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

Static failure mechanisms in sensitive volcanic soils in the Tauranga Region, New Zealand

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ABSTRACT

Sensitive soils derived from weathered rhyolitic volcanic materials have contributed to major landslides in the Bay of Plenty. Soil was sampled from sensitive layers near the failure surface of two coastal landslides bordering Tauranga Harbour. Methods were adapted from Gylland et al. (2014) including undrained, consolidated static triaxial tests at a high compression rate of 0.5 mm/min in order to replicate rapid loading during landsliding. Like sensitive materials derived from glacial till, sensitive volcanic derived material showed contractive p-q plots, strain softening stress- strain behaviour, coupled with rising pore pressures, and single or double shear band formation after peak strength was reached. Little to no cohesion or friction softening occurred between peak and residual states. This evidence indicates that like sensitive soils derived from glacial till, the low permeability of the clay allows pore pressure gradients to evolve, eventually initiating collapse of clay microstructures into shear zones, where further excess pore pressure generation within the shear zone instigates progressive failure. Shear zone development causes a rapid loss of global resistance, expressed as strain softening behaviour.

Keywords: landslide, sensitive, volcanic, shear zone, strain localisation, strain softening

1 INTRODUCTION

1.1 Problem background

It is well documented that sensitive soil failures in Tauranga occur after heavy, prolonged periods of rainfall (Moon et al., 2015a & b). The fundamental failure mechanisms relating to rising pore pressures during heavy rainfall has not been studied for these materials. The strain softening failure properties of these soils are important to understand for accurate slope stability modelling (Gylland et al., 2014; Thakur et al., 2011). Several notable large landslides attributed to sensitive soils in the Tauranga Region include (1) the Ruahihi Canal failure in 1981, (2) a significant coastal cliff collapse at Bramley Drive in 1979 (Gulliver & Houghton, 1980), and (3) numerous landslides in the Tauranga city margins after heavy rainfall in 2005.

1.2 Literature review

Sensitive soils derived from both glacial till (Bjerrum, 1955; Bernander, 2000; Locat et al., 2011; and Gylland et al. 2013), and also from weathered volcanic material (Wyatt, 2009; Cunningham, 2012) show brittle failure and a reduction in the shearing resistance following peak strength, known as strain softening (Taylor, 1937; Skempton, 1964). Recently, at a microstructural level, brittle failure and strain softening in glacial till derived sensitive materials in the northern hemisphere has been attributed to excess pore pressure generation in localised shear bands (Thakur, 2007; Thakur, 2011; Gylland et al., 2013; and Gylland et al., 2014) rather than a reduction of c’ and φ’.

Recent evidence indicates that at both a macroscopic level (Skempton, 1964; Urciuoli, 2007; and Quinn et al., 2011), and microscopic level (Thakur, 2007; Thakur, 2011; Gylland et al., 2013; and Gylland et al., 2014), progressive failure, where strain softening in one soil element results in failure and strain softening in its neighbouring soil element, like falling dominoes, is a dominant process governing failure of sensitive soils. At the macroscopic level, three progressive failure mechanisms have been described: (i) upwards progressive failure, wherein a perturbation at a cut slope propagates upwards into the slope, (ii) downwards progressive failure, where a load further back from a free face, for example pile driving, initiates shear band propagation, and (iii) widespread liquefaction of the sensitive layer. Recently, fracture mechanics principles (Rice, 1968; Palmer, 1973) have been applied to progressive (stable), and catastrophic (unstable) growth of shear bands in sensitive soils at a macroscopic scale (Quinn et al., 2011). The high liquidity index, saturation, and void ratios of sensitive material mean that a significant amount of water is released during failure, allowing the soil to transport overlying material long distances (Torrance, 2014).

During undrained compression of glacial till sensitive soils at a microscopic scale, initial strain localisation and the following strain softening, contractive response observed has been attributed to excess pore pressure generation (Thakur, 2007; Thakur, 2011; Gylland et al., 2013; and Gylland et al., 2014). Low material permeability combined with high loading rates result in pore pressure gradients, leading to localised strain and further excess pore pressure generation due to contractant microstructure within a shear band (Thakur, 2007; Thakur, 2011; Gylland et al., 2013; and Gylland et al., 2014). All plastic straining occurs within the shear band following its formation.

