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Tatarniuk, C. and Bowman, E.T. orcid.org/0000-0001-7868-6688 (2019) Engineering behaviour and mechanical - empirical relationships for a problematic New Zealand tropical residual soil. Quarterly Journal of Engineering Geology and Hydrogeology. ISSN 1470-9236

https://doi.org/10.1144/qjegh2017-074

Engineering behaviour and mechanical – empirical relationships for a problematic New Zealand residual soil

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Abstract

Unlike sedimentary clays, many tropical residual soils do not exhibit clear mechanical-empirical relationships to assist in their engineering characterisation. In contrast, this paper discusses one residual clay in which such relationships may be determined, and further examines whether the effects of structure in this clay may be assessed using a framework previously developed for sedimentary clays. The Northland Allochthon tropical residual clay of New Zealand is a problematic soil of the fersiallitic type, prone to slope instability. Atterberg limit tests on soils from five field sites in the same geological unit show considerable variation, but that they are mechanically related. Laboratory element tests were performed on reconstituted and intact soil specimens from one field site. Normalization of the strength envelope using the equivalent stress on the intrinsic compression line suggests that soil structure, which is destroyed in reconstituted specimens, plays a role in the shear strength of this soil in its intact state. Overconsolidated behaviour, in the absence of geological preloading, points to the existence of a pseudo-preconsolidation pressure associated with weathering processes. The results further show that the saturated mechanical behaviour of this residual soil is in line with that of sedimentary clays and that mechanical-empirical relationships developed for such clays may be applied in this case.

Notation

C_c	compression index
C_s	swelling index
E	Young's modulus
e_L	void ratio at liquid limit
ε_a	axial strain
ε_{vol}	volumetric strain
φ_{crit}	critical state angle of internal shearing resistance
φ^*_e	Hvorslev true angle of shearing resistance
φ_e	Hvorslev angle of shearing resistance for intact soil
φ_{res}	residual angle of internal shearing resistance
φ_{peak}	peak angle of internal shearing resistance

G_s	specific gravity
γ_s	Shear strain
κ	slope of a swelling line based on a natural log scale
λ	gradient of normal compression and critical state line based on a natural log scale
M	gradient of critical state line
p'_0	effective consolidation pressure
PI	plasticity index
p'	mean effective stress
q	deviatoric stress
σ^*_{ve}	equivalent stress on intrinsic compression line corresponding to void ratio of soil
σ'_a	effective axial stress
σ'_r	effective radial stress
s'	$(\sigma'_a + \sigma'_r)/2$
t	$(\sigma'_a - \sigma'_r)/2$
Δu	change in pore pressure
v	specific volume
w_{PL}	plastic limit
w_{LL}	liquid limit
w_0	natural water content
χ	Hvorslev cohesion intercept
χ^*	intrinsic Hvorslev cohesion intercept

Introduction

Northland Allochthon residual clay is found in the northwest of the North Island of New Zealand. This soil forms via in situ weathering of parent mudstone to relatively shallow depths of between 1.5 m and 8 m (Winkler, 2003). Slope instability is very common within the saturated soil, at gradients as low as 8° (Glade, 1998; Tonkin & Taylor, 2006; Harris et al. 2011). This results in many small, often shallow, landslides that particularly affect the road network (East & George, 2001), leading to significant regional economic losses in the order of several millions of dollars per year (Glade 1998; NIWA and GNS, 2009). With precipitation events that may exceed 40 mm/hour and several hundred mm per day (NIWA, 2015), instability is usually triggered by high rainfall (Glade, 1998), and may occur in the form of successive slides at low angles (O'Sullivan, 2009; Tatarniuk, 2014). In spite of this, while there have been concerted efforts to understand and describe the geological provenance of the soils and their parent rocks (see Spörli & Hayward (2002) and references therein), only limited geotechnical laboratory data has been published on Northland Allochthon soils. This has been in part due to the difficulty of acquiring high quality undisturbed soil, which is friable in its natural state (O'Sullivan, 2009; Tatarniuk, 2014) and in part because of an historically local-empirical approach to geotechnical design, in the main guided by in situ rather than laboratory testing (East & George, 2001).

It is difficult to establish clear mechanical-empirical relationships for many residual soils to assist in their engineering characterisation (Geological Society Working Party, 1990). In addition, in terms of qualitative mesoscale strength of in situ material, the presence of soil structure can produce greater or lesser strength in intact soils compared to soils reconstituted for routine laboratory testing. An increase in strength may be due to microstructural ageing and chemical weathering, while a decrease in strength in overconsolidated fissured soils may be due to the presence of such fissures at the mesoscale (Hosseini Kamal et al. 2014). As residual soils, Northland Allochthon soils are highly weathered (Class E “Residual” according to BS 5930:2015). O’Sullivan (2009) has attested to the influence of fissuring on these soils, suggesting that in situ weathering may impart forms of structure in the transition zone from rock to soil, namely a decreased resistance to shearing and a change in the state boundary surface (Cafaro and Cotecchia; 2001).

Soil testing framework

According to Burland (1990), appropriate triaxial and oedometer testing of reconstituted soil can give a robust framework for comparison to intact soil, and can provide well-defined critical state parameters which can be difficult to obtain otherwise. This approach has been successfully applied to examine the influence of structure on the behaviour of intact stiff sedimentary clays (Burland et al. 1996; Gasparre et al. 2007; Hosseini Kamal et al. 2014), glacial till (Clarke et al. 1998), and aged compacted clay (Chiu et al. 2010).

As an alternative, Cotecchia & Chandler (2000) present a sensitivity framework for clays where either stress or strength sensitivity uniquely defines the yield stresses, providing a single parameter by which the clay structure may be represented. In order to use this framework, both reconstituted and intact oedometer tests must be available. However, the conventional oedometer test has been found to have limited usefulness in testing of stiff residual soils, such that for intact Auckland residual clay, Pender et al. (2003) found lower values of stiffness modulus from oedometer tests than from conventional triaxial tests while O'Sullivan (2009) found in difficulties interpreting the end of primary consolidation and atypical results in oedometer tests on intact Northland Allochthon residual clay. In addition to difficulties related to residual soil characteristics, other issues can arise. For instance, Gasparre (2005) found the calculation of stress sensitivity for London clay to be problematic because the presumed post yield behaviour diverged from the corresponding intrinsic compression curve at the end of her tests.

As a result of these considerations, in this study, we use the framework developed by Burland (1990) and Burland et al. (1996) for comparing natural and reconstituted clay to determine the degree to which the strength of this residual soil is related to its structure. Further, the results of Atterberg limit index tests and determined mechanical properties are compared from different sites, in order to explore the relationship between soils across the Allochthon sites in Northland.

This paper has two points of focus. The first is to compare reconstituted Northland Allochthon residual clay soil from one site with intact specimens under laboratory triaxial stress path tests in order to investigate its structure. The second is to determine if a relationship exists between residual clays obtained from different locations in the Mangakahia Complex (late Cretaceous, Paleocene and Eocene sedimentary soft rocks (Isaac et al. 1994; Spörli & Harrison, 2004)) of the Northland Allochthon and how this compares to other clays. The aim is to examine the potential for a framework that has been developed previously for sedimentary clays to be used for Northland Allocthon residual soil.

Geology and context of Northland Allochthon residual soil

The Northland region of New Zealand extends northeastward 60 km from Auckland to the northernmost tip of the country and encompasses an area of 14,000 km^2. The recent designation of an inland freight route as a new State Highway together with other road improvements (NZTA, 2015), means better transport links will be provided to the rest of the country in future. However, while the geology has been described as "difficult", little to no testing for physical properties has been published on the region's soils.

An Allochthon is a body of rock which has been uplifted from its original site of formation through a low angle thrust fault. The Northland Allochthon extends over much of Northland and offshore to the north and east (Ballance and Spörli, 1979; Bradshaw, 2004) (Figure 1) and can be subdivided into three main complexes (Isaac et al. 1994):

- Tangihua (submarine basaltic volcanics)
- Mangakahia (variable calcareous clay shales and siliceous mudstones and sandstones)
- Motatau (predominantly calcareous limestones, mudstones and sandstones)

Here we focus on the fersiallitic residual soils derived from Hukerenui mudstone in the Mangakahia complex, which typically comprise very soft to stiff, plastic, light coloured, clayey silts and silty clays with some sands or gravel sized clasts (Lentfer, 2007). Between the residual soil and the parent rock lies a transition zone (Figure 2), which tends to retain the sheared fabric of the underlying rock, and may contain gravel sized clasts of the parent rock. The residual zone is generally lighter in colour and has a lower permeability (Winkler, 2003) than this transition zone. The thickness of the residual soil layer varies between 2 m to 9 m, the transition zone between 4 m to 7 m, and the underlying rock between 10 m and 35 m. Note that only the residual zone soil is discussed in detail in this paper.

