GEOTECHNICAL INTERPRETATIONS USING VISUAL TACTILE METHODS

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ABSTRACT

Every engineering project has a scope, timeframe and budget. The budget or scope for a geotechnical investigation is generally a fraction of the entire project's budget. For small projects, due to limited time frames and budgets, the scope at each site can be limited to only one borehole with in situ testing and materials to be logged using visual tactile methods by a trained engineer/geologist. Using limited investigation data presents challenges to determine geotechnical design parameters and the engineering design of foundations. This paper presents a process for establishing geotechnical design parameters in the absence of laboratory testing for small projects such as telecommunication towers. Geotechnical design parameters are derived using established correlations and engineering experience.

1 INTRODUCTION

In the perfect world, all projects would have sufficient funding for site investigations to deliver the project requirements and minimise risk. However, smaller projects often have a restricted budget and costs are reduced by removing aspects such as laboratory testing. With the initiation of the Australian Government to implement the National Broadband Network (NBN), telecommunication towers are required at hundreds of sites across Australia. Geotechnical investigations for NBN sites are limited to one borehole with in situ testing and materials to be logged using visual tactile methods by a trained engineer/geologist. Using limited investigation data presents a challenge to determine geotechnical design parameters and the engineering design of foundations for a 30 to 40m monopole or 50m lattice tower. A typical required allowable bearing capacity for a shallow footing could range between 150 kPa and 200 kPa.

This paper provides a process for deriving geotechnical parameters without laboratory testing. It requires a site investigation with in situ testing and material classification by a trained engineer/geologist using visual tactile methods as per AS1726 (2017). It should be noted, that the methods used to derive each parameter in this paper are not new, and the same or similar procedures are commonly used by various engineering practices. The intent of this paper is to provide a process for the wider community, in a clear format, which delivers the requirements for small projects whilst maintaining confidence in the provided geotechnical parameters.

2 PROCESS

Small projects with budget constraints are not uncommon in the engineering industry. Second to this, with limited geotechnical information, designers tend to be conservative in design. It is important to understand the true requirements of a project and to propose a suitable methodology.

In situations where budgets do not allow for laboratory testing, the following process is proposed:

1. Understand the requirements and budget of the project
2. Perform a desktop study of the site area
3. Organise a geotechnical site investigation tailored to the project
4. Ensure a well-trained engineer/geologist undertakes the field investigation using visual tactile methods (in accordance with AS1726) and in situ testing
5. Verification of borehole log/s and material samples by an experienced engineering geologist / geotechnical engineer
6. Develop material parameters using empirical methods, engineering judgement and typical values
7. Verification of material parameters by an experienced geotechnical professional

A minimum of one borehole with in situ testing is considered, which is suitable for small projects such as telecommunication towers, where required allowable bearing for shallow foundation is around 200 kPa. This process has been used successfully in practice for over one hundred telecommunication towers. It should be noted, that for projects such as these, it is not uncommon that the equivalent of a graduate engineer/geologist is used on site, again due to budget constraints. This is acceptable provided the engineer/geologist on site has been trained and the verification process is followed.

2.1 DESKTOP STUDY

The desktop study should typically entail reference to local geology maps, local acid sulphate soil maps, available literature on the geology of the area and the review of nearby boreholes where available. In addition, satellite imagery from tools such as Google Earth can provide an important insight into access constraints and the site conditions when a site visit has not previously occurred. These tools can assist with selecting the most suitable field investigation and equipment for the site. Lastly, it is important, Dial Before You Dig's are carried out to determine if service locating is a requirement.

2.2 INVESTIGATION

As defined previously, the minimum requirement for the field investigation includes the following:

1. Trained engineer/geologist on site
2. One borehole*
3. In situ testing
4. Materials classified using visual tactile methods in accordance with AS1726

*CPT testing has not been considered in this paper

The process for characterising materials by visual tactile assessment is provided in AS1726. To ensure materials can be verified properly, it is important soil samples and core boxes are taken and maintained for verification purposes. As laboratory testing is not being undertaken, the design parameters solely rely on the validity of the logged materials. Due to this, the borehole log and materials from site must be verified by an experienced geologist to reduce the risk.

