Shear Strength Properties of Northland Allochthon Soils under Rainwater Infiltration

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ABSTRACT

The Northland Allochthon soil group is one of New Zealand's most problematic soil groups; in particular, it is renowned for its susceptibility to landsliding. This study presents the results of an investigation into the shear strength properties of the Northland Allochthon soil group, with reference to its relevance on landsliding. The laboratory investigation consisted of a series of consolidated drained (CD) tests, and consolidated drained constant shear stress tests (CSD) tests on undisturbed samples of Northland Allochthon. The results from these tests are used to back-analyse a landslide which occurred on the Northland Allochthon site following a period of prolonged rainfall. The results indicate that the different stress paths obtained due to the different failure mechanism assumed for the two different triaxial tests can lead to significant differences in the shear strength parameters obtained, and thus lead to vastly different estimated factors of safety against slope failure.

Keywords: landslide, triaxial test, rainfall, Northland Allochthon

1 INTRODUCTION

The Northland Allochthon is a problematic soil group widely distributed in Northland, New Zealand (refer Figure 1). The Allochthon is particularly prone to landsliding of many different types; rotational slides, translational slides, earth flows and soil creep (Power, 2005). For example, a landslide occurred in the winter of 2008 at a man-made slope, which consisted largely of Northland Allochthon residual soil, and lies adjacent to State Highway One. The debris almost crossed over into the motorway, which could have had a disastrous effect on the Auckland roading network.

To further understand the behaviour of this problematic soil group in relation to landsliding, a series of triaxial tests were undertaken on soil samples obtained from the landslide site. Consolidated drained (CD) and consolidated drained constant shear stress (CSD) tests and ring shear tests were undertaken on saturated soil specimens. The shear strength parameters obtained were then used in a simple limit equilibrium analyses of the site in order to explain the behaviour of the slope when subjected to a period of prolonged rainfall.

Figure 1. Extent of the Northland Allochthon (Source: Winkler, 2003)

2 GEOLOGY AND SOIL DESCRIPTION

The Northland Allochthon, previously known as the Onerahi Chaos, consists predominantly of sedimentary rocks and ocean floor basalts, dating back from the Late Cretaceous (90Ma) to the earliest Miocene (25Ma) (O'Sullivan, 2009; Winkler, 2003). Submarine basaltic volcanoes, developed approximately 150kms off the Northland coast, formed a basin between the volcanoes and the Northland coast. This basin was filled with sediments; sands in close to the Northland coast and muds further out to sea. These sediments were subsequently buried by limestones, calcareous sandstones and mudstones during the Oligocene period (O'Sullivan, 2009). Although there is still some doubt as to the forming of the present day Allochthon (Tilsley, 1998), the prevailing view is as the Pacific Plate began to sub-duct beneath the Northland continental crust at the beginning of the Miocene period, the sediments that had accumulated in the basin were obducted over the submerged continental crust. This movement is thought to have occurred approximately 3 million years ago (Isaac, 1994; O'Sullivan, 2009; Winkler, 2003). The rock mass is highly deformed, sheared, faulted and folded, as a consequence of the gravitational and tectonic forces involved in the transportation and emplacement of the Northland Allochthon (Rafferty, 2001; Sporli, 2002) cited in (Lentfer, 2007).

In this research, soil samples were obtained from a site located in the Mangakahia complex of the Northland Allochthon group. From a geotechnical engineering perspective, the Mangakahia complex is the most problematic of the Allochthons (O'Sullivan, 2009). The weathering process of the Northland Allochthon leads to a generic slope profile consisting of a low permeability residual silty clay layer at ground surface. Between this residual clay layer and the rock mass lies a transition zone, which is described by Lentfer (2007) as *"silts, clays and broken rock fragments with a relatively high permeability"*. The residual soils from the Whangai Formation are described by Lentfer (2007) as *"typically comprising of a soft to very stiff (often stiff to very stiff), slightly to highly plastic, orange, brown, grey silts and clays with traces of sand."*
This transition zone described above is a combination of highly weathered rock fragments in a silt or clay matrix, often with up to minor sand present (Lentfer, 2007). This transition zone can range from a few hundred millimetres thick to over three metres thick. Slope failures have been identified to have manifested in the transition zone, on either a continuous clay seam or shear plane, or within a highly deformed shear zone within the transition zone. These shear zones can be up to 600mm thick and containing many slickensided shear planes.

3 METHODOLOGY

Undisturbed samples were retrieved from the site by hand augering to the desired depth and using push tubes which were pushed into the sample using a scalar weight. For the triaxial testing, the samples were trimmed using a lathe to form a test specimen with a diameter of 38mm and a length of 76mm. The left-overs were recycled for ring shear testing.