1.3 Aim

In this study we aim to answer whether sensitive volcanic material soils bear similar failure characteristics to glacial till sensitive soils, by relating lab observations to field observations. We studied two coastal cliff landslide sites in the Tauranga Region where sensitivity is believed to contribute to failure: (i) a significant landslide at Bramley Drive, Omokoroa, which has been the subject of ongoing research by the University of Waikato and University of Bremen, and (ii) a smaller landslide, initially investigated by Coffey Geotechnics in 2012, on the south side of Matua Peninsula within Tauranga City urban area.

2 METHODS

Geomorphic maps were drawn over aerial photographs overlain with contours during a site walkover (WBOPDC, 2014). Sampling sites of the sensitive material were chosen based on (a), safe location (b) compliance of property owners and (c) extra sensitive material in the layer suspected to contribute to failure. *In situ* peak and remoulded strength was measured with a Pilcon Geotechnics vane. Stratigraphy was described in accordance with NZGS (2005). Stratigraphic layers at the sampling site at Omokoroa were correlated with a borehole drilled behind the failure scarp in February 2012. At Matua, stratigraphy was correlated with hand auger boreholes drilled in February 2013 (Geotechnics 2013). For triaxial sampling, push tubes were chosen over block samples as the material was too brittle for carving. At each site, a bench was dug horizontally into the sensitive layer, and separate stainless steel push tubes (d=48 mm, h=148 mm) were gently tapped into the dug bench from the vertical direction. Four push tubes of Matua material (M1) were collected and three of Omokoroa material (OM1), as well as bulk samples for geomechanical properties. Field moisture was preserved by thorough plastic wrapping and storage containers. Moisture content, wet and dry bulk densities, voids ratio, and Atterberg limits all followed ISO standards (ISO/TS 17892-1:2004), undrained effective triaxial testing, and cohesion and friction angle methods followed NZS 4402 (1986) with the exception of the compression rate, where a higher rate of 0.5 mm/min was chosen over the testing time recommended, in accordance with Gylland et al. (2013), who justified using a higher rate because undrained loading occurs rapidly during landslides. All samples were saturated to B values greater than 95%. A membrane correction was applied in accordance with NZS 4402 (1986). To determine brittleness or strain softening of the soil, we used Bishops (1971) parameter:

$$\text{Brittleness} = (q_{max} - q_{residual} / q_{max}) \times 100 \tag{1}$$

where q_{max} and $q_{residual}$ are peak and residual deviator stresses respectively.

3 STRATIGRAPHY AND GEOMOROPHOLOGY

3.1 Stratigraphy

The stratigraphy of the steep to gently inclined, roughly NE aligned peninsulas around the Tauranga Harbour margins comprise a sequence of locally and distally sourced, reworked and *in situ* volcanogenic sediments (Briggs, 1996). Problem-causing sensitive layers occur within the Matua Subgroup (60 ka – 2.09 Ma), a variable fluvial sequence consisting mainly of reworked pyroclastics. The Pahoia Tephras (0.35 – 2.18 Ma), a series of air fall tephras and ignimbrites, are intercalated with the Matua Subgroup (Briggs, 1996). Overlying the Matua Subgroup are younger airfall tephra sequences (Briggs, 1996; Lowe et al., 2001). Stratigraphic profiles and soil sample descriptions at each site are outlined in Figure 1.

3.2 Geomorphology

The Bramley Drive landslide has geomorphic features which are most similar to flow failures (Bernander, 2000; Locat et al., 2011). These features include a largely scarred, empty landslide crater, a large flowslide runout component (~160 m after 2012 event), and to a lesser degree, a narrow landslide neck (Locat et al., 2011). The approximate equidimensional and bowl-shaped scarp are similar to lump landslides (Hungr et al., 2014). Like landslides in glacial till, retrogression has occurred, but over a much longer time scale, and shorter distances (initially ~ 30 m in 1979, then 3 -5 m in 2011-2012) (Moon et al., 2015). The failure scarp of the slide at Matua is shallow relative to the surface, slightly back rotated toward the base (52º) and roughly equidimensional (11 m long, 10 m wide), indicating planar-slight rotational sliding (Hungr et al., 2014). The long runout component (~ 50 m), remoulded nature of the debris, and extra sensitive material near the base of the failure scarp at Matua indicate that sensitivity contributed to failure.