In addition, it is important the site engineer captures site photographs and documents a description of the site, detailing features such as drainage, vegetation, terrain, weather/ground conditions, presence of outcrops and the presence of any landslips. This assists with the verifier understanding the site conditions.

3 PARAMETER DEVELOPMENT

When applying parameters to be used in design, a number of factors need to be considered such as regional geology, sub-surface profile, in situ test results, consistency/strength of materials, and the sensitivity of each parameter on design. It is important to be aware of the parameters which have the largest impact on design. For example, variation of undrained shear strength and effective friction angle of soil has a significant influence on the resulting bearing capacity. Similarly, for rock, the uniaxial compressive strength has a high influence on the resulting bearing capacity.

When laboratory results are not available, parameters can be based on: borehole logs (including material classification and in situ testing), empirical formulae, correlations, typical values and general geotechnical 'rules of thumbs'. Published technical literature and textbooks are available to assist with deriving parameters. For this paper, the following resources have been used:

- Australian Standards
- Handbook of Geotechnical Investigation and Design Tables by B.G. Look (2014)
- Craig's Soil Mechanics by Craig and Knappet (2012)
- Correlations of Soil and Rock Properties in Ground Engineering by Jay Ameratunga et. Al. (2016)

Using the above literatures and field logs, the following parameters for geotechnical design will be estimated:

- The effective friction angle of soil and rock (ϕ')
- Undrained shear strength of soil (c_u)
- Unit weight of soil and rock (γ)
- The effective cohesion of soil (c')

- Poisson's ratio of soil for both undrained (υu) and drained conditions (υ ')
- Uniaxial compressive strength

3.1 SOIL PARAMETERS

Effective Friction Angle

The effective friction angle (ϕ ') for non-cohesive and cohesive soils can be estimated using AS4678-2002 or typical values from Look (2014) or other textbooks such as "Correlations of Soil and Rock Properties in Geotechnical Engineering" by Jay Ameratunga et. al (2016). For non-cohesive soils, the effective friction angle can be estimated with respect to angularity, grading, standard penetration test (SPT) results and fines component using Equation D1 and Table D2 from AS4678-2002 shown below in Equation 1 and Figure 1 respectively.

$$\emptyset' = 30 + k_A + k_B + k_C \text{……………………...….(Equation 1)}$$

Angularity (see Note 1)		**(k_A) (degrees)**
	Rounded	0
	Sub-angular	2
	Angular	4
Grading of soil (see Note 2 and Note 3		**(k_B) (degrees)**
	Uniform	0
	Moderate grading	2
	Well graded	4
N' (below 300 mm) (see Note 4)		**(k_C) (degrees)**
	<10	0
	20	2
	40	6
	60	9

Figure 1: φ′ For Siliceous Sands and Gravels (AS4678-2002, Table D2)

To determine the friction angle of clays, Figure D1 from AS4678 (refer Figure 2) can be used which is based on the plasticity index of the soil. It is important to note, the term 'residual' in this graph refers to the state of the soil, not residual clay. In addition, 'remoulded' clay would be appropriate to use for fill material for example. Generally, it would be appropriate to use the undisturbed clays trend line. The plasticity index is based on the logged material plasticity, which has been verified by an experienced geotechnical professional.