The Northland region is classed as temperate, with no dry season and warm summers (Peel et al. 2007). Mean annual rainfall varies from 1000 – 2000 mm across the region, hence, this fine grained residual soil generally remains saturated, year round, to near the ground surface (Wesley, 2010). The Hukerenui mudstone has been found to contain a significant amount of smectite (89%), with 8% illite, kaolinite < 3% and chlorite <1% (Lentfer, 2007), which can result in the clay being highly plastic.

In this paper, we examine data from five field sites. These are: Mountain Road, Kaeo, Ogles, Puhoi, and Silverdale. Figure 1 shows the five site locations, spread over a linear distance of nearly 250 km, and the geology at each site. Triaxial testing and detailed characterisation were carried out on soil from Mountain Road, while more limited classification testing was carried out on soil obtained from the Kaeo site, supplemented by data obtained by O'Sullivan (2009) for Ogles, Lentfer (2007) and Melrose and Willis (2010) for Silverdale, and the Further North Alliance (2013) for Puhoi. While the grouping of unpublished data in this way from different sites may not be considered "best practice", here we take a pragmatic approach to provide as much information as possible, given the sparsity of research on this soil. In doing so, we show that while the region may be geologically complex, this does not necessarily translate to geotechnical complexity, at least in terms of physical properties of the soil.

Atterberg limit tests and other soil properties

Atterberg plastic and liquid limit index tests conducted on samples obtained from Mountain Road and Kaeo sites were combined with the data available from Ogles (O'Sullivan, 2009) as well as the consulting reports and unpublished theses (Lentfer, 2007; Melrose and Willis, 2010; Further North Alliance, 2013) – Figure 3. Collectively they show that the soils at the five sites range from medium to high plasticity and despite the large distances between sites, all plot on or near the A-line, indicating their relatedness (Muir-Wood, 1990).

Particle size distributions (Figure 4), were determined for specimens from Mountain Road and Kaeo sites via sieve analysis (New Zealand Standard, 1986a) and hydrometer testing (New Zealand Standard, 1986b). Kaeo samples are seen to be more well-graded and have a significantly higher clay content than Mountain Road samples which are silt dominated and least plastic (Figure 3). Note that there is no relationship between index properties and geographical location within Northland (compare Figures 1 and 3) – that is, Mountain Road, located most centrally geographically but towards the south, has the lowest PI, while Puhoi, slightly further south again appears to be most weathered and most plastic.

Index properties for soil from the Mountain Road site are shown in Table 1, along with specific gravity and organic content. The data was obtained from laboratory testing (ASTM, 2010a; ASTM, 2010b; Germaine and Germaine, 2009) namely, specific gravity (G_s), natural water content (w_0), and organic content by loss of ignition.

Site Sampling

To obtain high quality laboratory test samples, while it is widely reported that undisturbed samples cut from carefully extracted blocks provide the best results, the high potential for shrink-swell behaviour, large number of lithic fragments and delicate friable structure of Northland Allochthon residual soil make it difficult to trim them without excessive pitting. Therefore, a different approach was taken which is similar to that taken by O'Sullivan (2009). Samples were collected in 72 mm diameter thin-walled Shelby tubes so that the soil specimens could be extruded and placed directly into a triaxial cell with little additional trimming. The adopted sampling procedure proceeded as follows. A shallow pit with a level base was excavated to between 1.5 m and 3.2 m below ground level at Mountain Road and between 1.5 m and 6.4 m below ground level at Kaeo. The 450 mm long, 72 mm diameter stainless steel tubes were hydraulically jacked into the ground using a specially constructed rig and jacking frame at a controlled rate of around 20 mm / min with the excavator buck used to provide a vertical reaction (Figure 5). The tubes were then manually excavated with care, the ends sealed with wax then double wrapped in plastic and fully sealed for transport. In total, six tube samples were taken from Mountain Road and a further six from Kaeo. These included soils from both the residual and transition zones. This method of sampling eliminates the drilling or boring stage, which is recognized to result in the greatest disturbance (Ladd and DeGroot, 2003). Sample disturbance may nevertheless have impacted the behaviour of the intact soil: Lunne et al (1997) showed that use of tube samplers may result in a decreased peak and residual stress measurement in clay, due primarily to soil destructuring (DeGroot et al 2005). In the current study, it is recognized that the effects of this may result in a less pronounced difference between the intact and undisturbed properties of the soil.

In Situ Flat Plate Dilatometer Testing

In order to obtain data on the in situ stress history of the soil, Flat Plate Dilatometer tests (DMT) were conducted at Mountain Road (Marchetti, 1980; 2015). Three DMTs, denoted sDMT 5, 6, and 7 were located approximately 5 m from the extraction point for the samples (other tests were carried out further away and results are not used here, although results produced similar trends).

The horizontal stress index, K_D is one of the key intermediate parameters obtained from the DMT (Marchetti, 1980). The overconsolidation ratio (OCR), defined as the ratio of maximum vertical effective stress to current effective stress, may be estimated from K_D through Equation 1 (Marchetti, 2015):

$$OCR = (0.5\,K_D)^{1.56} \qquad 1.$$

As shown in Figure 6, despite there being nothing in the geologic history of this residual soil to suggest that the present overburden stress of the soil has been exceeded in the past (Isaac et al., 1994) it exhibits mean OCR's well above unity throughout its depth, with a particularly marked high value near the surface. The potential reasons for this are discussed further in the "preconsolidation" section at the end of this paper.

Laboratory mechanical testing

The laboratory testing programme focuses on soil specimens from the Mountain Road site. This soil is light brown, mottled with iron staining, and moderately weathered. While generally described (and treated here) as a clay, the particle size distribution indicates that it is in fact a clayey silt with traces of gravel and sand.

Triaxial testing

Undisturbed residual soil samples were taken at the Mountain Road site from a depth of between 1.5 m and 3 m below ground using 72 mm inner diameter Shelby tubes. Samples were extruded from the tubes using a motorised hydraulic ram fitted with a 70 mm shoe. To do this, the wax seal end caps were trimmed off and the samples extruded and then re-trimmed using thin steel wire cutters. When a sufficient length had been extruded for the current test, the remaining sample within the tube (if any) was sealed with a mixture of paraffin wax and petroleum jelly, double wrapped at both ends using cling film and the tube placed in two tightly sealed plastic bags. The samples were then placed in a plastic sealed box with water at the bottom to further ensure that they were in a moist environment. Once extruded, samples were taken to the preparation area in mitre boxes specially made for the samples. Trimmings were weighed and dried to verify moisture content.

Reconstituted specimens were prepared from a slurry of the Mountain Road soil at a water content of 1.25 to 1.5 times its liquid limit (Burland, 1990). The soil was mixed with water for a minimum of 6 hours and then slowly poured into and consolidated over time in a 150 mm long and 50.7 mm internal diameter consolidometer constructed for the purpose of obtaining 50mm diameter samples. Two-way drainage was allowed via perforated discs, porous stones and filter paper applied to each end. The consolidation device utilized the application of pressure to the top of the sample through the addition of weights and pressure was increased in stages according to the method for consolidation in oedometer testing (Head & Epps, 2011). The change in height was monitored and plotted versus the log of time to ensure end of primary consolidation was reached before next load was added. All specimens were consolidated to 150 kPa vertical effective stress before extrusion. The method used for trimming and extruding the samples was the same as that used for the intact samples. Trimmings were used to determine the water content of the specimen at the start of each test.

Triaxial testing was performed using a GDS triaxial stress path cell with computer control. The room temperature was well-controlled and continuously monitored to ensure it did not vary by more than 0.5°C. Reconstituted specimens were 50 mm in diameter and 100 mm long, whilst undisturbed specimens were 72 mm in diameter and 130 mm long. Stress path cell instrumentation included pore pressure and cell pressure transducers, and an internal load cell. Local displacements were measured by using axial submersible on-sample LVDTs (linearly variable differential transformers) for all specimens. Radial on-sample LVDTs were also used on all reconstituted specimens. Pore pressure measurement was carried out at the top of the specimen and drainage was provided at the bottom. Undisturbed specimens were tested using enlarged lubricated ends in order to improve uniformity at all strain levels.

The saturation of reconstituted specimens was carried out for a minimum of 24 hours. However for the undisturbed samples, with the use of lubricated end platens which had a much smaller porous stones, the saturation process took as long as 5-7 days. Saturation was completed in stages of 100 kPa up to a back pressure of between 400 kPa and 1000 kPa. The Skempton B value was checked after each 100 kPa increment as well as at the end of the saturation period. A minimum B value ($\Delta u/\Delta\sigma'$) of 0.95 was obtained, equating to a saturation ratio of greater than 97%.

Following saturation, isotropic pressure ramps were used to bring the sample to the desired maximum mean effective stress for each test. Consolidation of the soil specimens to between 250 kPa and 800 kPa typically took between two and five days. Five of the reconstituted soil specimens (tests R1 to R5) were then subjected to a new reduced isotropic pressure in order to attain desired overconsolidation ratios of between 2 and 10 (in line with stress history of the Mountain Road site, Figure 6). The other five tests (R6 to R10) were normally consolidated to pressures between 250 and 800 kPa. Table 2 summarizes the tests performed. CU indicates consolidated undrained tests, CD indicates consolidated drained tests, and OCR indicates the overconsolidation ratio. Upon completion of each test, the samples were removed from the stress path cell and their water contents were measured.

