Pahoia Tephra Sequence: strength, sensitivity, and stabilisation

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Abstract

Large, retrogressive landslides with long runouts have been attributed to the sensitive nature of quick-clay soils. In the Northern hemisphere remediation attempts have been made with chloride salt solutions. The use of potassium chloride salt has been shown to significantly reduce sensitivity *in situ* and to strengthen quick clays dominated by illite to a point where they can be regarded as stable. In Bay of Plenty, New Zealand, the Pahoia Tephra Sequence (PTS) has been identified as a sensitive soil layer, dominated by the 1:1 clay halloysite. This paper examines previously published data, together with fresh samples from the base of a recent large landslide using consolidated undrained triaxial strength testing. Initial results indicate a highly sensitive, weak layer at the base of the PTS, near the interface with the underlying Kidnappers Ignimbrite. The possibility of *in situ* stabilisation was investigated at a laboratory scale through submersion of samples in 2 molL-1 potassium acetate solution. Submersion resulted in increased shear strength and an overall reduction in sensitivity. These results are attributed to interaction between the potassium ions with the dominant halloysite clays, suggesting that there may be a method to universally increase shear strength in the sensitive soil *in situ*.

Keywords: landslide, sensitive, volcanic, slope-modelling, soil-treatment, halloysite.

1. Introduction

Large, retrogressive landslides with long runouts sourced from quick clays are a worldwide phenomenon. These events cause significant damage to infrastructure, and in some cases loss of life, and many have been directly attributed to the presence of sensitive soils (Moum *et al.*, 1968; Tavenas *et al.*, 1971). In the Northern Hemisphere it has been found that some sensitive clay soils derive from uplifted glacial outwash deposits. The leaching of sodium from glacial outwash by meteoric water causes a re-alignment of a 'house of cards' structure between the clay minerals, resulting in a drastic loss of the soil's strength upon remoulding (Locat *et al.*, 2011; Skempton & Northey, 1952).

Due to the unpredictable and highly destructive nature of quick clay slides, attempts to remediate their sensitivity have been undertaken, particularly in Norway, and to a limited extent in Italy (De Rosa *et al.*, 2016). Amelioration attempts of failures in Norway date back to the 1950's, with sodium chloride salt mixed into runout deposit debris of a large failure to strengthen and stabilise the soil (Eide & Bjerrum, 1955; Torrance, 2014). Subsequent testing in the 1960's and further re-investigation in more recent years has shown success in the use of potassium chloride salt wells to reduce sensitivity *in situ* and to strengthen illite-dominated quick-clays (Helle *et al.*, 2017; Helle *et al.*, 2015; Moum *et al.*, 1968).

New Zealand sensitive soils are generally derived from volcanic deposits in the form of ignimbrites, volcanic tuffs, and weathered or reworked tephras (Torrance, 1983). Tephra deposits with pyroclastic materials of any grain size are highly susceptible to failure, and in several cases have been responsible for catastrophic landslides (Chigira, 2014; Sidle & Ochiai, 2006). Landslides with long runouts are well documented in the North Island's Bay of Plenty region (BOP), with the Pahoia Tephra Sequence (PTS), being a significant contributor (Moon, 2016; Moon *et al.*, 2015). BOP soil sensitivity is thought to derive from the presence of large quantities of spheroidal 1:1 halloysite clay minerals (Smalley *et al.*, 1980). The minerals exhibit short range electrostatic van der Waals interactions, which upon failure and reworking cause a detachment on the clay surfaces. Edge-face charge imbalances result in an overall repulsive behaviour by the clay and leads to the large flow like nature of these deposits (Kluger *et al.*, 2017).

Halloysite, unlike illite does not preferentially strengthen with the addition of either potassium or sodium chloride salts. However, numerous studies have shown that there are distinctive changes made to the halloysite structure following the introduction of potassium acetate (K-acetate) (Garrett & Walker, 1959; Theng & Wells, 1995). These studies report that K-acetate enters the halloysite crystal lattice to form a stronger intercalate and expands the clay layers.