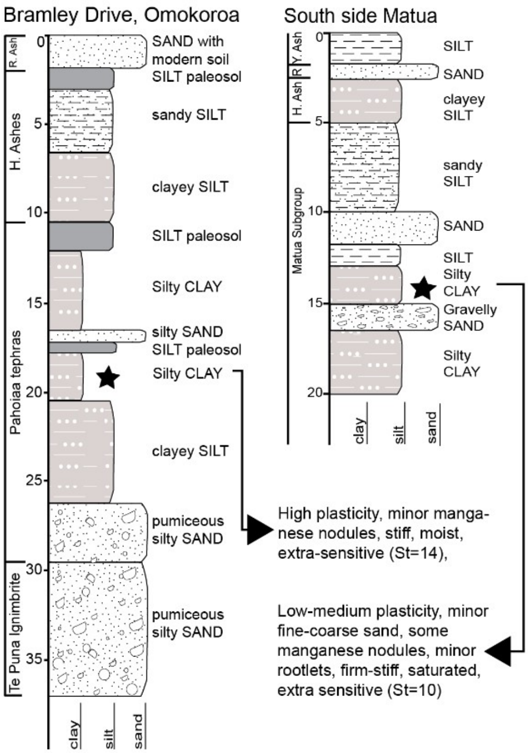

Figure 1. Stratigraphic profiles of Bramley Drive and Matua.

Table 1. Geomechanical properties of soil sampled from Omokoroa and Matua

	Omokoroa ± S. Error	Matua ± S. Error
Peak FS[a] (kPa)	66 ± 3	60 ± 0.4
Remoulded FS (kPa)	5 ± 1	6 ± 0.6
Moisture Content, w (%)	72 ± 1	64 ± 0.3
ρ_d (kg/m^3)[b]	1320 ± 46	1690 ± 98
Porosity, n (%)	70 ± 0	65 ± 3
Void Ratio, e (%)	2.3 ± 0	1.8 ± 0.3
Sensitivity, St (%)	15 ± 3	10 ± 1
ρ_c (kg/m^3)[c]	2517 ± 9.7	2777 ± 257
Clay (%)	63 ± 4	40 ± 11
Silt (%)	37 ± 4	23 ± 7
Sand (%)	0 ± 0	37 ± 17
Liquid Limit (%)	66 (R^2 = 0.99)	52 (R^2 = 0.87)
Plastic Limit (%)	41 ± 0.4	37 ± 2
Plasticity Index (%)	25	15
Liquidity Index (%)	2.9	1.8
Activity (%)	0.4	0.4

[a] FS = Field Vane Strength;
[b] ρ_d = wet bulk density;
[c] ρ_c = Particle Density

4 GEOMECHANICS

4.1 Standard geomechanical properties

Porosity, void ratio, liquidity indices and moisture content are high for Matua (M1) and Omokoroa (OM1) materials, in keeping with previously published research on material derived from halloysite (Table 1) (Wesley, 1977; Wesley, 2009; Wyatt, 2009; Cunningham, 2012; and Moon et al., 2015). Soils from both sites are silty clays, in contrast to most soils reported in previous publications (Keam, 2008; Wyatt, 2009; and Cunningham, 2012), which are silts or clayey silts. SEM scans suggest material from both sites to be halloysite dominated, with an open structure of point-point contacts, small, ubiquitous pore spaces, and clay mineral coatings on larger silt and sand sized grains (Moon et al., 2015). Many, saturated pores accounts for the low wet bulk densities observed in M1 and OM1. Both M1 and OM1 have high Atterberg limits, plotting below the A-line as high compressibility silts, and low activities (Table 1), in line with halloysite dominated sensitive material (Wesley, 2009; Moon et al., 2015).

4.2 Triaxial results

Triaxial results for M1 and OM1 samples are presented in Table 2, for tests a – d. At greater confining stresses, peak deviator stress, curvature of peak, and degree of strain softening increases for both M1 and OM1. Tests where pore pressure rises after peak stress correlate with a contractive, left trending curve along the CSL (critical state line) (c, d), while tests where pore pressure drops following peak stress correlate with curves that touch the CSL and trend slightly to the right. All samples failed along either one sliding plane by shear, or as a wedge, where two sliding planes occur at roughly perpendicular angles. Only sample M1a failed by barrel deformation with very minor shear zones observed. High effective friction and variable effective cohesion values are in the range of previous publications (Keam, 2008; Wyatt, 2009; and Cunningham, 2012). Effective friction angles and cohesion for samples stayed mostly consistent between peak and residual states, albeit minor reductions i.e. little to no friction and cohesion softening occurred.