3.1 Consolidated Drained Tests

Standard consolidated drained (CD) tests were undertaken at a range of confining pressures following the methodology described by Head (1986). The shearing rate for a specimen with two way drainage was determined using the method described by Bishop & Henkel (1962). Shearing was continued until failure occurred or an axial strain of approximately 15% was reached.

3.2 Consolidated Drained Constant Shear Stress Tests

Consolidated drained constant shear stress tests were undertaken in a modified oedometer device. A triaxial baseplate and cell was placed in the oedometer, and weights applied on a hanger which acted as a level arm, transferring the load to the specimen as a deviator stress. In this way, the shear stress on the specimen was held constant by maintaining a constant deviator stress. This method is similar to that described by Anderson and Sitar (1995). The specimens were saturated and consolidated in the same manner as the consolidated drained tests Head (1986)(Head, 1986). Once fully consolidated, weights were added to the hanger arm until an initial principal stress ratio, k_i, of 2 was obtained. According to Orense et al. (2004), this k_i value affects the amount of dilatancy. It is also comparable to that used by Anderson & Sitar (1995), who used a value of approximately 2.4 – 2.8. Anderson & Sitar (1995) show that the in-situ k_i value for slope angles ranging from 25 to 35° ranges from 2.5 to 3.7, depending on a range of assumptions made. As the slope at the site which the

specimens were obtained from had a slope angle of 15°, a k_i value of 2 was selected. Drainage was allowed out of the specimen while these weights were added. The weights had to be added incrementally, with each weight not added until the specimen had finished undergoing volume change from the previous weight. If not, as the initial shear stress induced by the weights maybe greater than the undrained shear strength of the soil, an undrained failure of the soil could occur due to the low permeability of the soil.
Once the desired k_i was reached, the pore pressure within the specimen was slowly increased at a rate of 1 kPa per hour until failure occurred. This rate of pore pressure increase is comparable with those described in the literature on other soils with relatively a high fines content (0.67 kPa/hour; Anderson & Sitar (1995), 2 kPa/hour; Xu et al., (2011)).

4 RESULTS

The axial strain (ε) – mean effective stress (p') plots obtained from the CD and CSD tests for specimens taken at both the transition and residual soil zones are shown in Figure 2. In this figure, p' is defined as $(\sigma_1'-2\sigma_3')/2$, where σ_1' and σ_3' are the principal stresses acting on the soil specimen (σ_3' is equal to the effective confining pressure and σ_1' is equal to the effective confining pressure plus the deviator stress acting on the specimen). As observed, during the CD tests (Figure 2a), there is a continuous increase in p' as the axial strain on the specimen increases. This p' appears to reach a maximum value at a strain of approximately 7 -8 %, in both the residual zoned soils and the transition zone soils. The three stages during the shearing procedure of the CSD tests (weights applied, pore-pressure reduced and failure occurring) have been identified in Figure 2b for one of the tests. Failure during a CSD test appears to occur at a very definite p' value, indicating that there appears to be strain softening of the soil. The strain at failure during the CSD tests depends on the initial p' value applied and the initial stiffness of the soil. As a k_i value of 2 was used for the tests, a higher deviator stress was applied to the soil as p' increased. This higher deviator stress initially compressed the specimens more before the pore pressure was increased, meaning that failure occurred at a larger axial strain. In other words, the level of axial strain at failure increases with increasing confining pressure.