This paper first examines previously published data on the PTS (Wesley 2007, Mills 2016) through comparison of effective cohesion and friction angles of the soil at different BOP sites. Original results for a newly sampled layer found near to the base of the Pahoia Tephra unit, close to the interface with the Kidnappers Ignimbrite at the Kowhai Grove landslide, Ōmokoroa, are then presented. Samples were tested under consolidated undrained (CU) triaxial conditions. Finally, these data were compared with samples immersed for 1 month in a 2 $\mathrm{MolL^{-1}}$ potassium acetate solution in order to observe potential strength changes of the soil over time.

2. Materials and Methods

2.1 Materials – Geological Setting

The Tauranga basin, located within the BOP, is covered with a series of thick tephra and ignimbrite layers deposited within the last 3 Ma (Briggs *et al.*, 1996). Throughout the basin there are many north-northeast tending peninsulas that extend into the Tauranga harbour. These peninsulas range from 20-40 m in elevation, exhibit steep coastal slopes or cliffs, and are known to be prone to landslides. The stratigraphy is comprised of Matua Subgroup deposits (c. 2 Ma – 0.35 Ma) which consist of primary and reworked pyroclastic deposits, most notably the Pahoia Tephra Sequence (PTS c. 2.18 - 0.35 Ma), intercalated with non-welded ignimbrites (Kidnappers and Te Ranga Ignimbrites). The Matua Subgroup is usually overlain by younger airfall tephra deposits including the Hamilton Ashes (0.08 – 0.38 Ma), Rotoehu Ash and Post Rotoehu tephras (< 60,0000 years). Unit thicknesses and deposit characteristics, particularly of the Matua Subgroup materials, vary considerably throughout the Tauranga basin.

The PTS is spatially variable both horizontally and laterally, with materials within split informally into two main units (Kluger et al., 2017). The upper portion is generally made up of silty sand and sand beds (Briggs *et al.*, 1996; Oliver, 1997), while the lower unit is made up of thick halloysite-rich clay, silt, and silty clay beds. These clay-rich beds are a key contributor to instability; they generally exhibit large void ratios, very low permeability, and varying sensitivity with depth, with sensitivity ranging from 10-140 (Gulliver & Houghton, 1980; Mills & Moon, 2016).

2.2 Methods

Soil samples were collected from the Kowhai Grove landslide scarp, Ōmokoroa (see Figure 1). Determination of the layer with highest sensitivity was achieved by shear vane testing of each distinctive soil change with depth in a pit dug approximately 4 m in front of the scarp face. A highly sensitive ($S_t = 28.5$) layer was identified around 750 mm below the ground surface. Samples were extracted using 50 x 150 mm stainless steel push tubes as opposed to block sampling due to challenges with sample location and depth.

Water content (NMC) was determined in line with NZ standards NZ4402:1986 Test 2.1, and bulk density (ρ_d) in line with ISO 17892-2(2004). Particle density (ρ_s) was measured via a gas pycnometer using ASTM D5550-14, this was then used to determine both porosity (n) and void ratio via equations (1) (Carey *et al.*, 1996) and (2) (Jamiolkowski *et al.*, 1995).

$$Porosity(n) = \frac{dry\ bulk\ density\ (kgm^{-3})}{particle\ density} \times 100 \tag{1}$$

$$e = \frac{V_v\ (Volume\ voids)}{V\ (volume)} = \frac{n}{1-n} \tag{2}$$

Treatment of soil was achieved through total immersion of 50 mm x 150 mm soil cores in 2 molL^{-1} K- acetate solution for a period of 1 month. Note that NWC, ρ_d, ρ_s and Atterberg limit values for treated soil have been corrected to account for the presence of salt within the soil. Triaxial tests were conducted in accordance with BS 1377-8. Confining stresses were set at points 20 and 40% lower than calculated *in situ*, pre-failure confining stress, in order to model the threshold failure point of 36% reduction in effective stress determined by (Kluger *et al.*, 2020). Effective cohesion and friction angle values have been compared with values derived by Mills (2016) and Wesley (2007) for PTS samples.