Oedometer testing

Oedometer samples were prepared using reconstituted Northland Allochthon clay soil from Mountain Road, mixed to a water content approximately equal to its liquid limit. The oedometer apparatus was controlled by air pressure, and load increments were applied through computer control. The inner diameter of the oedometer ring was 63 mm and inner height was 19 mm.

The load increments were applied in sequence from 3 kPa, up to a maximum pressure of 800kPa, before unloading in stages to between 25 kPa and 6 kPa. Deformation was monitored using a digital linear gauge to ensure primary consolidation was nearing completion prior to increasing the applied stress, similar to the method used for consolidating the reconstituted soil specimens for triaxial testing. Once the final increment of loading was reached, the test was dismantled and the final water content was recorded.

Results of laboratory tests

Oedometer tests on reconstituted soil

Oedometer testing on reconstituted soil from Mountain Road was performed to acquire an intrinsic compression line (ICL) (Burland, 1990). The results of the three tests, Oed1 to Oed3, conducted are provided in Figure 7 and Table 3.

It is now useful to put the Northland soil into the context of other clays, albeit sedimentary ones. The mean slope of the ICL, λ of 0.121, lies between that of London clay ($\lambda \approx 0.1$), and pure kaolin ($\lambda \approx 0.2$) (Atkinson and Evans, 1985). The ratio between PI and λ is 176, and compares well to the mean value of 170 given by Schofield and Wroth (1968) based on five clay soils. The λ/κ value is approximately 8.1, which is larger than the range of values (2-5) given by Schofield and Wroth (1968) but similar to the value of 7.2 reported by Allman and Atkinson (1992) for Bothkennar soil, which is often described as a clay but is predominantly comprised of silt sized particles.

Noting that $\lambda = C_c / 2.303$, the compression index (C_c) of reconstituted sedimentary clays (i.e. the gradient of the intrinsic compression index) that lies on or slightly above the A-line can be estimated using the empirically derived Equation 2 (Burland, 1990):

$$C_c = 0.253e_L - 0.04 \qquad 2.$$

Where e_L is the void ratio at the liquid limit. Using this equation gives a C_c of 0.29 (λ of 0.126) for two samples of residual clayey silt soil from Mountain Road, which is consistent with the average C_c of 0.278 (λ of 0.121) attained from the oedometer tests on reconstituted specimens.

Triaxial tests on reconstituted soil

Figure 8 shows (a) the deviatoric stress, q versus axial strain, ε_a, and (b) the pore pressure change Δu versus axial strain, ε_a for the consolidated undrained (CU) triaxial tests on both the normally consolidated and overconsolidated reconstituted specimens. The normally consolidated (NC) samples exhibited well-defined critical states, failure took place after slight bulging, and most of the samples did not develop shear bands. The slightly overconsolidated sample (OCR of 2) exhibited rather similar deformation to the samples that were normally consolidated, in that a gradual peak in deviatoric stress was reached, with no post peak reduction and pore pressure increase was positive. This is in contrast to the results reported by Burland et al (1996) on four overconsolidated clay samples with OCR of 5 and above, which developed modest post peak strength reduction coupled with negative pore pressure change. Higher OCR values were not examined in this study for the intact specimens (72 mm in diameter) due to limitations in the load cell utilized.

Figure 9 shows corresponding triaxial data for the consolidated drained (CD) tests. The CD tests were performed on overconsolidated specimens and all displayed failure along shear bands after slight bulging. The specimens at OCR values of 5, 7 and 10 showed post-peak reductions in shear strength (Figure 9(a) and corresponding contraction followed by dilation (Figure 9(b)), although OCR of 7 sample produced less dilation than the OCR of 5 sample, which was not expected. At the lowest OCR of 3, the behaviour was purely contractive and there was no peak in shear strength with increasing strain.

Burland et al (1996) referred to the mechanical properties of a reconstituted clay as intrinsic properties, since they are inherent to the material and are independent of its natural state. The normally consolidated intrinsic failure line in t-s' space derived from the reconstituted specimens is shown in Figure 10. The intrinsic failure line is defined by the peak strengths of the normally consolidated specimens and is very slightly curved. The slope of the intrinsic failure line at s' = 210 kPa is 34.9°, while at s' = 550 kPa it is 33.6°. The peak strengths for the overconsolidated drained samples (open triangles in Figure 10) lie slightly above the intrinsic failure line. This behaviour can also be seen in the results for all four of the stiff sedimentary clays studied by Burland et al. (1996). The final sharp bend to the right in the undrained stress paths indicates the material is slightly dilatant.

The intrinsic Hvorslev strength envelope represents the strength of the soil in the reconstituted state (Figure 11). This envelope was obtained by normalizing the results from Figure 10 by using the void ratio of each sample at failure and the one-dimensional compression curves from test Oed2 (Figure 7). The Hvorslev true angle of shearing resistance (φ^*_e) is 32.6°. The intrinsic Hvorslev cohesive intercept, χ^*, is 0.04 and the value of s'/σ^*_{ve} at critical state is 1.42.

The stress ratio q/p', versus axial strain, ε_a for the normally consolidated undrained tests on reconstituted specimens is shown in Figure 12. The mean stress ratio at critical state, M, in Equation 3 is 1.38:

$$q = Mp' \qquad 3.$$

The value of φ_{crit} in compression can be determined from Equation 4:

$$sin\varphi'_{crit} = 3M/(6 + M) \qquad 4.$$

Giving a value of 34.2° for φ_{crit}, in line with the values of 33.6° to 34.9° from the intrinsic critical state line in s'-t space.

Triaxial tests on undisturbed samples

Three triaxial tests were performed on undisturbed specimens of Northland Allochthon residual clay from the Mountain Road site, at confining pressures of 110 kPa, 130 kPa and 190 kPa, respectively (Table 2). These confining pressures are representative of the in situ mean effective stress conditions of the soil specimens. As shown in Figure 13(a), the specimen tested at the lowest confining stress, 110 kPa, showed a gradual post peak reduction in strength, and developed a single failure surface, while the samples tested at 130 kPa and 190 kPa exhibited well-defined critical states, and failure of these samples took place after slight bulging. Although generally ductile, the test at 130 kPa exhibited a higher peak deviatoric stress than at 190kPa. Although this is not expected, this type of behaviour does occasionally occur in intact specimens due to their variability, and can be seen in the results on drained specimens of Todi clay in Burland et al. (1996). Despite this, all the samples exhibited positive pore pressure change (Figure 13(b)), with higher confining stresses resulting in higher development of pore pressure.

The intact failure line in t-s' space (determined via linear regression) is shown in Figure 14. All stress paths show dilatant behaviour, indicating the soil is behaving as an overconsolidated material at these confining pressures. The behaviour is more dilatant at p'_0 = 110 kPa and 130 kPa than at p'_0= 190 kPa, as expected. The peak intact strengths (hence intact failure line) lie close to the intrinsic failure line for the reconstituted material, however, this does not necessarily indicate that the microstructure of the soil does not contribute to the soil strength. The normalization parameters for the intact specimens are shown in Table 4, Figure 15 evaluates where the intact soil failure envelope lies in comparison to the intrinsic failure envelope in normalized t/σ^*_{ve} –s'/σ^*_{ve} space. The Hvorslev failure line for the intact material lies above the intrinsic line. The Hvorslev cohesive intercept for the intact material, χ, is 0.18, giving a ratio of χ/χ^* of 4.5. The ratio of the normalized strengths at intrinsic critical strength (T) is 1.2, indicating there is an influence of structure on the strength of the intact soil.

Strength characteristics from the literature

Little published data is available on the strength characteristics of Northland Allochthon residual soil. Available data from three field sites is summarized in Table 5. The data from Mountain Road is that obtained from this study. The values of the residual angle of shearing resistance (φ_{res}) for the Ogles and Silverdale field sites were obtained from ring shear tests and peak and/or critical state parameters from triaxial testing (O'Sullivan, 2009; Harris, 2013).

In summary, for Northland Allochthon residual soil from the Mangakahia complex, the critical angle of internal shearing resistance (φ_{crit}) for two field sites ranges from 26° to 34° (Table 5). The value of the peak critical angle of shearing resistance (φ_{peak}) also has significant variation, ranging from 27° to 36°. The cohesion intercept in the Northland Allochthon residual soil was found to be quite low (0 kPa – 6 kPa) for all three of the sites. With respect to ring shear results, φ_{res} varied from 11° to 19° between two sites, with the larger angle corresponding to a larger peak value.

