Proceedings of the 11th ANZ Young Geotechnical Professionals Conference – 11YGPC
Queenstown, New Zealand
25 – 28 October 2016

Interpreting geotechnical measurements and observations: Does one size fit all?

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ABSTRACT

This paper illustrates via simple real life examples, the constraints and limitations of three common geotechnical assessment techniques. The purpose of these case studies is to highlight the need for geotechnical engineers to not only choose the right tool for their analysis, but also to fully understand and appreciate the limitations of these techniques in the context of the project requirements. In the first example, a borehole and test pit excavated within close proximity to one another is shown to lead to different interpretations of stratigraphy, and potential implications for the project in question are discussed. In the second example, PDA tests undertaken on the same driven pile at the same time are shown to result in inferred pile capacities which vary by over 20%, as a consequence of the way the data is processed and assumptions made, highlighting the need to avoid 'black box thinking' when interpreting geotechnical observations and measurements. Finally, an example is presented of an assessment of "native soil modulus" in the context of flexible pipeline design. Conventional methods of assessing the stiffness of the ground, which may be conservative for the design of a foundation, are shown to be potentially unconservative for the purposes of design of the pipeline. In this case, it is necessary to understand the wider project context and performance criteria of the pipeline, in order to provide appropriate geotechnical design advice.

Keywords: geotechnical investigation, PDA testing, native soil modulus, observations, interpretation

1 INTRODUCTION

Geotechnical engineering practice requires the application of theories and techniques which are usually based on empiricism and observation. Rarely are situations encountered where completely unambiguous ground conditions exist, such that a single, closed-form solution can be applied to solve a geotechnical problem. Geotechnical solutions require the synthesis of a ground model, selection of design parameters based on predicted material behaviour and ultimately an engineering solution based on inferences made from a finite array of data points. If over-conservative design approaches are to be avoided, then observation during construction and/or operation of the design is necessary to obtain pragmatic solutions. Burland (1987) discusses the importance of understanding the ground profile, soil behaviour and applied mechanics and design model, and stresses the need for "empiricism and well-winnowed experience" in order to reliably apply these skills to a geotechnical problem. The interplay between these aspects of geotechnical engineering is illustrated in Figure 1 as a triangle, with empiricism and experience common to the correct application of each. In this paper, the importance of experience and judgement in interpreting 'each point of the triangle' using common geotechnical tools and techniques will be explored, using case studies from the author's recent project experience.

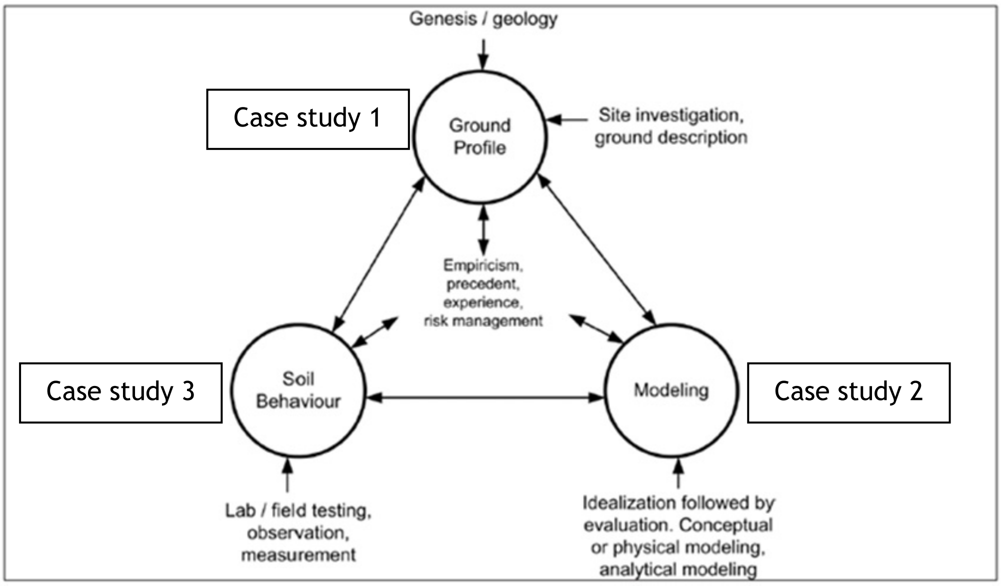

Figure 1. Ground engineering triangle (after Burland, 1987)

2 CASE STUDY 1: ENGINEERING IMPLICATIONS OF DIFFERENT GROUND PROFILES INFERRED FROM TEST PITS AND BOREHOLES

When planning and undertaking site investigations, it is important to select appropriate investigation techniques to suit the anticipated ground conditions, and also the design intent and purpose of the investigation. Use of different techniques can skew one's understanding and interpretation of the ground profile. How data is recorded and reported, particularly when it will be passed on to someone with without a primary background in geotechnics, can affect the correct application of the information at subsequent stages of a project.