3. Results

3.1 Geomechanical properties

General geomechanical properties of the study samples are displayed in Table 1. Porosity, void ratio, and Atterberg limits for the untreated soil materials were all high which are representative of a clayey-silt with little sand, in keeping with previously published research. NWC for our untreated soil was in excess of the Liquid Limit, in keeping with soils of a sensitive nature both in New Zealand and overseas. Particle densities were higher than previous studies, the cause of which will likely require further examination through XRD testing. Shear vane measurements placed the soil within the realms of 'quick-clay' (57/2 kPa) , though it should be noted that remoulded field shear vane produced strengths higher than the 0.5 kPa required to deem a clay 'quick' in international literature (Torrance, 1983).

Table 1.Geomechanical properties of both treated (2molL^{-1} K-acetate for 1 month) and untreated soil sampled from Kowhai Grove field site. Errors were determined to one sd.

	Untreated	**Treated**
FS (kPa)	57	N/A
RFS (kPa)	2	N/A
S_t	28.5	N/A
NWC, (%)	67.3 ± 1	57 ± 0.4
ρ_d (kg m^{-3})	1576 ± 5	1628 ± 19
porosity (n) (%)	71 ± 0.1	63 ± 0.3
void ratio	2.48	1.75
ρ_s (kg m^{-3})	3282	2847
fines (<63 μm) (%)	98.4	98.6
clay (<2 μm) (%)	39 ± 8	40.2 ± 5
Liquid Limit, LL (%)	58.6 (R^2=0.99)	N/A
Plastic limit, PL (%)	45.9 ± 0.21	N/A
Plasticity index, PI (%)	12.79	N/A
Liquidity index, LI (%)	1.71	N/A
Activity, A	0.28	N/A

FS= Undisturbed field shearvane; RFS= Remolded field shear vane; St=sensitivity (FS/RFS)

3.2 Triaxial Results

Triaxial results including deviator and pore pressure paths, stress paths for untreated and 1 month treated soils are presented in Table 2 and Figure 1. For untreated soil (Fig 1A), as confining stress increased peak deviator stress similarly increased. Pore pressures at all confining stresses exhibited initial peaks prior to peak deviator stress. These were followed by slight declines in pore pressure as samples reached peak deviator stress. Upon entering into the post peak phase, deviator stresses declined noticeably. Pore pressures when entering into the post peak phase, after the slight decrease following peak deviator stress, increased when heading towards the residual, with the sample at 225 kPa confining stress showing an immediate increase in pore pressure following failure.

Table 2. Consolidated undrained triaxial results for untreated and treated samples. Note confining stresses for untreated samples are different from treated due to correction of overburden stresses following testing of untreated samples.

Sample	**ECP (kPa)^a**	**ε_f (%)^b**	**Q_f (kPa)^c**	**U_f (kPa)^d**	**SS (%)^e**	**c’ (kPa)**	**Φ’ $(^0)$**
Un-1	120	10.3	134.88	50.3	19.3		
Un-2	150	4.6	149.02	87.7	31.6		
Un-3	225	3.5	184.96	130.3	36.9	4.2	29.8
Tr-1	120	4.9	171.68	62	21.1		
Tr-2	160	3.3	201.85	86.8	31.5		
Tr-3	200	9.1	223.8	110.7	10	21.5	27.8
Mills & Moon 2016 (Ōmokoroa) (Un)						26	31
Mills & Moon 2016 (Matua) (Un)						17	32
Wesley 2007 (Un)						10	35

ECP= Effective Confining Pressure; εf=axial strain at failure; Qf=deviator stress at failure; Uf=Pore Pressure at failure; SS=strain softening; c’=effective cohesion; Φ’ effective friction angle; Un=untreated soil; Tr=treated soil

Treated samples followed similar trends (Fig 1B), with declines in deviator stress and slight increases in pore pressure when heading towards a residual. Pore pressure at peak deviator stress varied with differing confining stresses. At the lowest confining stress treated samples were ~12 kPa higher treated over untreated, middle confining stresses ~1 kPa lower and approximately 20 kPa lower at the highest confining stress. It should be noted though that the middle confining stress was 10kPa higher for treated samples and 25 kPa lower for highest confining stress which has likely impacted the results slightly. Added to this is the much smaller increase in pore pressures following failure at the highest confining stress (32 kPa increase for untreated, 6 kPa increases for treated).