Discussion

Comparison to four stiff clays from Burland et al. (1996)

As Burland's (1990) framework for examining the influence of soil structure on soil strength has been utilized, some comparisons can be made to the four stiff clays examined by Burland et al. (1996). The value of s'/σ^*_{ve} at critical state attained for the normally consolidated and reconstituted Northland Allochthon residual clay soil at Mountain Road was 1.5. This is slightly higher than for the clays tested by Burland et al. (1996) (the three Italian clays had s'/σ^*_{ve} of 0.75 and the Corinth marl s'/σ^*_{ve} of 1.2). The slight dilatancy of the Northland Allochthon residual clay as it approaches critical state would contribute to this higher s'/σ^*_{ve}. The ratio of the normalized strengths at intrinsic critical strength (T) at 1.2 is very close to the range of values obtained by Burland et al. (1996) from sedimentary clays, which ranged from 1.23 to 1.5. Burland et al. (1996) found that φ'_e was similar for intact and reconstituted specimens for all of the tested clays and the results for the Northland Allochthon residual clay soil also showed similar φ'_e for intact and reconstituted specimens. Hence, with respect to this framework, the Northland Allochthon residual soil at Mountain Road behaves similarly to overconsolidated sedimentary clays, despite originating from a quite different geological process.

Variation in critical state parameters λ and φ_{crit}

The CU tests on normally consolidated reconstituted specimens from Mountain Road reached a well-defined critical state. The mean critical state angle ϕ'_{crit} of 34.2° is high for a clay, but is similar to that attained for reconstituted Bothkennar soil (1.38) (Allman & Atkinson, 1992), which was attributed to its high silt content. The Northland Allochthon soil at Mountain Road has a silt content at over 60% (Figure 3).

Table 6 compares the critical state parameters acquired from the residual soil in this study to several other clay soils. The top four results are for sedimentary soils (Atkinson, 1993; Allman & Atkinson, 1992), representing a typical till, a stiff overconsolidated clay, a pure clay and a soft clayey silt (Hight et al. 1992), and the last four are from a study on tropical residual soil in Bangladesh (Hossain, 2001) and this study.

Atkinson (1993) suggested that the intrinsic critical state parameters (λ and φ_{crit}) may vary due to differences in grading and mineralogy between samples, and depends primarily on the nature of the soil. The critical state parameters acquired by Hossain (2001) for tropical residual soil of Dhaka, Bangladesh show clear variability between the boreholes, which are considered to be in the same formation. For instance, samples of borehole 1 and 3 were highly oxidized and much more weathered compared to those of borehole 2. Research by Rahardjo et al. (2004) and Rocchi & Coop (2015) on grantitic residual soils provides further evidence for their properties varying geographically and with depth due to different degrees of weathering. In these cases, the peak and critical state angles of internal shearing resistance were found to reduce as the mean particle size decreased and PI increased. The variation in the values of φ_{crit} found between Ogles (O'Sullivan, 2009) and Mountain Road are in line with these observations.

Muir-Wood (1990) proposed that all related soils (perhaps of similar activity) should pass through a single point in compression space (specific volume, v versus mean effective stress, p'). In terms of critical states, this implies that movement of a soil down the A-line on the plasticity chart leads to a reduction in compressibility, and thus changes the slope of the critical state line, M. The w_L at Ogles was 69-72% and at Mountain Road it was 48-50%, implying a lower compressibility at Mountain Road.

Muir-Wood (1990), summarized an empirical relation between PI and φ_{crit} (after Mitchell, 1976) for several normally compressed sedimentary soils (Equation 5), where:

$$sin\emptyset' = 0.35 - 0.1\ln(PI) \qquad 5.$$

And φ' is related to M according to Equation 4.

Using Equation 5, φ_{crit} should be around 30° for the Mountain Road site and around 27° for the Ogles No.3 site (compared to 34° and 26° from triaxial tests, respectively). The correlation, though loos e, demonstrates that a lower PI returns a higher φ_{crit}.

The relationship between PI and M (after Muir-Wood 1990) with the addition of data from the Northland Allochthon residual clay, Bothkennar soil and Dhaka clay, is shown in Figure 16. Both Bothkennar and Northland Allochthon soil from Mountain Road plot above the mean relationship. Allman & Atkinson (1992) for Bothkennar soil with 3.5% organic content, obtained a φ_{crit} of 34° from reconstituted triaxial results, whereas the Muir-Wood relationship results in a φ_{crit} of 26.5° based on a PI of 38%. T herefore, Equation 4 does not hold well. However, when an organic content of 3%-8% was removed from Bothkennar soil (Albert et al. 2003), the PI reduced to 18%-20%. Considering the 5.5% organic content in the Mountain Road soil, taking a similar approach would result in an estimated φ_{crit} of 30°-31° from the Muir-Wood M-PI relationship, which is a notably better match to the φ_{crit} of 34°.

The Northland Allochthon residual soil from Mountain Road falls close to the average trend line established for sedimentary clays. It is offset above it and outside the realm of the pure soils of kaolinite and illite, suggesting that other clay minerals and its 5.5% organic content also play a role in its plasticity and strength at critical state. While the Geological Society Working Party (1990) urges caution on using correlations between Atterberg Limits and soil strength parameters for residual soils, further substantiation to the argument that the high φ_{crit} of the Mountain Road soil is related to its lower PI and w_{LL} can be found by examining correlations between w_{LL} and the residual frictional angle from numerous clays by Mesri and Cepeda-Diaz (1986). Based on their correlations, the residual friction angle should be 12°-14° for the Ogles soil, which is in agreement with values of 11°-14° obtained from ring shear tests (O'Sullivan, 2009). For the Mountain Road site, the w_{LL} correlates to a residual friction angle of 21°-22°. It follows that the critical state angle is likely to be relatively high at Mountain Road as well.

Correlations between index properties and shear strength for many residual soils have been found to be difficult (Geological Society Working Party, 1990). However, the results from Lentfer (2007) on the Hukerenui mudstone mineralogy, and the clear reduction in internal angle of shearing resistance from critical state to residual for this soil is indicative of a dominance of platey clay minerals, indicating an absence of allophones and halloysites. The presence of clay minerals may be responsible for the particularly complex behaviour for which tropical residual soils are known apply (Geological Society Working Party, 1990). Hence as the mineralogy of Northland Allochthon residual clay soil is similar to many sedimentary soils, this may be a contributing factor as to why correlations for sedimentary clays are found to be applicable here.

Preconsolidation

There is nothing in the geologic history of the residual soil derived from the Northland Allochthon rock (Isaac et al. 1994) to suggest that the present overburden stress of the soil has been exceeded in the past. Wesley (1990) studied three different residual soils, and found that they all behaved as moderately or heavily overconsolidated soils, but similarly to the Northland Allochthon residual clay soil, at least two of them had never been subjected to overconsolidation by preloading. Since "preconsolidation" pressures of such soils do not bear relation to overburden history, it is therefore likely to be the result of weathering processes. For this reason, the preconsolidation pressure found for residual soils is often termed pseudo-preconsolidation pressure (Wesley, 1990). The mechanisms that could lead to pseudo-preconsolidation (OCR higher than 1) include desiccation (especially near the surface), drained creep (long term secondary compression), and structuration which can be due to physiochemical processes (natural cementation due to carbonates and silica, bonding due to ion exchange) (Mitchell and Soga, 2005). The slightly dilatant behaviour of the intact specimens of Northland Allochthon residual clay during undrained loading shows that they behave as overconsolidated at these confining pressures, providing substantiation of a pseudo-preconsolidation pressure in this soil. Other authors (Futai et al. 2004; Wang & Yan, 2006) have also noted overconsolidated soil behaviour in residual soils at confining pressures of 400 kPa or less.

Conclusions

Northland Allochthon residual clay soil is a problematic soil type upon which very little geotechnical investigation has been performed. Atterberg limits from five sites located in Northland Allochthon residual soil derived from Hukerenui mudstone all fall on or near the A-line, suggesting that the soil from the five sites, separated by nearly 250 km, is related.

Oedometer and triaxial tests were carried out on reconstituted and intact soil from one site, Mountain Road, in order to determine how the soil behaved with respect to Burland et al's (1990) framework developed for natural sedimentary clays. The results of these tests showed that this framework is appropriate for this soil, despite it being classed as a residual soil. The Hvorslev cohesive intercept for the intact material, χ, was found to be 0.18, giving a ratio of χ/χ* of 4.5. The ratio of the normalized strengths at intrinsic critical strength (T) is 1.2, indicating that the intact soil is stronger than that of the reconstituted soil. The value of φ_e of 32.8° was only 0.2° more than φ^*_e, however, suggesting that the difference in strength is cohesive rather than frictional in nature and hence, bonding plays a role in the strength of the intact soil. The undrained stress paths of the intact specimens of Northland Allochthon residual clay demonstrated slightly dilatant behaviour, suggesting a pseudo-preconsolidation pressure which is sometimes seen in residual soils.

The critical state angle of shearing resistance at Mountain Road φ_{crit} was found to be 34.2°, which is high for clay. It is also considerably higher than the value of 26° acquired for soil in the same for mation at another field site. Muir-Wood's (1990) correlation relating M (and therefore also φ_{crit}) to the PI indicates that the higher φ_{crit} is likely related to the low plasticity at the test site and its organic content. The results also suggest that simple plasticity limits tests coupled with Muir Wood's (1990) correlation developed for sedimentary clays could be used, with due caution, as a first order screening tool in selecting an initial φ_{crit} in slope design or back-analysis of failure in Northland Allochthon residual clay soil.