At a site north of Melbourne, a geotechnical investigation was undertaken to assess ground conditions for new water-reticulation infrastructure. The investigation needed to assess conditions for a shallow slab footing and several hundred metres of trenches to allow installation of new buried pipes. The investigation comprised two shallow boreholes (drilled using a combination of auger and HQ coring techniques) at the site of the slab footing, and several mechanically excavated test pits (excavated using a 6t tracked excavator) along the route of the proposed trenches. The boreholes and test pits were logged by different geotechnical engineers at different times, and therefore at the time of logging, no direct comparison between logs produced from the test pits and boreholes was made. One borehole and one test pit however were located adjacent to each other (within 5m), and inferred ground conditions from each are illustrated in Figure 2.

Depth mbgl	BOREHOLE	TEST PIT
0	**Silty Clay (CH)**	**Silty Clay (CH)**
0.1	dark brown, trace fine sand	brown, some rootlets in top 200mm
0.2		Basalt fragments to 75mm
0.3		
0.4		
0.5		
0.6		
0.7		
0.8		increasing basalt cobbles
0.9		
1	**Basalt (MW)**	
1.1	grey, dark grey, slightly vesicular	
1.2	medium to high strength	
1.3	RQD = 12%	
1.4		Refusal at 1.4m
1.5	(hole continues to 5m)	

Figure 2. Comparison of inferred ground profile from borehole and test pit

The borehole log identified medium to high strength basalt at 0.9m below ground level, whereas the adjacent test pit identified clay with basalt fragments and cobbles to 1.4m depth. Whilst neither interpretation is incorrect when considered in isolation, it is important to consider which interpretation is the *most suitable* to include in an interpretative report. Figure 3 shows the core box retrieved from the borehole, and one of the basalt fragments retrieved from the test pit. The defect spacing and RQD of the rock means that it was easily excavated. During excavation, some mixing of soil with the rock fragments (possibly along with crushing of extremely weathered and residual material), gave the appearance of rock fragments within a clayey matrix, rather than material that was indeed mostly rock, as was inferred from the borehole.

The observations arising from the different investigation techniques have important implications in the engineering design for which the investigation was intended, noting that subsequent stages of the design would be undertaken with limited direct input by a geotechnical engineer:

- If only boreholes were undertaken, rock core similar to that shown in Figure 3 would likely have been retrieved at all investigation points. An empirical assessment of excavatability could be made based on personal experience or published approaches but would likely be quite cautious, without direct evidence of the excavatability of the material;
- If only test pits were undertaken at the site, a sound understanding of excavatability across the site would be obtained. Bearing capacity assessments in the vicinity of the footings would however tend to be more cautious, and would likely infer lower allowable bearing capacities and higher shrink-swell potential, owing to the engineering logs suggesting a mostly soil-like profile.

Thus, this case study highlights the need to apply different investigation techniques to achieve different design outcomes, and being careful to interpret and report data in a way that is not misleading or ambiguous.

Figure 3. Rock encountered in borehole and test pit

3 CASE STUDY 2: INTERPRETING PDA TESTS ON DRIVEN PILES

Dynamic pile testing by way of the pile driving analyser (PDA) is a commonly used tool with which to directly verify the capacity of driven piles. Whilst most commonly used for verification during construction, the technique can be used to refine designs and to design remedial solutions to rectify problems encountered at difficult sites (Reed et al, 2016).

At a site where multiple testing contractors were engaged to undertake PDA and CAPWAP testing, a reference pile was selected to benchmark and compare the results of two of the testing contractors. A single steel 1500mm diameter tubular pile was selected, and two testing contractors were invited to instrument and take readings from the pile during testing, and subsequently provide a recommended 'as-built' capacity for the pile. Both contractors instrumented the pile with their own strain gauges and accelerometers and used their own testing kit; however both contractors recorded and independently interpreted the same sequence of hammer strikes on the pile.

