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Shear Strength Anisotropy within an Aged Fill

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ABSTRACT

The shear strength of a material is generally measured via tests orientated in the vertical direction only due to accessibility issues in the field or to minimise the cost of laboratory testing. Unlike deformation parameters, which utilise a Poisson's ratio to account for any differences present in the vertical and horizontal orientations, no typical factors are readily available for shear strength conversion. Industry practice is to instead assume all soil materials are isotropic and utilise factors of safety to allow for any departure from the adopted strength value. However, as some form of anisotropy is inherent in all materials, the blind adoption of the measured vertical shear strength value as a design parameter is potentially unconservative. This paper investigates the shear strength anisotropy of a fill material derived from residual clay. A series of vane shear tests orientated in both the horizontal and vertical direction were completed over a near surface depth profile and evaluated for anisotropic properties and its testing variability.

Keywords: Vane shear, residual fill, soil anisotropy, shear strength, insitu testing variation

1 INTRODUCTION

Insitu tests are generally orientated vertically and a factor subsequently applied to any derived material property to account for differences that may exist between the principal axes or along planes of weakness. In the case of deformation parameters, a Poisson's ratio value is employed to convert the vertical modulus into a horizontal orientated value. The range of Poisson's ratio generally applicable to various soil and rock materials is relatively well defined.

However, no similar ratio or factor is applied to strength parameters, with the material generally accepted to be isotropic. This is rarely the case, especially in residual material profiles where, due to the weathering history of the material, insitu strength measured from vertically orientated tests can be significantly higher than the material's strength as measured by horizontally orientated testing. Similarly, in sedimentary materials, as deposition essentially occurs along a horizontal plane, anisotropic effects are also common. The work described in this paper details an estimate of the magnitude of anisotropy in a residual clay fill material, such as that commonly utilised for large scale earthworks in Queensland.

2 PREVIOUS WORK ON THE INFLUENCE OF ANISOTROPY

Classical stress distribution theory assumes homogeneous and isotropic conditions. Even in critical state theory, the elastic wall assumes soil isotropy. Yet in practice anisotropy may have a considerable effect. For example, Burland et al. (1978) discuss the ratio of initial to total settlement for various soil conditions and showed such a ratio decreased with increasing anisotropy. In situ testing equipment such as dilatometers and pressuremeters provide parameters related to the horizontal direction and these parameters are often applied vertically, via the assumption of isotropy of the soil of interest.

In practice, anisotropic conditions often exist ($E_H \neq E_V$, or $Cu_H \neq Cu_V$) in over-consolidated clays and certain rock types. Soil materials are stiffer in the horizontal direction ($E_H > E_V$), and $E_H = 2E_V$ is a typical assumption adopted in design. This anisotropic condition also affects flow paths and a horizontal permeability $K_H = 2$ to $10K_V$ often occurs.

Parry (1995) summarised the Cu_H / Cu_V values reported by various researchers and showed a range of 0.64 to 0.84 has been reported, with a median value of 0.77. Published data also suggests that a material's anisotropy has little influence on its angle of friction (Mayne, 1985; Parry, 1995). Similarly, Reddy and Srinivasan (1970), in examining the bearing capacity of shallow foundations, reported that soil is anisotropic only with respect to cohesion.

Anisotropic strength properties of soil materials are also part of many software packages (eg SLOPE/W) yet there is little guidance in the determination of this parameter. Lo (1965) showed that the influence of anisotropy is minimal for steep slopes, but becomes significant for flatter slopes. Al-Karni and Al-Shamrani (2000) also found that cohesive anisotropy has a significant effect on the type of slip circle for purely cohesive soil slopes with angles greater than 53^0, but only a minor effect when a soil's friction angle exceeds 10^0.

Given the work completed by others, this research focuses primarily on investigating the cohesive anisotropy of a material, rather than the variation of friction angle.

3 SITE DESCRIPTION

The site upon which vane shear testing was completed was located in the suburb of Chapel Hill, Brisbane, QLD, Australia. The sloping nature of the residential site was such that, prior to the initial dwelling construction in the early 1990's, a cut and fill levelling technique was employed. The natural material encountered onsite was gravelly / sandy clay colluvium overlying weathered phyllite bedrock.