Much of the literature on residual soils is focused on soil that is unsaturated, or partially saturated (Geological Society Working Party, 1990) while clay minerals such as halloysite and allophane are frequently present in residual soils and are fundamentally different from smectite and illite, found largely in sedimentary soils. As such, many of the empirical relationships derived from sedimentary soils cannot be easily applied. However, the results of this study have demonstrated that these relationships may, in fact, be applied to the behaviour of some residual soils, such as that of the Northland Allochthon, which are generally saturated in situ, and are composed of common, platey, clay minerals. We recommend further testing of both intact and reconstituted soil samples from sites across the Northland Allochthon. Scanning electron microscopy would be useful to support mineralogy associations for observed soil behaviour. Large-ring oedometer tests on intact specimens should be considered to provide an assessment of K_0 compression that should be more effective for this soil type. These results further provide groundwork for conducting and interpreting future laboratory tests on Northland Allochthon residual clay soil towards better geotechnical design.

Acknowledgements

The first author was supported in her research at the University of Canterbury, New Zealand, by the Commonwealth Scholarship Plan of Canada. The authors would also like to thank Hiways Geotechnical Ltd. for assistance with soil sampling.

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Table 1: Atterberg indices and other soil properties (Mountain Road site).

w_0 (%)	w_{LL} (%)	w_{PL} (%)	G_s	Organic Fraction (%)
36	49.6	28.3	2.61	5.5

Table 2: Summary of triaxial tests.

Specimen Type	Name	CD/CU	Effective Stress (kPa)	Consolidaton Pressure (kPa)	OCR
Reconstituted	R1	CD	80	800	10
	R2	CD	83	250	3
	R3	CD	160	800	5
	R4	CD	114	800	7
	R5	CU	125	250	2
	R6	CU	250	250	1
	R7	CU	400	400	1
	R8	CU	800	800	1
	R9	CU	550	550	1
Undisturbed	N1	CU	110	110	n/a
	N2	CU	190	190	n/a
	N2	CU	130	130	n/a

Table 3: Parameters acquired from oedometer results.

Test #	Cc	Cs	λ	κ
Oed1	0.240	0.036	0.104	0.016
Oed2	0.285	0.034	0.124	0.015
Oed3	0.308	0.033	0.134	0.014
Mean	0.278	0.034	0.121	0.015

Table 4: Normalization parameters of in situ triaxial tests.

Specimen Name	e	σ^*_{ve} (kPa)
N1	0.934	68.9
N2	0.891	98.3
N3	0.900	90.7

Table 5: Summary of strength characteristics of Northland Allochthon residual soil from three field sites.

Parameter	Units	Field Site	Mountain Road	Ogles[1]	Silverdale[2]	Silverdale[3]
		Complex/ Formation	Undifferentiated Mélange, predominantly Mangakahia Complex Mudstones	Mangakahia Complex - Hukerenui Mudstone	Mangakahia Complex - Whangai Formation	Mangakahia Complex - Whangai Formation
c'	kN/m^2		2.5	6	0	1
ϕ'_{peak}	°		35.6	30	36	27
ϕ'_{crit}	°		34.2	26	-	-
M			1.38	1.03	-	-
ϕ_{res}	°		-	11 - 14	19	-

[1]O'Sullivan (2009)
[2]Harris (2013)
[3]Tilsley (1998) as cited in Harris (2013)

Table 6: Comparison of critical state parameters from the Mountain Road site to other soils.

Soil Type	λ	M	φ_{crit} (°)
Glacial till[1]	0.09	1.18	29.5
London clay[1]	0.16	0.89	22.8
Kaolin clay[1]	0.19	1	25.4
Bothkennar clay[2]	0.18	1.38	34.1
Tropical residual clay (Dhaka- Borehole 1)[3]	0.07	1.05	25.8-26.5
Tropical residual clay (Dhaka- Borehole 2)[3]	0.06	0.96	24.2-24.4
Tropical residual clay (Dhaka- Borehole 3)[3]	0.05	0.84	21.6
Northland Allochthon residual clay (Mountain Road	0.12	1.38	34.2

[1]Atkinson (1993)
[2]Allman & Atkinson (1992)
[3]Hossain (2001)

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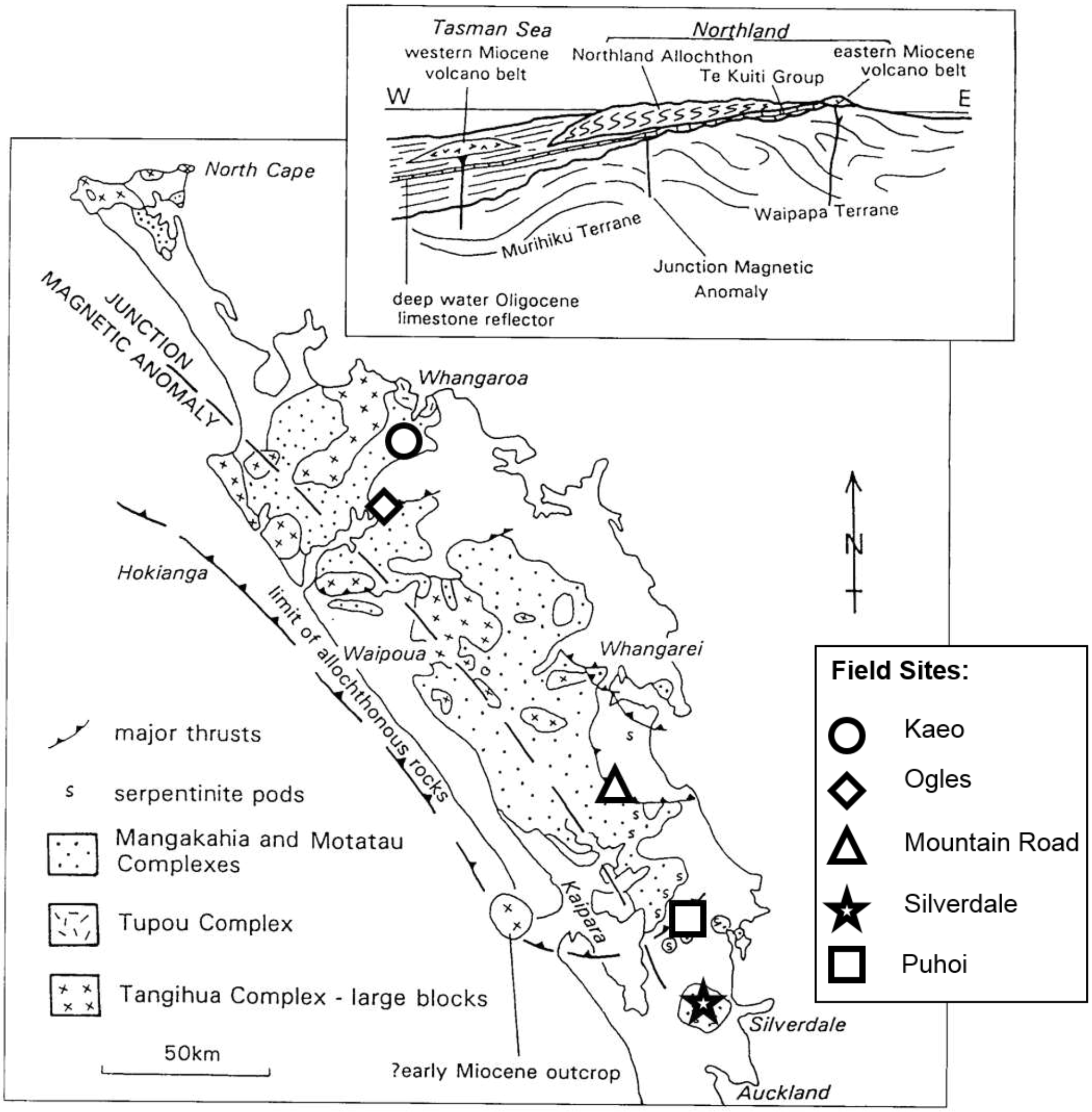
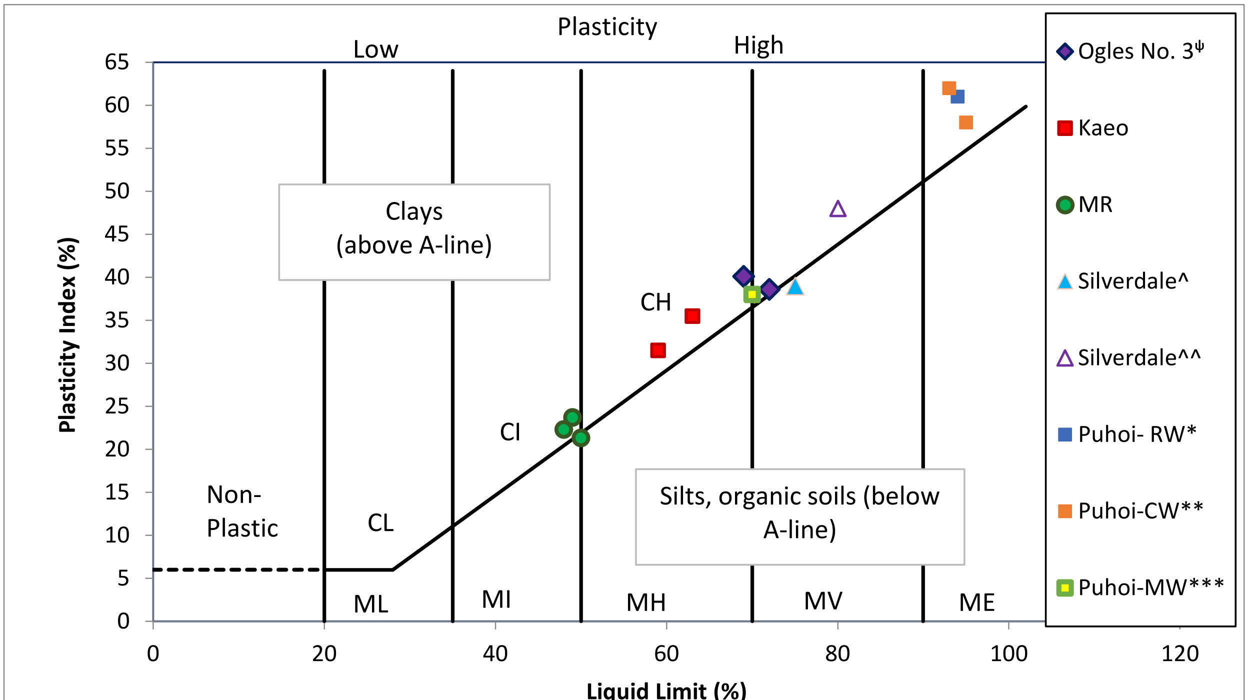