Table 1 compares the results for the same hammer strike which was reported by the two contractors. Hammer blow 2 was agreed by the contractors to be the blow of comparison; other blows were reported by Contractor A but not by Contractor B, and therefore a more thorough comparison is not possible.

Table 1. Difference in test results between testing contractors

Contractor	Hammer blow	Inferred shaft resistance (kN)	Inferred base resistance (kN)	Inferred total resistance (kN)
Contractor A	2	7312	1400	8712
Contractor B	2	5779	1368	7147
Difference between A and B on Blow 2		26.5%	2.3%	21.9%

The difference in test results are negligible when base resistance is considered, however the differences happen to be considerable on shaft resistance. This is particularly important for piles which rely on shaft resistance as a significant portion of their design capacity, such as tension piles. In this instance, the differences between the results from Contractor A and Contractor B were attributed to apparently poor correlation between force and velocity measurements, thought to arise from nearby sheet piles, which in turn were dealt with in slightly different ways by the two testers. There was also some confusion over the average set and temporary compression measured by the piling contractor and used in the analysis, and also whether or not paint was scraped off the pile prior to the gauges being attached.

Table 2 shows interpreted results from different hammer blows from the same testing sequence on the test pile at the site, which were provided by Contractor A. The earlier, 1st hammer blow shows significantly higher resistance, whereas the later blow shows values more consistent with those shown in Table 1. The higher resistance on the 1st blow is attributed to more "undisturbed" ground conditions; each hammer blow moves the pile and leads to some breakdown of shaft resistance along the pile.

Table 2. Difference in PDA test results between different hammer blows on the same pile

Contractor	Hammer blow	Inferred shaft resistance (kN)	Inferred base resistance (kN)	Inferred total resistance (kN)
Contractor A	1	9062	2800	11862
	4	6127	1000	7127

Whilst this author places no judgement on either contractor for the results provided, it is nevertheless concerning that such variability could occur under highly controlled circumstances for what is normally considered a relatively routine test procedure. It is therefore important for a geotechnical engineer to have at least a basic understanding of how tests like PDA tests are undertaken and interpreted, before using the results to validate or calibrate their designs. A PDA test, not unlike other geotechnical tests, is a measurement which must be interpreted within the context of the ground model, expected soil behaviour, and sound engineering principles. Such a test is a measurement of the resistance of the pile in the ground subjected to a certain hammer blow, and it follows that a test result could be unrepresentative of the true ultimate geotechnical capacity of the pile if hammer energy is too low (which usually manifests in very low set readings) or too high (which can lead to inaccurate velocity or force measurements), if there is interference in the readings (which may have been the case in this example), or if set and temporary compression is recorded incorrectly.

As discussed by Reed et al (2016), understanding how to interpret a measurement such as PDA testing whilst not relying on it solely as a pass/fail result can prove invaluable in identifying and rectifying problems on site. At another location at the same site, piles were being installed with recorded final driving sets of between 20 to 45mm, which suggested ultimate geotechnical capacities of less than 2MN by application of the Hiley formula for piles requiring a design geotechnical capacity in excess of 5MN. Inspection of the ground model in the area suggested that the subsurface profile should be horizontally bedded and similar to the adjacent piling zone where piles were driven to the same toe level with a final set of 3 to 4mm. The discrepancy was resolved by reallocating a planned PDA test to the pile with the worst (highest) recorded set value. A conservative interpretation of the PDA testing on this pile (taking into consideration the factors discussed earlier) proved a design capacity of over 6MN for this pile. Together with the ground model, this data was subsequently used to demonstrate acceptance of all the piles with high sets in that zone. The data was also a trigger to review record keeping and quality procedures on site, as it suggested the recorded sets were likely too high.

4 CASE STUDY 3: NATIVE SOIL MODULUS FOR BURIED FLEXIBLE PIPES – WHICH MODULUS IS WHICH?

The design of non-rigid pipes (for example, HDPE) for many water infrastructure projects in Australia and New Zealand is governed by AS/NZS 2566.1 – 1998 Buried flexible pipelines Part 1: Structural design (Standards Australia, 1998). According to this standard the support provided to the pipe by the ground depends on the geometry of the trench, the properties of the embedment fill within the trench and the stiffness of the natural soil, which is defined by the parameter referred to as *Native Soil Modulus*, E'_n. AS2566.1 recommends values of E'_n based on soil type and SPT N value, replicated in Figure 4. Unfortunately, the standard does not differentiate between the stiffness of embedment material (which includes the deformation of the pipe itself) or natural material. These differences are discussed by ATV-DVWK (2000) and Howard (1996) which provide details on the derivation of these values via plate load testing or direct measurement of model pipe deformations.