It is understood that during the levelling of the site, residual material was cut from the higher edge of the site and pushed to the lower area until the required platform level was achieved. Accordingly, it was expected that any significant anisotropy inherent within the residual soil material (ie from presence of existing remnants of rock fabric within the soil materials) would have been lost due to material disturbance during the excavation, transport, and placement as a fill. Thus, the fill can be considered to be an essentially isotropic material at the time of placement.

The dwelling at this site was suspended with bored piers and pedestals socketed into the natural ground. Hence, no compaction requirement or Quality Assurance (QA) testing regime was adhered to during fill placement, as would be required in larger earthworks or infrastructure projects. The weight of the relatively small earthmoving equipment provided the only compactive effort. Accordingly, it was expected that some minor anisotropic strength effects may be observed, especially across the upper portion of each lift of fill material, due to the load imparted upon the vertical plane of the fill and the absence of any load imposed on the material in any other direction (ie unconfined in vertical plane). Similarly, compaction due to self-weight settlement over the 20 years since placement would also be expected within the fill material and underlying natural subsurface in the vertical direction.

Characterisation of the subsurface profile of the site was conducted via Dynamic Cone Penetrometer (DCP) tests completed prior to the construction of the existing dwelling in 1991. Additional DCP tests were completed prior to extension of the dwelling in 2010 and also at the time of the vane shear investigation (2011). Based on correlation with existing borehole logs of the original site investigation, Table 1 defines the encountered subsurface of the site. The thickness of the recent fill was interpreted to range between 0.7m and 1.1m from the existing surface. Thus, all investigation points and data analysed were considered to be located within this recently placed fill material.

Table 1: Subsurface present at tested site, historical and current investigations

Material	Origin	DCP 'n' Range (blows / 100mm)		
		1991	**2010**	**2011**
Sandy Gravelly CLAY	Recent Fill (cut to fill, 1991)	NA+	0 – 2	0 – 3
Clayey Sandy GRAVEL	Older Fill / Colluvium (Slope Wash)	1 – 3	3 – 8	3 – 5
Gravelly Clayey SAND	Residual	6 – 16	6 – 9	6 – 9
Sandy CLAY	Residual	7 – 12	> 10	NE
Phyllite	Bedrock	13 – 19 (at rear of site) > 20 (at front of site)	NE*	NE

*NE – Not Encountered; +NA – Not Applicable

Additional laboratory classification of the encountered material was also completed. Particle Size Distribution (PSD), Atterberg limits, linear shrinkage and moisture content tests were completed at regular intervals within the subsurface investigated. Range, average and Coefficient of Variation (CV) values are shown in Table 2, and indicate a Unified Classification System (UCS) designation of Gravelly Sandy Clay would be appropriate for the tested residual fill material.

Table 2: Residual clayey fill material properties

Parameter	Range (%)	Average / CV (%)	Parameter	Range (%)	Average / CV (%)
No. of Samples	4		No. of Samples	3	
Liquid Limit	42 - 50	47 / 9%	Gravel (>2.36mm)	7 - 49	31 / 70%
Plastic Limit	25 - 27	26 / 4%	Sand (>0.075mm, <2.36mm)	28 - 46	31 / 43%
Plasticity Index	17 - 25	21/ 19%			
Linear Shrinkage	7 - 8.5	8 / 8%	Fines (<0.075mm)	31 - 47	38 / 22%

4 TESTING METHODOLOGY

Three (3) test pits were excavated within the fill and residual profile to a depth of up to approximately 500mm. Each test pit was located within 5m of the adjacent pit. At the surface, prior to excavation, vertically orientated vane shear investigations were completed, with the vane located at depths of 100mm and 250mm. Once each pit was excavated to a depth exceeding 250mm, horizontal orientated vane shear investigations were then completed at depths of 100mm and 250mm, thus producing vertical / horizontal pairs from which the anisotropy of the tested material could be determined. This methodology was repeated at a depth of 400mm from the ground surface in one (1) of the test pits.

For each testing location and depth interval, at least five (5) vane shear readings were conducted to ensure any abnormal readings could be identified and characteristic shear strength values determined. In total, 91 shear vane tests were completed in the production of horizontal / vertical pairs at seven (7) specific locations within three (3) spatially separate test pits, with either two (2) or (3) vertically spaced points located in each test pit. A DCP profile was also collected at each test location.

All shear vane testing was undertaken using a 16mm by 32mm vane (Geotest Inspection Vane, Model E-286), and completed in general accordance with the testing procedure detailed within Australian Standard (AS1289.6.2.1).