Figure 1: Map of approximate field site locations and Northland Allochthon outcrop, modified after Hayward et al. (1989).

Figure 2: Upper (residual) and lower (transition) soil zones at Mountain Road.



C = Clay M = Silt O = Organic Pt = Peat	L = Low Plasticity I = Intermediate Placticity H = High Plasticity V = Very High Plasticity E = Extremely High Plasticity	$w_{LL}< 35\%$ $35\% < w_{LL} < 50\%$ $50\% < w_{LL} < 90\%$ $70\% < w_{LL} < 90\%$ $w_{LL} > 90\%$	ψO'Sullivan (2009) ^ Lentfer (2007) ^^Melrose & Willis (2010) *RW-Residually Weathered, Far North Alliance (2013) **CW- Completely Weathered, Far North Alliance (2013) ***MW- Moderately Weathered, Far

Figure 3: Results of Atterberg limits on Northland Allochthon residual clay soil.

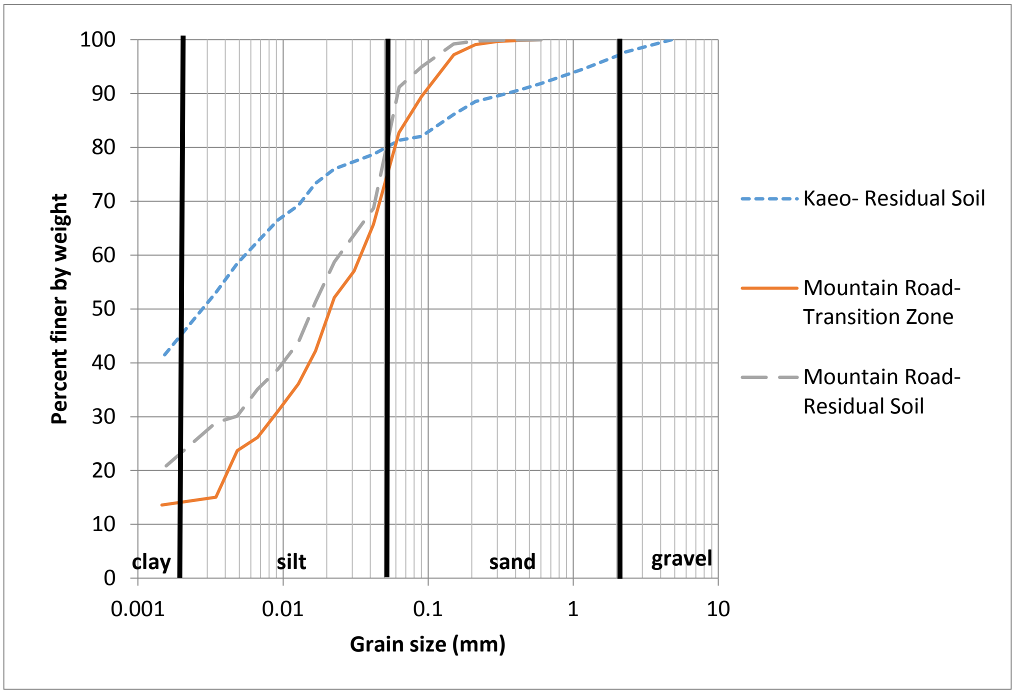


Figure 4: Grain size distribution for specimens from Kaeo and Mountain Road field sites.

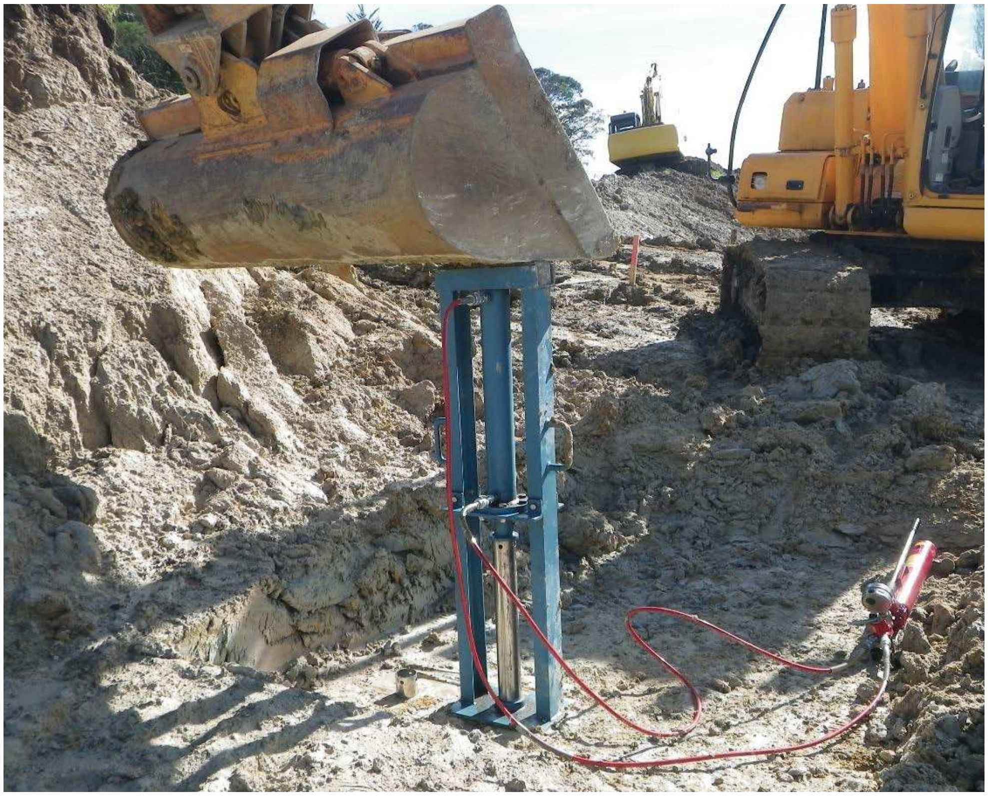


Figure 5: Rig and jacking frame used for Shelby tube same collection.

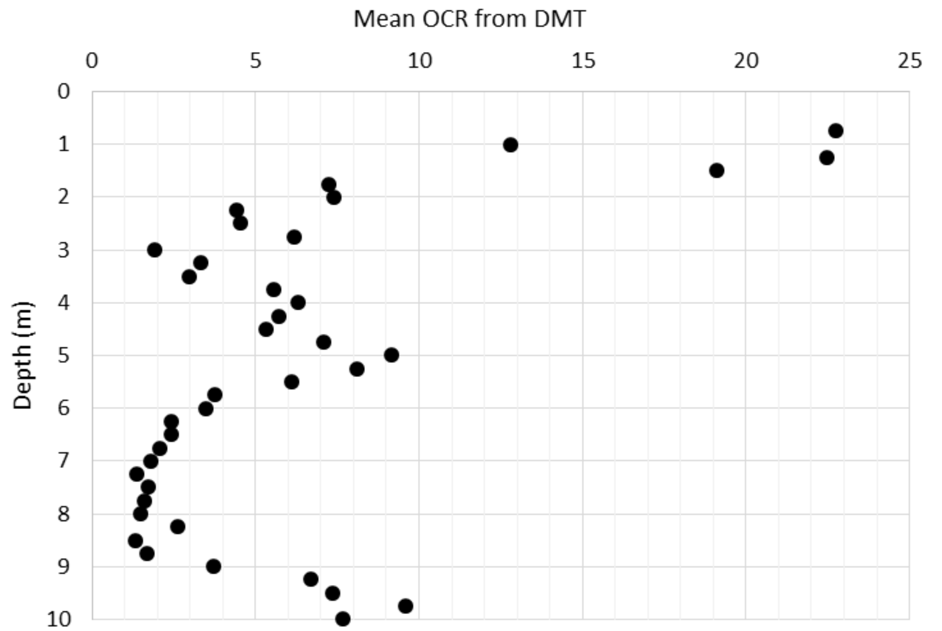


Figure 6: Depth (m) versus overconsolidation ratio as estimated from the DMT results at the Mountain Road field site.