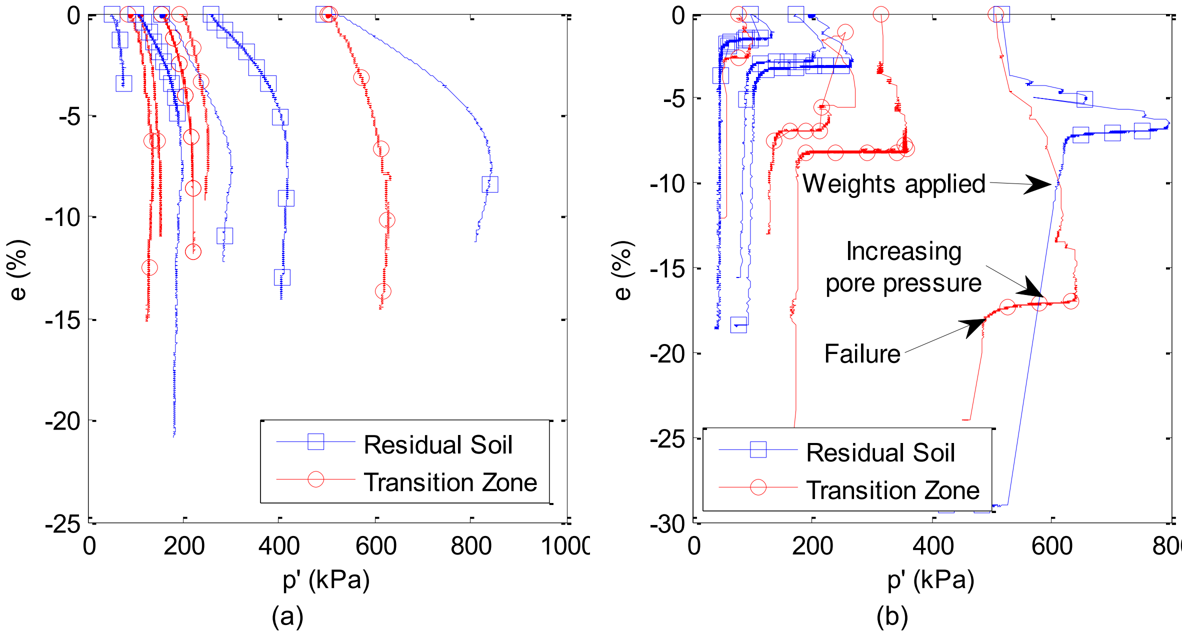

Figure 2. Axial Strain – Mean Effective Stress plots obtained for (a) CD tests; and (b) CSD tests. Note that a negative strains denotes compression.

The volume change (v) – mean effective stress (p') plots for both tests are shown in Figure 3. The residual zone soils appear to be dilatant across the range of confining pressures used, bar one test which could be due to variation within the undisturbed samples obtained. Soils obtained from the transition zone all appear to be contractive across the range of confining pressures used. The volumetric strain measured during the CSD tests consists of two stages; first, when the deviator stress is applied the specimens all decrease in volume, then as the pore pressure is increased, the volume of the specimens increases. For the tests undertaken at lower confining pressures, this resulted in a specimen failing at a greater volume than the initial volume. It appears that even during failure of the specimen, the samples still increase in volume.

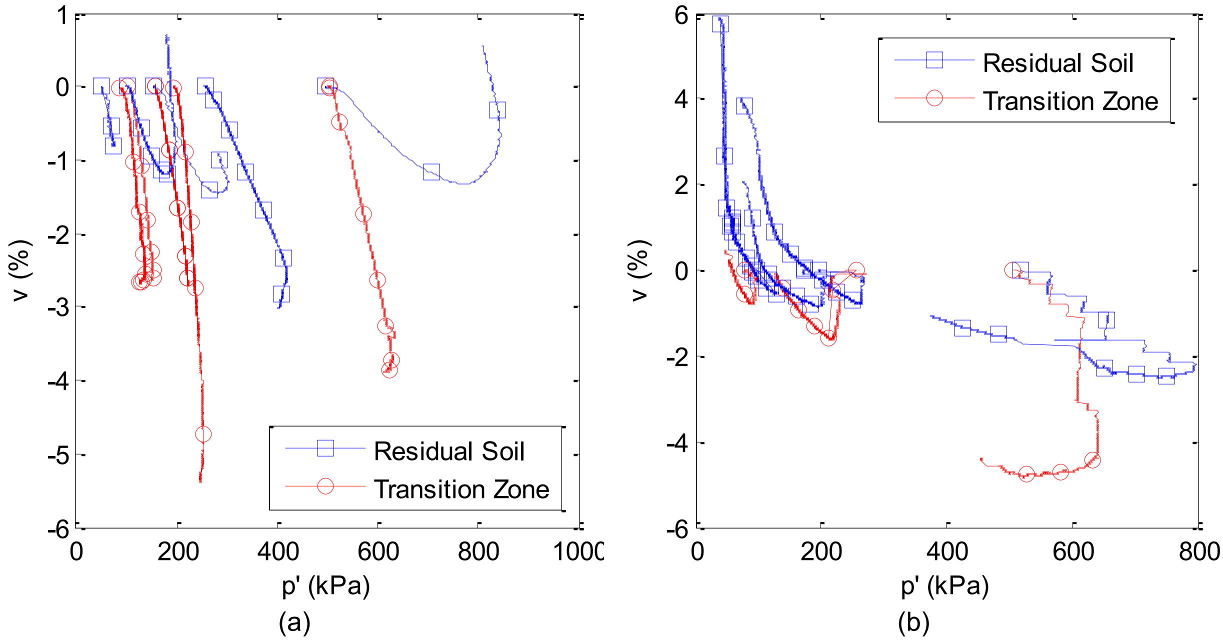

Figure 3. Volumetric Strain – Mean Effective Stress plots for (a) CD tests; and (b) CSD tests. Note that a positive volumetric strain refers to a volume increase.