5 RESULTS

Statistical analysis assuming a normal distribution was conducted for the individual series of vane shear tests completed in each of the three (3) test pits, and are tabulated in Table 3 and Table 4. It is noted that for each Vane Shear test completed, only a single shear strength value was calculated, as per industry standard, with no allowance made for the potential of different shear strengths being present across the various shear planes (Aas, 1967).

The anisotropic ratio values presented in Table 3 and Table 4 have been derived as per the process detailed in AS4133.4.1 for determination of the Anisotropy Index, I_a, of rock materials via use of point load index datasets. This index compares the average determined shear strength values and excludes the maximum and minimum values of the datasets prior to determination of the anisotropic ratio. The ratios can thus be directly compared to one another to examine the variation of the anisotropy ratio across the spatially distributed test locations. In this paper, the ratio is presented as a multiplier comparing the vertical to horizontal value. Thus, a number greater than one (1) indicates the calculated typical horizontal shear strength of the material is greater than the characteristic vertical shear strength value. A number equal to one (1) suggests no bias in shear strength based on test orientation exists, whilst a number below one (1) indicates that the horizontal shear strength of the material is typically lower than the vertical shear strength determined for the same location.

This is a theoretical concept however, and given the calculated CVs which indicate the dispersion of the results encountered at this site, an isotropic ratio value of between 0.8 and 1.2 can be considered

Table 3: *Shear Strength Values calculated from Shear Vane testing, Test Pits 1 & 2*

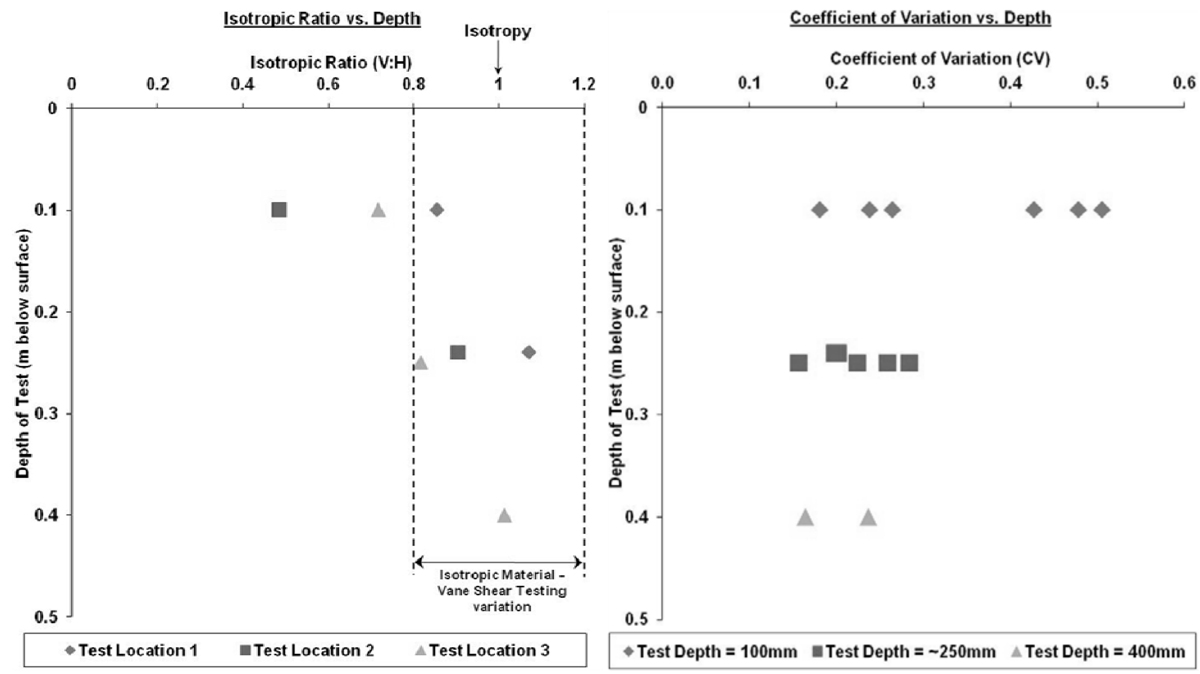
Test Pit ID	**Test Pit 1**				**Test Pit 2**			
Test Depth	**100mm**		**240mm**		**100mm**		**250mm**	
Test Orientation	**Vert.**	**Horiz.**	**Vert.**	**Horiz.**	**Vert.**	**Horiz.**	**Vert.**	**Horiz.**
No. of Readings	6	5	5	6	10	6	12	6
Minimum (kPa)	52	52	86	110	62	16	36	74
Median (kPa)	144	120	132	130	115	55	117	113
Average (kPa)	143	127	131	137	111	53	118	107
Maximum (kPa)	240	220	168	168	150	86	180	132
Coefficient of Variation (%)	50	48	26	16	24	43	28	22
Anisotropic ratio (V_{ave}:H_{ave})	0.85		1.02		0.49		0.90	