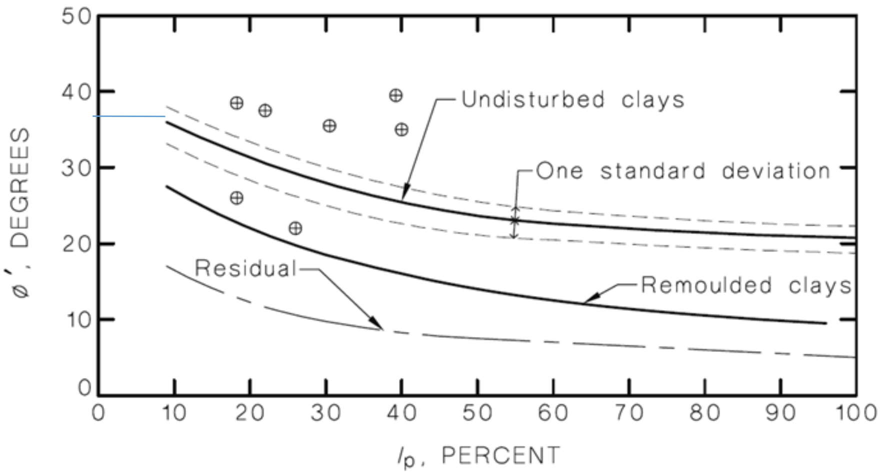

Figure 2: Correlation between φ' and Plasticity Index I_p for normally consolidated (including marine) clays (Figure D1, AS4678)

It is evident that varying plasticity indexes can lead to significant variance in the resulting friction angle. For example, a plasticity index of 10%, 20% and 30% correlates to a friction angle of approximately 35 degrees, 31 degrees and 28 degrees respectively. The impact of the friction angle from the plasticity of the soil can be assessed using constant values for the unit weight, undrained shear strength and cohesion as presented in

Table 1. The bearing capacity has been calculated at 2 m bgl under drained and undrained conditions, assuming a 7 m wide square pad footing and the water table at ground surface. As shown in Table 1, the resulting bearing capacity under drained conditions varies significantly ~1000kPa by using a friction angle of 28 degrees and 35 degrees. The bearing capacity equations used for the calculations have been based on Terzaghi's ultimate bearing capacity theory provided below in Equation 2.

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma \text{..................(Equation 2)}$$

Where:

q_u	Ultimate Bearing Capacity (kPa)
c'	Cohesion of soil (kPa)
N_c , N_q , N_γ	Bearing Capacity Constant
q	Overburden stress (kPa)
γ	Unit Weight of soil (kN/m^3)
B	Footing Width (m)

Table 1: Bearing Capacity under Drained and Undrained Conditions with varying friction angle

Material	Depth (m bgl)		γ (kN/m³)	c_u (kPa)	c' (kPa)	*ϕ' (deg)*	Bearing Capacity (kPa) – Undrained Condition	Bearing Capacity (kPa) – Drained Condition
	From	To						
Silty CLAY	0.0	2.0	18.5	60	6	**35**	**445**	**1530**
Silty CLAY	0.0	2.0	18.5	60	6	**31**	**445**	**850**
Silty CLAY	0.0	2.0	18.5	60	6	**28**	**445**	**570**

Unit Weight

The soil unit weight can be based on AS 4678-2002 Table D1 'Unit weights of soils (and similar materials)' as presented in Figure 3. Using this method, the bulk unit weight is dependent on the soil description and consistency/density of the material.

Material	γ_m: moist bulk weight (kN/m^3)		γ_s: saturated bulk weight (kN/m^3)	
	Loose	Dense	Loose	Dense
A—Granular				
Gravel	16.0	18.0	20.0	21.0
Well-graded sand and gravel	19.0	21.0	21.5	23.0
Coarse or medium sand	16.5	18.5	20.0	21.5
Well-graded sand	18.0	21.0	20.5	22.5
Fine or silty sand	17.0	19.0	20.0	21.5
Rock fill	15.0	17.5	19.5	21.0
Brick hardcore	13.0	17.5	16.5	19.0
Slag fill	12.0	15.0	18.0	20.0
Ash fill	6.5	10.0	13.0	15.0
B—Cohesive				
Peat (very variable)	12.0		12.0	
Organic clay	15.0		15.0	
Soft clay	17.0		17.0	
Firm clay	18.0		18.0	
Stiff clay	19.0		19.0	
Hard clay	20.0		20.0	
Stiff or hard glacial clay	21.0		21.0	

Figure 3: Unit Weights of Soils (and similar materials) (AS4678-2002 Table D1)

Undrained Shear Strength

The undrained shear strength can be estimated using tests such as shear vanes, pocket penetrometers and/or SPTs.