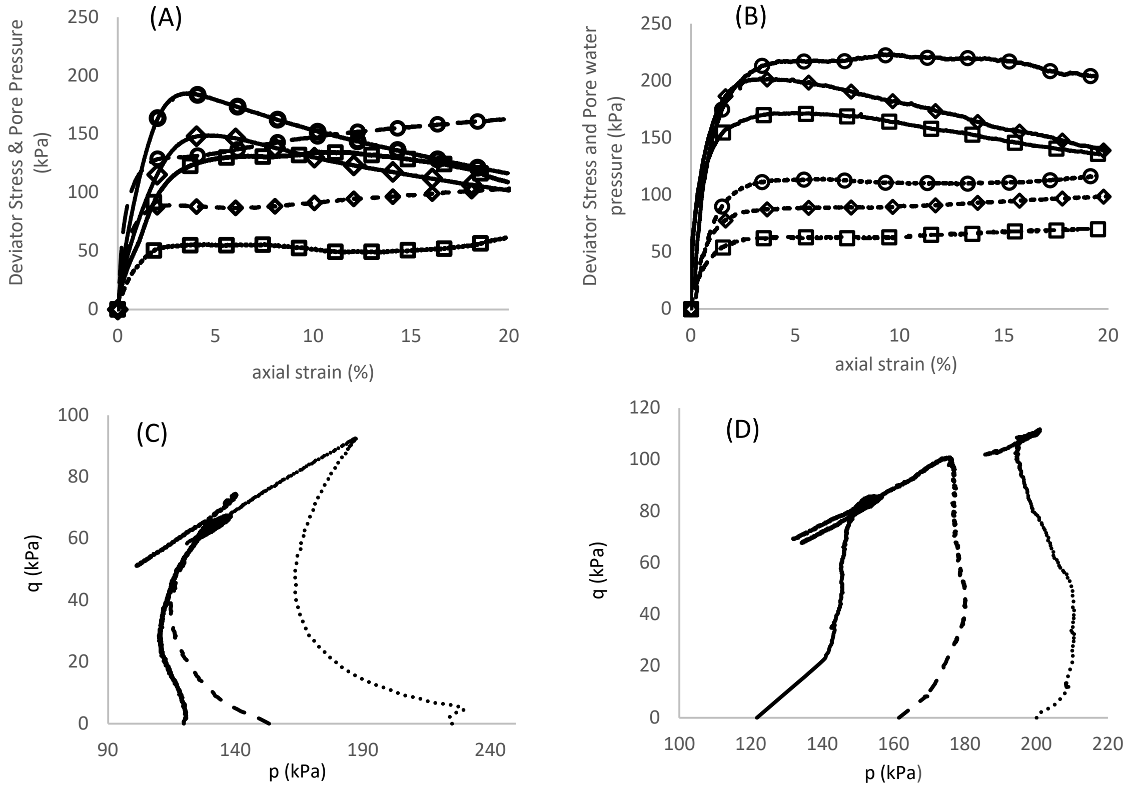

Figure 1: (A) Stress/strain (solid Lines) & Pore water/strain curves (dashed lines) for samples of untreated PTS soil at differing confining pressures. (ECP- 120=Squares, 150=diamond, 225=circle). (B) Stress/strain (solid Lines) & Pore water/strain curves (dashed lines) for samples of treated PTS soil at differing confining pressures. (ECP- 120=Squares, 160=diamond, 200=circle). (C) Stress paths for untreated PTS soil (Solid line=120, dashed line=150, dots=225). (D) Stress paths for untreated PTS soil (Solid line=120, dashed line=160, dots=200).

One key difference was for the treated sample tested at the highest confining pressure (200 kPa). This sample had a notable reduction in strain softening as a potential result. Stress paths (Figure 1C,D) exhibited noticeable trends, with untreated samples all exhibiting an initial contraction during loading, before strongly dilating prior to reaching the CSL (Critical State Line), though once all samples reached the CSL, following failure, they proceeded to contract in the strain softening phase. Treated samples, by contrast, showed mostly straight stress paths, tending slightly to the right before proceeding left along the CSL upon failure. Untreated soil strength, c' and Φ' (Table 2), was considerably lower than those observed in Mills' 2016 study, and plotted much closer to the results of Wesley (2007). Treatment had a considerable effect on the soil strength, increasing cohesion by 17 kPa.