Table 4: *Shear Strength Values calculated from Shear Vane testing, Test Pit 3*

Test Pit ID	**Test Pit 3**					
Test Depth	**100mm**		**250mm**		**400mm**	
Test Orientation	**Vert.**	**Horiz.**	**Vert.**	**Horiz.**	**Vert.**	**Horiz.**
No. of Readings	5	5	5	5	10	5
Minimum (kPa)	152	128	168	128	104	140
Median (kPa)	236	164	200	160	169	162
Average (kPa)	220	174	206	163	160	176
Maximum (kPa)	248	236	270	204	184	238
Coefficient of Variation (%)	18	26	20	20	16	24
Anisotropic ratio (V_{ave}:H_{ave})	0.72		0.82		1.01	

approximately equivalent to an isotropic material (ie the variation within tests needs to also be considered). The selection of such a range is consistent with data published by Phoon and Kulhawy (1999) who suggested that for shear strength determination via the use of vane shear testing, a CV of between 10 and 40% could be attributed to inherent soil variability (mean = 25%).

The tests completed closest to the surface (100mm depth) show the highest anisotropic nature, and in each case the vertical shear strength is greater than the horizontal shear strength of the fill material (Figure 1a). If a calculated isotropic ratio of 0.8 is accepted to represent an isotropic material, only two (2) of the three (3) tests completed at 100mm depth demonstrate an anisotropic property (Figure 1a).



(a) (b)

Figure 1: (a) Calculated isotropic ratio (horizontal / vertical) of shear vane results plotted against depth of test; and (b) Coefficient of variation (CV) of shear vane results plotted against depth of test

As the depth from surface increases the tested material approaches isotropy, with the magnitude of the anisotropic nature of the material decreasing. In Test Pit 1, isotropy is observed once a depth of 240mm is reached, whilst Test Pit 3 observed isotropy at a depth of 400mm. Although Test Pit 2 did not find a level at which isotropy was encountered, extrapolation of the available test results suggests that it would be achieved at a depth of approximately 280mm from the surface.

The CV has been plotted for each test location against the test depth in Figure 1b. The CV value for each dataset was 0.5 or below, indicating at all testing points had low variance datasets. The majority of datasets returned relatively closely grouped CV values, of between 0.15 and 0.30. The exceptions to this group were both test orientations completed at 100mm depth in Test Pit 1 (both of which had a calculated CV of approximately 0.5) and the horizontally orientated test at a depth of 100mm in Test Pit 2. This suggests that there is a higher overall variation of data at the near surface tests when compared to the tests completed at locations deeper below the existing surface.

Whereas Tables 3 and 4 presented the I_a value based on the average value of each compiled dataset, Table 5 details the anisotropic ratio calculated based on other basic statistics. The I_a ratios have again been calculated as per the procedure outlined in AS4133.4.1, and thus both the extreme upper and lower shear strength values have been excluded. Accordingly, this filtering may influence the extremity I_a values (minimum and maximum) presented in this range of statistics.

Table 5: Anisotropic Ratio (I_a) calculated from comparison of basic statistics, all test pits

Test Pit ID	Test Depth	Anisotropic ratio of shear strength, I_a (V:H)					
		Min.	Lower Quart.	Median	Average	Upper Quart.	Max.
1	**100mm**	1.42	0.97	0.83	0.85	0.76	0.67
	240mm	1.16	1.06	0.98	1.02	0.96	1.00
2	**100mm**	0.60	0.47	0.48	0.49	0.49	0.43
	240mm	0.77	0.92	0.97	0.90	0.90	0.94
3	**100mm**	0.62	0.66	0.69	0.72	0.76	0.83
	250mm	0.79	0.80	0.80	0.82	0.83	0.85
	400mm	1.15	0.94	0.96	1.01	1.02	1.09