Figure 4 displays the p'-q plots obtained from the CD tests and CSD tests. In these figures, *q* is defined as $(\sigma_1'-\sigma_3')$. As observed, the residual angle of shearing resistance for the residual zone soils displays a slight curvature. The shear strength parameters shown are those for the initial part of the failure envelope. The angle of shearing resistance exhibited by the transition zone is far lower for both types of tests than those exhibited by the residual soil. The shear strength parameters, Table 1, obtained from the residual soil is comparable between both the CD tests and the CSD tests. Any variance could be explained by soil variability in the sampling. The shear strength parameters obtained from transition zone samples however appear to be significantly different between the CD tests and the CSD tests. Although the angle of shearing resistance are similar (16° and 21° respectively), the tests results from the CD tests indicate that there is a significant apparent cohesion present in the transition samples. This could be due to a high curvature in the failure envelope, or due to over-consolidation (the slope was formed from a cut operation). Nevertheless, the apparent cohesion is not present in the results obtained from the CSD tests.

Table 1: Shear strength parameters obtained from triaxial testing

	CD		**CSD**	
	ϕ	**c'**	**ϕ**	**c'**
Residual	36	0	41	0
Transition Zone	16	19	21	4

5 DISCUSSION

Similar studies such as that by Orense et al. (2004) have reported results where the ϕ_{CSD} is significantly lower than $\phi_{CD.}$ However, such studies are generally undertaken on sandy materials. Of tests undertaken on materials with higher fines content, Anderson & Sitar (1995) reported very similar shear strength between compression tests and CSD tests. Atkinson & Farrar (1985), Brenner et al. (1985) and Bressani & Vaughan (1989), all cited in Anderson & Sitar (1995) all undertook similar studies on reconstituted or artificially bonded soils, and also reported dilation during shear being present.