Correlations between undrained shear strength (C_u) and SPT 'N' values typically vary between 2N and 8N (Look, 2014); where N is the uncorrected field blow count. In addition, Stroud and Butler suggest 4.5N for plasticity indices (PI) greater than 30% (Craig and Knappett, 2012). When the PI varies from 15% to 30%, Stroud and Butler suggest 4.5N to 6N (Craig and Knappett, 2012). An average value of 5N is often reasonable to be used for design. However, this needs to be used with caution and is not applicable for very soft materials. Figure 4 presents a correlation between soil description, SPT-N blows and clay strength which generally follows C_u=5N (Look 2014).

Material	*Description*	*SPT – N (blows/300 mm)*	*Strength*
Clay	V. Soft	≤2	0–12 kPa
	Soft	2–5	12–25 kPa
	Firm	5–10	25–50 kPa
	Stiff	10–20	50–100 kPa
	V. Stiff	20–40	100–200 kPa
	Hard	>40	>200 kPa

Figure 4: Clay strength from SPT data (Look, 2014)

Pocket penetrometer readings can be taken from push tube samples, and the undrained shear strength can be approximated using the below equation (Equation 3). It is recommended to take the average of three pocket penetrometer readings at the bottom of the push tube to increase confidence in the result.

$$Cu = \frac{PP}{2} \text{…………………(Equation 3)}$$

Where:

Cu	Undrained Shear Strength (kPa)
PP	Pocket Penetrometer Reading (kPa)

Effective Cohesion

Effective cohesion is arguably the most variable and unreliable soil parameter and therefore should be treated accordingly. The effective cohesion (c’) is typically found to be a maximum 25% – 50% of Cu for cohesive soils (Look, 2014). When deciding the effective cohesion parameter to be used, the long term softening of the material and loss of effective cohesion needs to be considered. Typically, a higher friction angle results in a lower c’ and a lower friction angle results in a lower c’ as shown in Figure 5. To be conservative, it is recommended to adopt a c’ which is 10% of Cu and limited to a maximum of 10 kPa. Caution should be taken when using Cu exceeding 10 kPa.

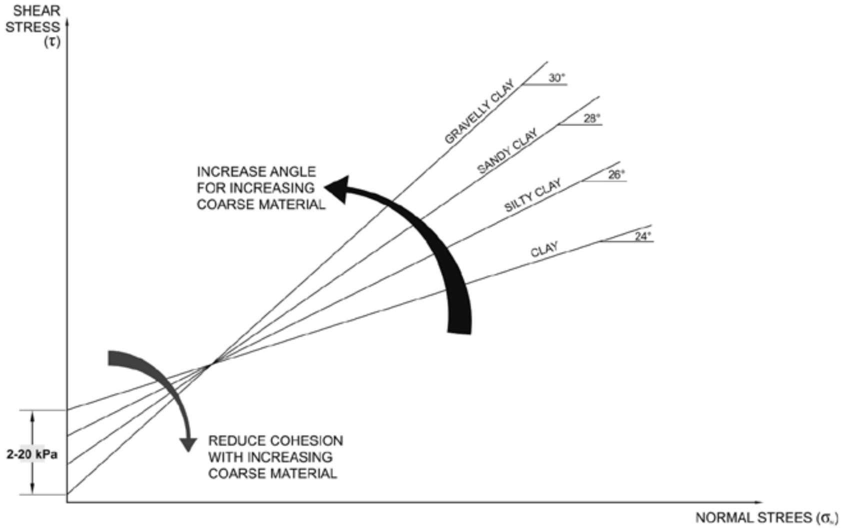

Figure 5: Relationship between cohesion and friction angle (Look, 2014)

Poisson’s Ratio

The recommended value for Poisson’s ratio, for both undrained and drained conditions (υu , υ ’) can be estimated with respect to the logged soil plasticity. Figure 6 presents Poisson’s ratio values for varying plasticity.