The I_a values calculated using the average of the filtered datasets is largely comparable to the I_a values determined using other statistical values. It is found that if the $I_{a(average)}$ value indicates either an isotropic ($0.8 \le I_a \le 1.2$) or anisotropic material the I_a values calculated using median or either of the quartile values will also indicate the same. The exception to this is the I_a determined by the upper quartile value for the 100mm deep tests in Test Pit 1, which falls just below the 0.8 threshold (0.76). Greater variability is also observed within the I_a values calculated from maximum and minimum values, where, at one (1) location in each of the three (3) test pits, these values indicate that an anisotropic material exists, a result which appears to contradict the I_a value calculated based on alternate statistical calculations (Test Pit 1 – 100mm, Test Pit 2- 240mm and Test Pit 3 – 250mm).

Both the interquartile and absolute anisotropic ratio range, as well as the calculated CV, is greatest in the tests completed at a depth of 100mm. I_a values then tend towards isotropy as the test depth is increased. This again indicates that the material tested is most variable at the upper levels of the fill, suggesting that the nominal equipment traffic at this site has produced a region of anisotropic material.

6 DISCUSSION

The vane shear test results indicate that an anisotropic environment was only present within the upper areas of the tested profile. The median values of the vertical shear strength of the test completed at 100mm depth appears to be up to 1.18 times the vertical shear strength of the tests completed at a depth of 250mm. This suggests that the compactive effect of the equipment used to complete the onsite filling operations extended to a depth of between 100 and 250mm below the existing ground surface. Based on the type and thickness of the fill, this is considered feasible for the earthmoving equipment utilised (anecdotally indicated to be a 14T tracked crawler (‘Drott’) loader). This depth also favourably compares to information published in BRE458 (2003) and Forsblad (1981), where the maximum penetration of an equivalent compactive effort was estimated to be 130mm and 150mm respectively. The US Army Field Manual (FM5-410, 1992) also indicates the appropriate lift thickness to ensure complete layer compaction for CL classified soils via rollers of various weights would range between 100 and 225mm.

Onitsuka et al. (1985) found, after completing direct shear tests (60 x 20mm sized samples) on four (4) types of disturbed residual granite material (classified as SM), the tested material was between 1.1 and 1.2 times stronger when sampled perpendicular to the orientation of sample compaction. In comparison, a similar strength difference of between 1.2 and 1.5 was noted in the tests completed in the compacted material (tests completed at 100mm depth) identified by this study.

Previous work has not normally attempted to characterise the upper reaches of soil profiles, as investigated by this study. However, the implications of the presented results (whereby a compactive effort can produce a region of significant anisotropy within placed fill materials) is that once extrapolated to projects that contain larger embankments, constructed via a series of lifts and compacted via the use of heavy plant and rollers, the difference in calculated shear strength values based on testing orientation could result in unrepresentative values being adopted for design purposes and/or QA control. As all investigation and compaction QA tests are generally completed via vertically orientated tests, the results of these tests would only ensure conformance of the vertical shear strength component of the tested material. Furthermore, as certain failure mechanisms are dependent on both vertical and horizontal strength components (eg slip surfaces contained fully within an embankment), the adoption of parameters derived solely from vertically orientated tests would likely result in the overestimation of the overall strength and/or stability of the construction. Similarly, adoption of shear strength parameters derived from vertical orientated testing for design of horizontally orientated reinforcement elements (eg pullout straps for RSS walls) would also lead to overestimation of the insitu capacity of a structure.

Another key finding relates to the use of the simple DCP test as a profiling tool. A blow count (n) within a soft to firm clay is typically considered to produce a value of 3 (or lower) for 100mm rod penetration. Yet the reverse relationship does not apply, and an ‘n’ value of less than 3 does not necessarily indicate the presence of a soft to firm clay; in this instance the material is considered a very stiff clay.

7 CONCLUSIONS

A series of vane shear tests, orientated both vertically and horizontally, were completed at various depths within a residual gravelly sandy clay fill. From these results, it was identified that the top region of the fill was anisotropic in nature whilst an essentially isotropic material was encountered at depth. It was interpreted that such a profile was due to the use of earthworks equipment at the time of filling.

Within the identified anisotropic region, vertically orientated tests resulted in larger shear strength values than corresponding horizontally orientated tests and increased shear strength variation within results was displayed. Implications relating to the adoption of shear strength values derived solely from vertically orientated vane shear tests for design have then been considered.

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