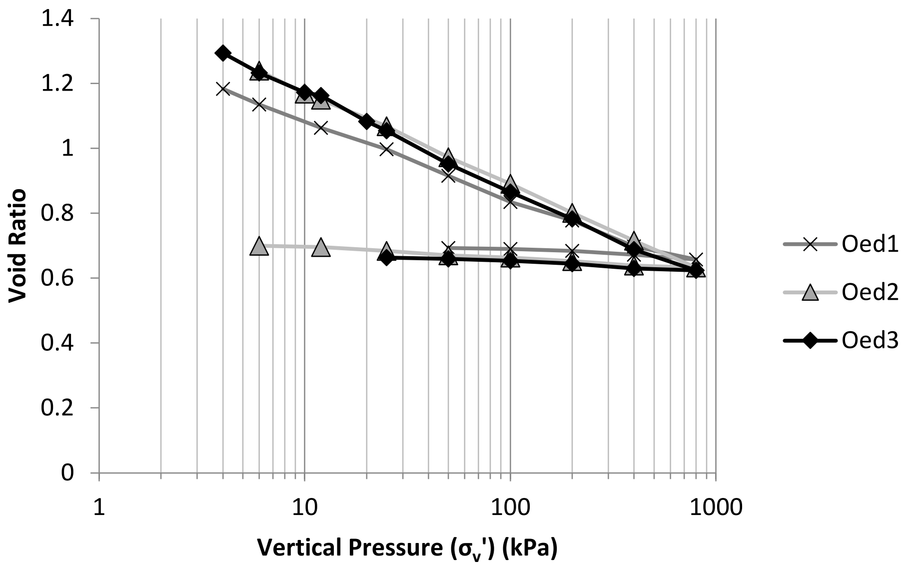
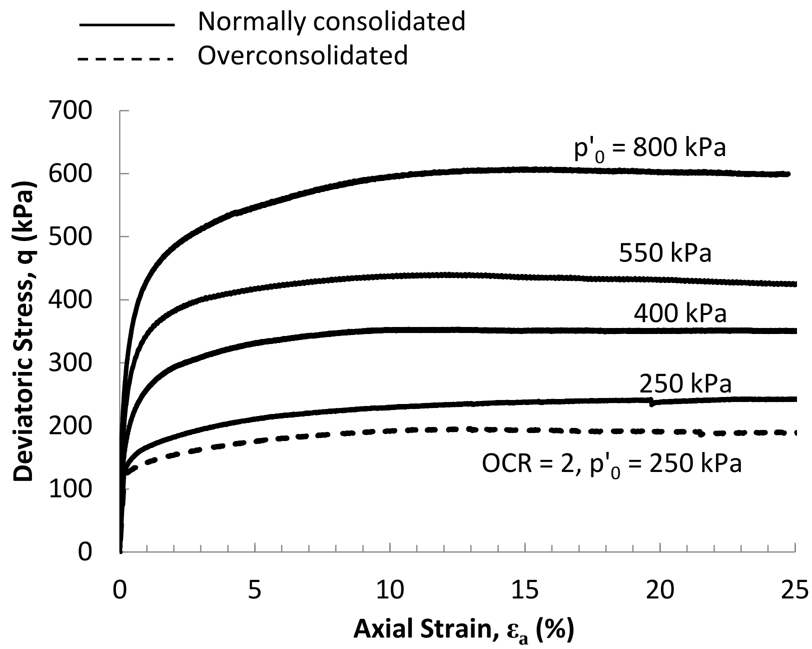
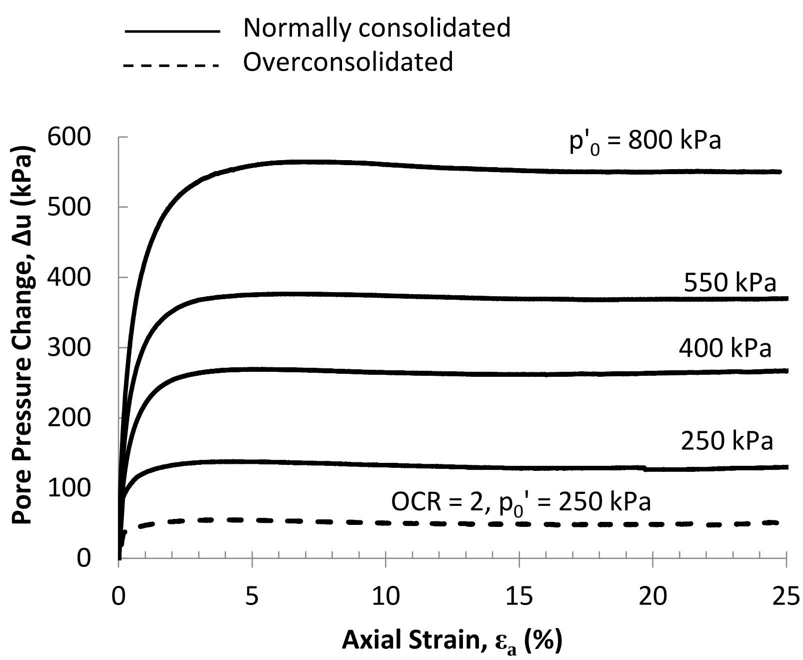


Figure 7: Intrinsic one-dimensional compression and swelling curves from three oedometer tests on reconstituted specimens.



(a)



(b)

Figure 8: (a) Deviatoric stress, q versus axial strain, ε_a; (b) pore pressure change, Δu versus axial strain, ε_a for the consolidated undrained triaxial tests on reconstituted specimens.

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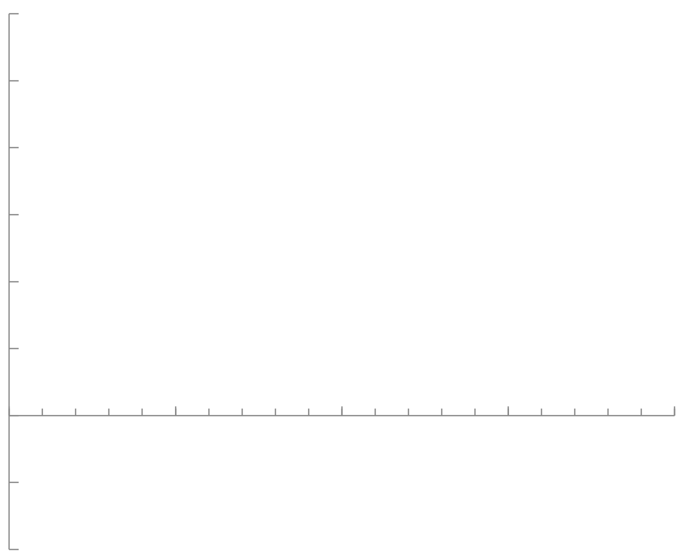
(a)



(b)

Figure 9: (a) Deviatoric stress, q versus axial strain, ε_a; (b) volumetric strain, ε_{vol} versus axial strain, ε_a for consolidated drained tests on reconstituted specimens.