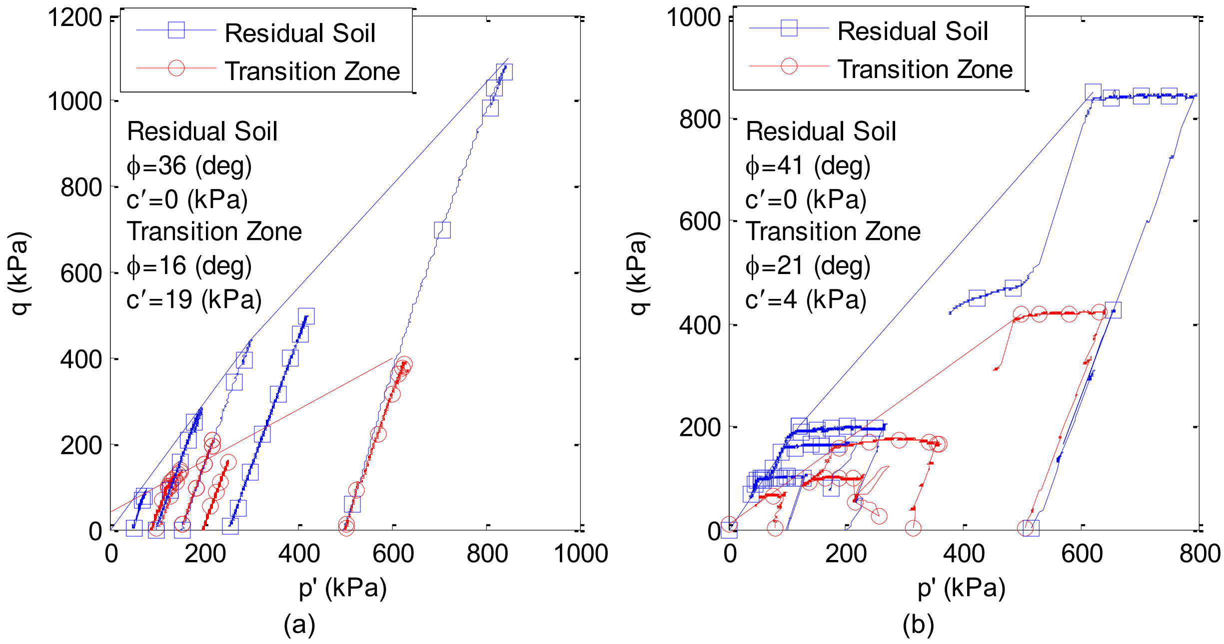

Figure 4. p'-q plots for (a) CD tests; and (b) CSD tests

A simple limit equilibrium analyses was undertaken using CD parameters (Figure 5a) and CSD parameters (Figure 5b). The landslide that occurred at the site followed a period of heavy rainfall, and hence the groundwater table was assumed to be near ground surface. Using the CD parameters, the minimum factor of safety obtained was 1.87, with the failure surface occurring in the residual soil zone. Using the CSD-obtained parameters, the minimum factor of safety obtained was 0.89, with the failure occurring in the transition zone. Site observations (Transfield Services (New Zealand) Ltd, 2008) indicate that that failure occurred across the full width of the cut. This failure surface is indicated in Figure 5. As observed, the failure surface predicted using the CSD obtained parameters is similar to the actual failure surface. Differences between the two could be because only the minimum factor of safety surface is shown – there may be other surfaces also with a factor of safety less than 1. Also, the failure could have initiated in the transition zone as shown, and progressed up slope.

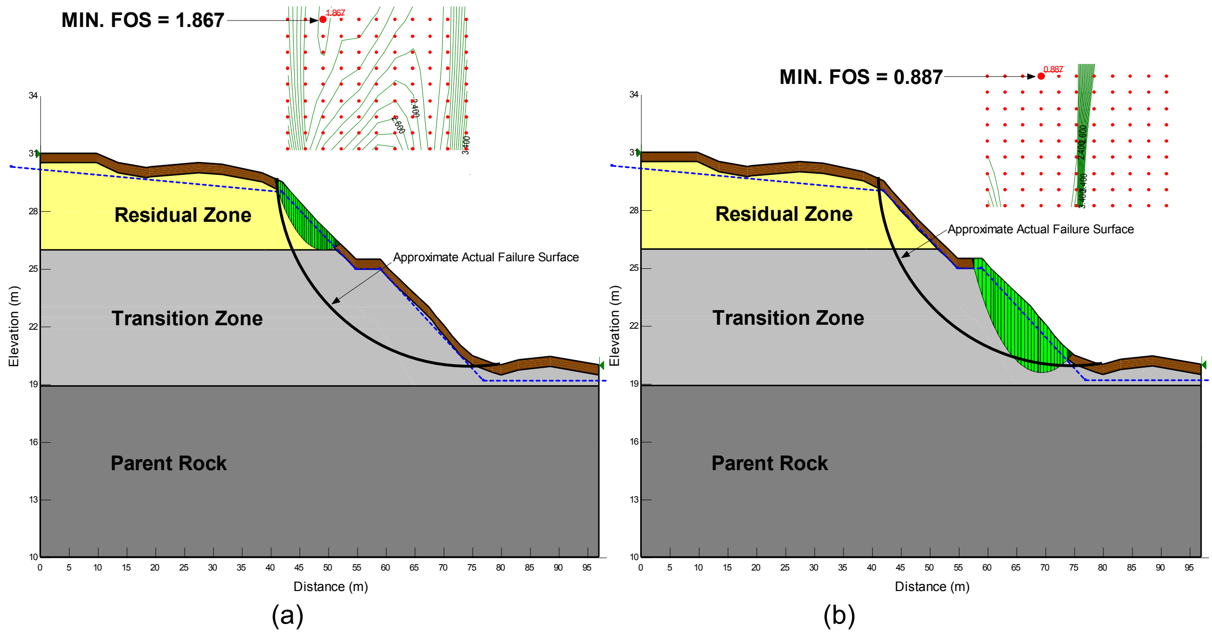

Figure 5. Simple limit equilibrium analyses undertaken using (a) CD parameters; and (b) CSD parameters.

6 CONCLUSION

Consolidated drained tests and constant shear stress drained tests were undertaken on samples obtained from both transition zone and residual zone of a Northland Allochthon site where rainfall-induced landslide occurred in 2008. Although the shear strength parameters obtained from the residual soil were similar for both tests, the transition zone displayed a significant apparent cohesion from CD tests which was not present in CSD tests. This apparent cohesion in CD test, possibly due to over consolidation of the soil, resulted in a factor of safety of 1.73 for a slope that failed during heavy rainfall. Using CSD test parameters, a factor of safety of 0.91 was obtained for the same slope. This study indicated that the failure mechanism present in the soil should be replicated in the laboratory to obtain the best shear strength parameters for use in analyses. This also indicated that when using standard triaxial testing parameters, gross overestimations of the factor of safety of the slope can be made. Further tests should be undertaken on different soils with similar characteristics (high fines content, large apparent cohesion) to verify the applicability of the results.

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