Table 2. Undrained, consolidated triaxial test results for Omokoroa and Matua

Sample	Test Label	ECP (kPa)[a]	B (%)[b]	TR[c] (mm/min)	ε_f (%)[d]	q_f (kPa)[e]	SS (%)[f]	FM[g]	c'_f[h]	c'_r[i]	φ'_f (°)[h]	φ'_r (°)[i]
OM1	a	140	95	0.5	1.9	179	14	W	26	24	31	26
	b	240	98	0.5	3.2	246	20	S				
	c	340	98	0.5	2.0	299	50	S-W				
M1	a	75	95	0.5	2.2	131	13	B-S	17	17	32	29
	b	150	98	0.5	2.3	137	29	S-W				
	c	225	98	0.5	2.2	207	33	S-W				
	d	255	96	0.5	4.4	250	29	S				

[a] ECP = Effective Confining Pressure; [b] B = Saturation; [c] TR = Test Rate; [d] ε_f = Axial strain at failure; [e] q_f = deviator stress at failure; [f] SS = Strain Softening defined by Bishop (1971); [g] FM = Failure Mode: W = Wedge, S = Shear, B = Barrel; [h] c'_f, φ'_f = Effective cohesion and friction at peak deviator stress; [i] c'_r, φ'_r = effective cohesion and friction at residual deviator stress

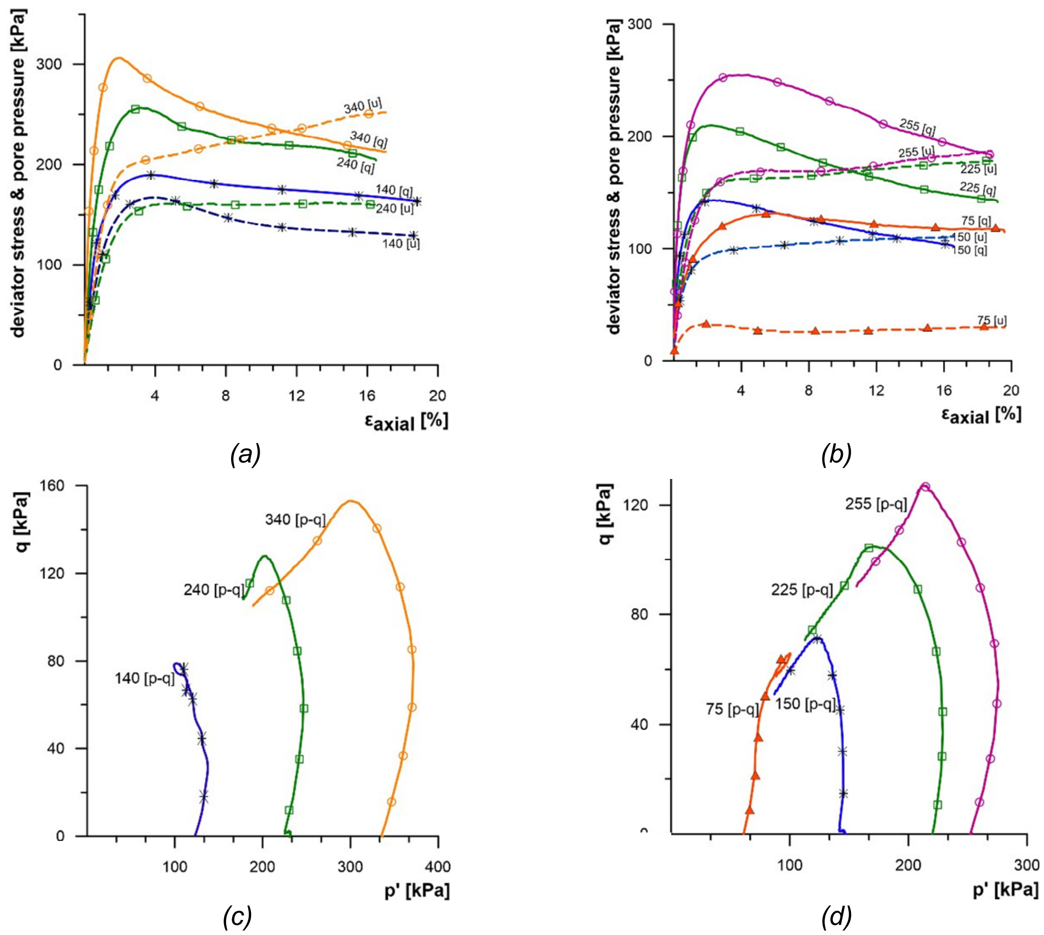

Figure 2. (a) & (b) Deviator stress & pore pressure vs. axial strain; (c) & (d) p'-q' stress path plots for Omokoroa (left – (a) & (c)) and Matua materials (right – (b) & (d)).