Materials			Moduli E'_e and E'_n MPa				
Description	Classification		Uncompacted	R_D (%)			
	AS 1726 †	AS 2758.1		85	90	95	100
				I_D (%)			
				50	60	70	80
			Standard penetration test ‡ Number of blows				
			≤ 4	> 4 ≤ 14	> 14 ≤ 24	> 24 ≤ 50	> 50
Gravel— single size	—	Coarse aggregate	5§	7§	7§	10§	14
Gravel— graded	GW	Coarse aggregate	3§	5§	7§	10§	20
Sand and coarse-grained soil with less than 12% fines	GP, SW, SP and GM-GL, GC-SC etc.	—	1	3§	5§	7§	14
Coarse-grained soil with more than 12% fines	GM, GC, SC SM and GM-SC, GC-SC	—	NR	1§	3§	5§	10
Fine-grained soil (LL<50%) with medium to no plasticity and containing more than 25% coarse-grained particles	CL, ML, mixtures ML-CL and ML-MH	—	NR	1§	3§	5§	10
Fine-grained soil (LL<50%) with medium to no plasticity and containing less than 25% coarse-grained particles	CI, CL, ML, mixtures ML-CL, CL-CH and ML-MH	—	NR	NR	1	3	7
Fine-grained soil (LL>50%) with medium to high plasticity	CH, MH and CH-MH	—	NR	NR	NR	NR	NR

Figure 4. Recommended E'_n values based on AS2566.1

"NR" in Figure 4 implies "no reliable modulus" can be ascertained from the simple correlations used to derive the table. How then, can a designer evaluate E'_n for a pipeline through high plasticity (CH) clays, or if test results other than SPT values are available? This author has seen various approaches to solving this problem, many of which involve estimating "equivalent SPT values" from vane shear, pocket penetrometer or DCP data, so that Figure 4 (Table 4.2 in the standard) can be used. Such approaches introduce considerable error and uncertainty to the project, and still do not provide a solution for CH soils. This author has seen, on several occasions, geotechnical reports of 'NR' for very stiff CH soils, which provides the designer with absolutely no information with which to design the pipe. To overcome these shortcomings, this author recommends a somewhat expedient approach based on first principles and related to back-calculating the effective or constrained soil modulus beneath a footing, which has proved valuable on a number of project sites.

Review of recommendations by several pipeline manufacturers suggests that lateral movements of the pipe should be limited to less than 10 to 15mm to prevent damage to collars or joints. If one considers that a shallow footing loaded to its allowable bearing capacity can be assumed to settle by as much as 25mm (Look, 2014), then limiting deformation to these magnitudes for a flexible pipe requires conventional estimates of stiffness to be modified by a factor of at least 2.0 in order to derive a value of E'_n. Thus, an estimate of E'_n can be found by increasing a global factor of safety from 3.0 to 6.0, and otherwise adopting typical correlations between E' (not E_u) and undrained shear strength, noting that Howard (1996) states that "…there are various opinions for the relationship between E_s, M_s and E' and in light of the variability of SPT results, the terms may be considered interchangeable". Consideration of these factors allows the derivation of parameters in natural soils which are not accommodated in the standard (such as CH soils) or where SPT testing is either unavailable or unreliable.

5 CONCLUDING REMARKS

This paper has provided case study examples which exemplify the need to critically analyse and consider the results of three investigation and design approaches used routinely in geotechnical practice. Common to all three case studies is the risk of misinterpreting observations if the engineer fails to understand the 'bigger picture' of the works taking place, or the assumptions on which the technique being used is based. It is necessary for all geotechnical engineers to avoid learning and applying techniques by rote, and consider first principals at all times when undertaking geotechnical work.

6 ACKNOWLEDGEMENTS

The author wishes to thank Jacobs for the opportunity to attend this conference and present this paper, my clients from whom these project examples have been drawn, and my mentors and peers throughout my career to-date who have impressed upon me the importance of developing and applying sound engineering judgement.

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