4. Discussion

It is vital that cohesion and friction angle are determined accurately as they are the two key parameters used in slope stability modelling and assessment. While changes in friction angle can be negligible, changes in cohesion can impact Factor of Safety (FoS) quite significantly. Some recent studies suggest an increase in cohesion of 8 kPa (8 kPa – 16 kPa) can result in increases in FoS by values of up to 0.7 in Quaternary gravels with high proportions of silt and clay, for a 4 m slope at an inclination of 1:1.25 (Harabinová & Panulinová, 2020). In the present study friction angles were somewhat lower (Un = 30° and Tr = 28°) than compared with previous studies (31 - 35°). However, our results found a significant variation in cohesion of our untreated samples (6 kPa) compared with previous studies (26 kPa).

These findings suggest that in slopes and areas where Pahoia Tephras may be present, care needs to be taken when modelling the unit. Current methods of reliance on pre-existing data, as well as regarding the PTS as a single unit may need to be reconsidered. When comparing the limited published data, it becomes clear that the Mohr-Coulomb parameters for PTS vary from site to site. As such, when creating slope stability models in future on sites where the PTS is likely to be present a series of key considerations need to be made. Firstly, if PTS is present, it is advisable to split the PTS, at the very least, into its constituent upper and lower portions to ensure the differences in two soil bodies are noted. Secondly, the values applied to these soil units need to be unique for each site. As c' and Φ' values vary markedly with location, reliance on pre-existing data for sites is likely not appropriate. Steps such as physical testing either *in situ* (i.e., CPT, Geonor vane testing) or within a laboratory setting should be taken. These steps would allow modelling of a much greater accuracy to be created and allow for a much more accurate and representative value of FoS be produced.

Examining the triaxial values it is clear that treatment of the soil with a 2 $molL^{-1}$ solution of potassium acetate has had a marked impact on both overall soil strength and behaviour. All samples exhibited classic post failure contractive, strain softening responses, in keeping with sensitive material tested both in New Zealand and overseas (Gylland *et al.*, 2014; Mills & Moon, 2016). However, stress paths show a clear deviation in initial loading paths between samples with pre-failure contraction then dilation in untreated soils being replaced with almost vertical paths before reaching the CSL following treatment. For treated soil, at 200 kPa there was a considerable reduction in the degree of contraction following failure of the sample, indicating a potential behaviour change of the soil at higher confining stresses. Pore pressure variations between treated and untreated soils do need to be noted, with treated soil at least at the lowest confining pressure exhibiting a much higher pore pressure at failure compared with untreated, and pore pressures at the higher confining stresses likely only lower due to the difference in confining stress. Overall strength increases are likely due to the favourable intercalation of the potassium acetate ions into the basal spaces within the halloysite, causing an expansion in the clay unit from 10 Å to 13 – 15 Å (Garrett & Walker, 1959). Furthermore, an increase in pH of the soil due to the strong alkaline nature of potassium acetate has likely caused the point zero charge (PZC) on the clay surface to shift to being negatively charged (Mitchell & Soga, 2005; Theng & Wells, 1995). This will result in much stronger van der Waals bonds between the clay spheroids, producing the significant increases in shear strength and effective cohesion we observe.

5. Conclusions

The spatial variability of the Pahoia tephra within the Tauranga basin, as well as its propensity for containing a variety of layers of significant and varying sensitivity makes it a challenge for engineers to model appropriately. With the drastic variation in c' values measured across 5 differing sites in Tauranga consideration needs to be made when creating slope stability models as to the way in which the PTS is treated. This study suggests that it may, in a number of cases, be more appropriate to break the PTS up into at the very least the upper and lower portion due to the vast differences even within the PTS at a single site. With this said, promising progress has been made on a novel method to potentially improve, strengthen and stabilise highly sensitive layers within the PTS through the use of a potassium acetate solution, with drastic changes in both cohesion and behaviour being noted following treatment.

6. References

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