Material	*Short term*	*Long term*
Sands, gravels and other cohesionless soils	0.30	0.30
Low PI (<12%)	0.35	0.25
Medium PI (12% < PI < 22%)	0.40	0.30
High PI (22% < PI < 32%)	0.45	0.35
Extremely high PI (PI > 32%)	0.45	0.40

Figure 6: Poisson's ratio for soils (Industrial Floors and Pavements Guidelines, 1999)

Summary

In this paper, soil parameters may be derived using the methods summarised below in Table 2. Alternative methods can be used by the engineer.

Table 2: Soil Parameter Summary

Parameter		Comment	
γ	Unit weight of soil	Based on AS 4678-2002 Table D1 ‘Unit weights of soils (and similar materials)’	
Cu	Undrained shear strength	• Cu=5N	Average value
		• Cu=PP/2	Obtained from field pocket penetrometer. Recommended to use the average of three PP readings.
		• Cu	Obtained from shear vane results
c’	Effective cohesion	• c’=0.1Cu	Conservative equation to reflect the variance in the parameter
ϕ’	Effective friction angle	Estimated from typical friction angle values as per AS4678 ‘Earth-retaining structures’ Equation D1 and Table D2 for non-cohesive soils and Figure D1 for cohesive soils	
υ_u	Poisson’s Ratio (undrained)	Typical values from Table 11.17 (Look, 2014) with respect to soil plasticity	
υ’	Poisson’s Ratio (drained)	Typical values from Table 11.17 (Look, 2014) with respect to soil plasticity	

3.2 ROCK PARAMETERS

Unit Weight

The unit weight of rock can be determined using typical rock unit weights from literature as presented below in Figure 7.

Origin	*Rock type*	*Unit weight range (kN/m^3)*			
	Weathering	*XW*	*DW*	*SW*	*Fr*
Sedimentary	Shale	20–22	21–23	22–24	23–25
	Sandstone	18–21	20–23	22–25	24–26
	Limestone	19–21	21–23	23–25	25–27
Metamorphic	Schist	23–25	24–26	25–27	26–28
	Gneiss	23–26	24–27	26–28	27–29
Igneous	Granite	25–27	26–27	27–28	28–29
	Basalt	20–23	23–26	25–28	27–30

Figure 7: Representative Range of Rock Unit Weight (Look, 2014)

Point Load Strength Index

The estimated Point Load Strength Index, $I_s(50)$, can be interpreted based on the SPT data and/or field observations. Figure 8 provides an example of the Is(50) values.

Table 6.4 Field evaluation of rock strength.

Strength	*Description*			*Approx. SPT N-value*	I_s *(50) (MPa)*
	By hand	*Point of pick*	*Hammer with hand held specimen*		
Extremely low	Easily crumbled in 1 hand	Crumbles		<100	Generally N/A
Very low				60–150	<0.1
Low	Broken into pieces in 1 hand	Deep indentations to 5 mm		100–350	0.1–0.3
Medium	Broken with difficulty in 2 hands	1 mm to 3 mm indentations	Easily broken with light blow (thud)	250–600	0.3–1
High			1 firm blow to break (rings)	500	1–3
Very high			>1 blow to break (rings)	>600	3–10
Extremely high			Many hammer blows to break (rings) – sparks		>10

Figure 8: Rock Material Strength (Look, 2014)

<u>Uniaxial Compressive Strength</u>

The Uniaxial Compressive Strength (UCS) is taken as 20 x Is (50), as presented in resources such as Table 19 in AS1726 (2017). However, in the author's experience, this correlation is generally applicable for high strength rock and should be used with caution when laboratory test results are unavailable and when weaker rock is encountered. For weaker rock, a typical multiplier of 10 is often suitable.

<u>Rock type constant</u>

A summary of how to derive each rock parameter is presented below in Table 3.

