to failure in order to determine the ultimate limit behaviour of the piles in these ground conditons.

7 ACKNOWLEDGEMENTS

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8 REFERENCES

BS EN 1997-1:2004+A1:2013. *Eurocode 7 Geotechnical Design Part 1 General Rules including National Annex*, British Standards Institute

BS EN 1997-2, 2007. *Eurocode 7 Geotechnical Design Part 2 Ground investigation and testing*, British Standards Institute

BS 8004, 2015, *Code of Practice for Foundations*. British Standards Institute

Buggy, F. & Peters, M. 2007. *Site Investigation and Characterisation of Soft Alluvium for Limerick Southern Ring Road - Phase II, Ireland*. Proceedings of the conference on Soft Ground Engineering, Portlaoise. Paper 1.6 Geotechnical Society of Ireland

Fellenius, B.H., 2001. *From strain measurements to load in an instrumented pile.* Geotechnical News Magazine, Vol. 19, No. 1, pp 35-38.

Gannon, J.A., Masterton, G.G.T, Wallace, W.A. & Muir Wood, D., 1999. *Piled foundations in weak rock*. CIRIA Report C181

Long, M. 2018. *Geotechnical Properties of Irish Compressible Soils*, 2nd Hanrahan Memorial Lecture, Lecture held at Engineers Ireland, Dublin on 25th April 2018.

Robertson, P.K. & Cabal, K.L., 2015, *Guide to Cone Penetration Testing for Geotechnical Engineering 6th Edition*, California

Tomlinson, M.J. 2001. *Foundation Design and Construction, 7th Ed*, Prentice Hall