5 DISCUSSION

The contractive, strain softening responses observed in our volcanic material samples concurs with the observations of undrained, consolidated tests on glacial till sensitive soils (Thakur, 2007; Gylland et al., 2014; and Thakur et al., 2014). The low material permeability and high compression rate induce excess pore pressure gradients to evolve, leading to strain localisation, likely in a weak region (Zhang et al., 2015), prior to or at peak stress (Gylland et al., 2014), ending in progressive failure (Mills, 2016) within one or more shear bands. Like Gylland et al. (2013), thin section analysis of shear zones shows differential development of Riedel shears and P shears, i.e. one end of the shear zone is more developed than the other (Mills, 2016). The shear band likely creates a preferential pathway for excess pore pressure to drain, registering as a delayed response with the pore pressure base sensor as observed in the pore pressure plots (Figure 2(a) and (b)). The minor strain softening observed in samples M1a and OM1a shows that some contraction occurred, but the drop in pore pressure after peak stress shows that the majority of the sample slightly dilated. This is because the lower confining pressures allowed pore pressure to dissipate within the sample, so no gradients could evolve and induce contractive failure in a shear band.

Strain softening was initially considered by Bishop (1971) to be a reduction in stress after peak stress. Importantly, recent authors (Tavenas, 1984; Quinn et al., 2011) consider the horizontal strain or shear band displacement required to reach the residual state equally important to consider. It was not feasible to include horizontal displacement in our experimental setup, therefore the brittleness parameter we used is an estimate of strain softening. The sampling method we used may have resulted in a reduction of sensitivity of the soil, as shown by (Thakur et al., 2014).

Friction and cohesion softening was first proposed as a stress reduction mechanism by Skempton in 1964, for the case of long term stability of overconsolidated clays under drained conditions. In the range of 10 – 20% strain, or short term stability, effective cohesion and friction in sensitive glacial till soils is consistent between the peak and residual states (Gylland et al., 2013; Gylland et al., 2014; and Thakur et al., 2014). Excess pore pressure in localised shear bands has been shown to be responsible for the strain softening contractive responses observed (Gylland et al., 2013; Gylland et al., 2014; and Thakur et al., 2014). At very large strains, i.e. greater than 20%, cohesion and friction softening have been observed in sensitive glacial till soils (Stark, 1994). In our samples, little to no reduction in effective cohesion and friction angle occurred between peak and residual states (Table 2), implying that shear induced pore pressure was the governing failure mechanism.

5.1 Connecting the dots: Laboratory observations to macroscopic failure mechanism

Landslide events at both sites occurred following intense, prolonged rainfall (Gulliver & Houghton, 1980; Coffey Geotechnics, 2013; and Moon et al., 2015). We postulate that failure likely initiated within a thin layer at 23 m depth, on the basis of its extra high sensitivity (St = 130) and correlation with the base of the landslide scarp. We suggest that the high rainfall prior to the main sliding events resulted in excess pore pressure accumulation within the already saturated extra sensitive layer at 23 m, initiating strain localisation and downwards progressive failure along the extra sensitive layer at 23 m, towards the free face of the adjacent cliff. Once a critical end length of the shear band reached the residual strength, the shear band propagated through stiffer overlying tephra layers (Quinn et al., 2011). The failure mechanism that initiated the Matua slide is suggested to be similar to the Bramley Drive slip, with a tentative conclusion that the different geomorphologies are explained by the lack of a paleovalley at Matua.

6 CONCLUSIONS

Field observations of two landslides in sensitive soils in Tauranga are linked to laboratory observations of undrained, consolidated triaxial tests, in an attempt to quantify static failure mechanisms. At high confining pressures, sensitive soils sampled from the suspected failure plane of each landslide show strain softening, contractive responses coupled with rising pore pressure post peak deviator stress, and single or double shear band failure modes. Very minor effective friction and cohesion softening occurred. This evidence points to excess pore pressure gradients initiating strain localisation and progressive failure in one or more shear bands i.e. the same failure mechanism that occurs in glacial till sensitive soils. The hypothesis of excess pore pressure ties in with observed landslides at both sites occurring after heavy rainfall, whereby excess pore pressure gradients are likely to have accumulated in extra sensitive layers at depth. The high liquidity indices and water content of the soil allows the soil to flow like a liquid, explaining the long runout distances observed at both landslides.

7 ACKNOWLEDGEMENTS

This paper was inspired by Gylland and Thakur's publications on sensitive soil failure. Research was under a partnership between University of Waikato and Coffey Geotechnics, funded by the Callaghan Innovation. Thank you to Kori Lentfer, Dave Sullivan and Max Kluger for mentoring, and Kah-Weng Ho for the constructive feedback on this paper.

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