Table 3: Rock Parameter Summary

<table>
<tr><th colspan="2">Parameter</th><th colspan="2">Comment</th></tr>
<tr><td>γ</td><td>Unit weight of rock</td><td colspan="2">Based on typical rock unit weights using resources such as Figure 7</td></tr>
<tr><td>Is(50)</td><td>Point Load Strength Index</td><td colspan="2">Based on SPT values and/or field observations using resources such as Figure 8</td></tr>
<tr><td rowspan="2">UCS</td><td rowspan="2">Uniaxial compressive strength</td><td>UCS=20xIs(50)</td><td>As per AS1726 (2017). However, it is recommended to consider the rock condition to determine if this is suitable to use.</td></tr>
<tr><td>UCS=10xIs(50)</td><td>May be more suitable, particularly for weaker rock.</td></tr>
</table>

3.3 PROJECT EXAMPLE

A telecommunication project required design geotechnical parameters without the use of laboratory testing due to budget constraints. The project entailed a single 10 m borehole for the design of the proposed 50 m lattice tower. SPT testing was undertaken every 1.5 m from 1.0 m bgl. Table 4 provides a summary of the verified ground conditions encountered at site.

Table 4: Summary of ground conditions

Origin	Material	Depth (m) bgl		Consistency / Strength[(1)]	SPT 'N' (blows)
		From	To		
Alluvial	Silty SAND (SM) -Poorly graded, rounded	0.0	0.8	Loose	--
Alluvial	Silty CLAY (CH) -High plasticity	0.8	5.0	Firm to Stiff	N = 6 @ 1.0m N = 12 @ 2.5m N = 17 @ 4.0m
Residual	Sandy CLAY (CL) -Low plasticity	5.0	7.0	Hard	N* = 100 @ 5.5m
Residual	Siltstone (considered Silty GRAVEL (GM)))*	7.0	10.0 (TD)	Extremely Low to Very Low Strength	N = 45 @ 7.0m N* = 60 @ 8.5m N* = 69 @ 10.0m

*Notes: **TD** – Termination Depth; N* - Extrapolated SPT N value, [(1)]: Where no in situ testing available for granular material, the strength has been inferred from engineering judgement. *it should be noted that extremely low strength rock is considered to have soil properties and as such has been treated as Silty Gravel*

Using the processes discussed in this paper, the following parameters were determined. It is important to note that the graphs and values provided are a guide, and engineering judgement should always be applied. For example, the sandy clay material and 'silty gravel' were capped at a maximum of 200 kPa for the undrained shear strength.

Table 5: Recommended design geotechnical parameters

Material	Depth (m bgl)		Consistency / Strength	γ (kN/m^3)	c_u (kPa)	c' (kPa)	ϕ' (deg)	υ_u	υ'
	From	To							
Silty SAND	0.0	0.8	Loose	17	-	-	30	0.30	0.30
Silty CLAY	0.8	5.0	Firm to Stiff	18.5	58	6	28	0.45	0.35
Sandy CLAY	5.0	7.0	Hard	20	200	20	32	0.35	0.25
Silty GRAVEL*	7.0	10.1 (TD)	Very Dense	21	200	20	40	0.30	0.30

*Notes: γ: Bulk unit weight; **Cu:** undrained shear strength; **c**': drained cohesion; ϕ': drained friction angle; **Eu**: undrained elastic modulus; **E**: drained elastic modulus; υ_u: undrained Poisson's ratio; υ': drained Poisson's Ratio, **TD**: Termination Depth*:For design purposes the extremely low to very low strength siltstone has been treated as a very dense sandy gravel*

4 CONCLUSION

Geotechnical parameters can be derived without laboratory testing using the visual tactile method and in situ testing data. To ensure the design is not compromised, it is important that adequate training and verification of materials and parameters occurs. Parameters such as the soil friction angle for cohesive soils rely on the logged plasticity and have a significant effect on the resulting bearing capacity. It is important to be mindful of the influence of the parameters and adopt caution (in the form of more conservative parameters) when required. It is also important that over optimistic parameters are not chosen due to poor data.

5 REFERENCES

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