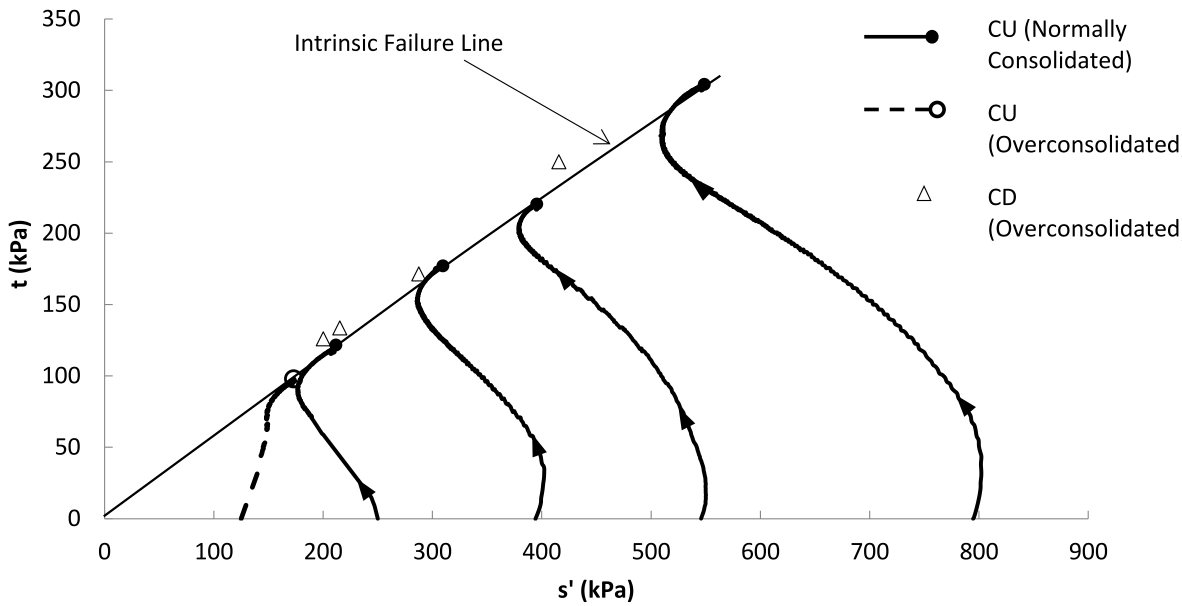
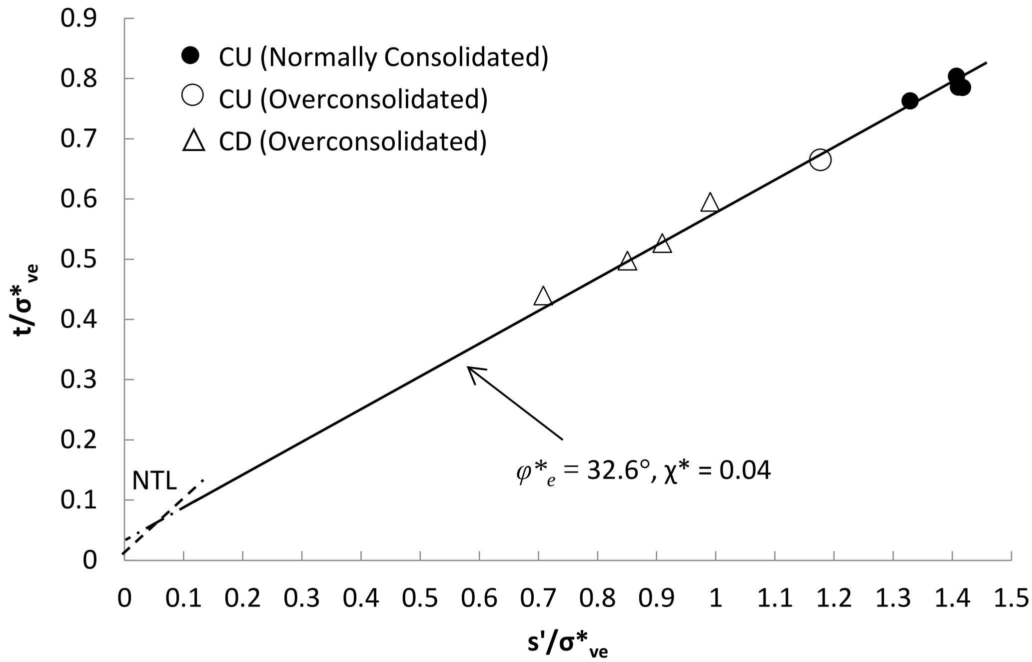


Figure 10: Peak strengths and undrained stress paths for the reconstituted specimens.



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785 **Figure 11: Intrinsic Hvorslev strength envelope for Northland Allochthon residual clay soil from**

786 **Mountain Road.**

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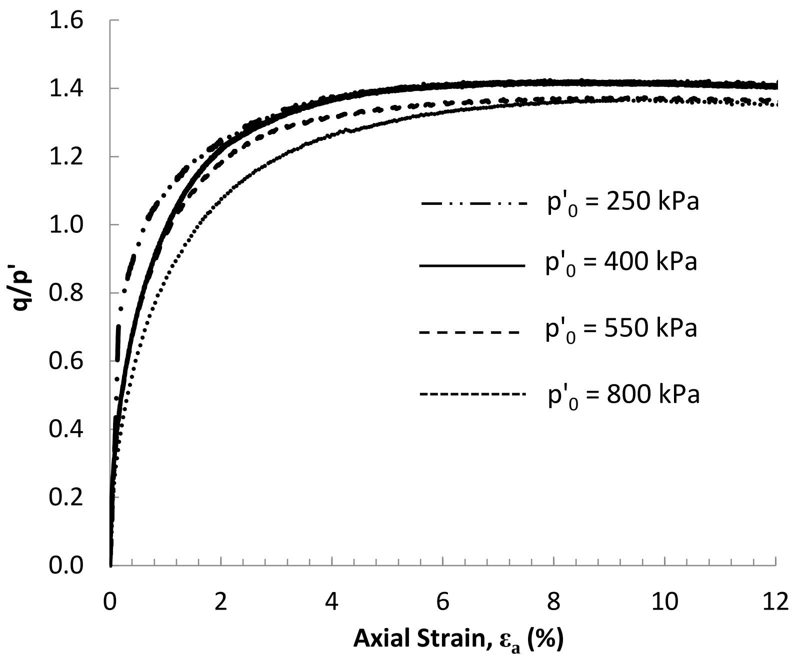
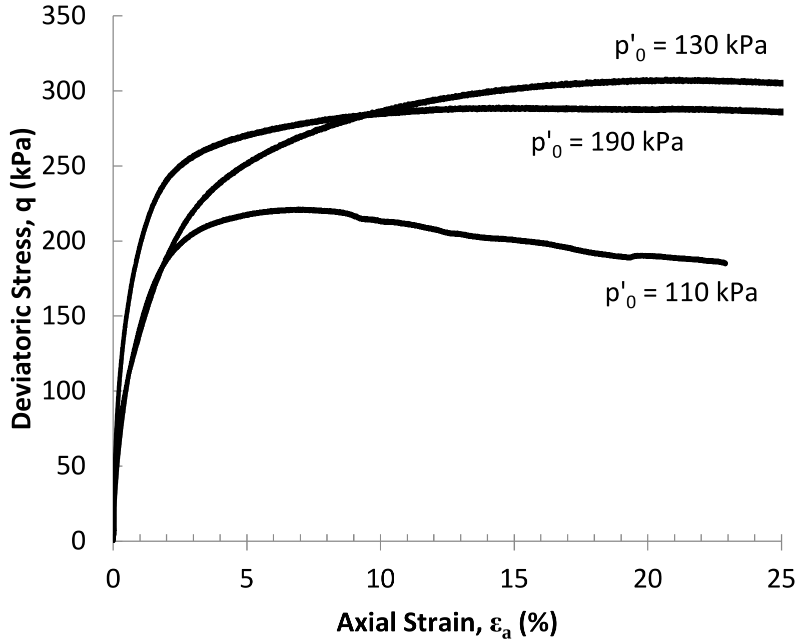
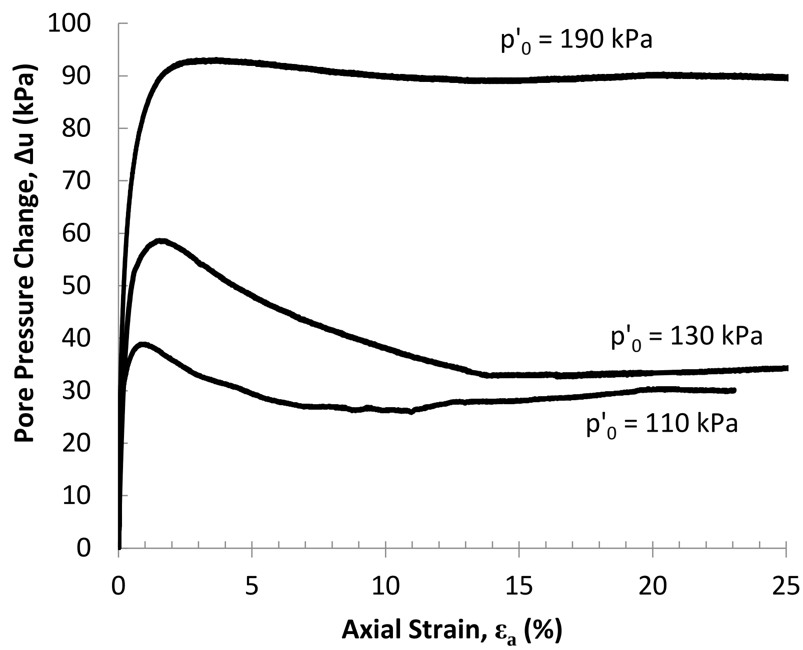


Figure 12: Stress ratio q/p' versus axial strain, ε_a for undrained tests on normally consolidated reconstituted specimens.



(a)



(b)

Figure 13: (a) Deviatoric stress, q versus axial strain, ε_a; (b) pore pressure change, Δu versus axial strain, ε_a for consolidated undrained triaxial tests on intact specimens.

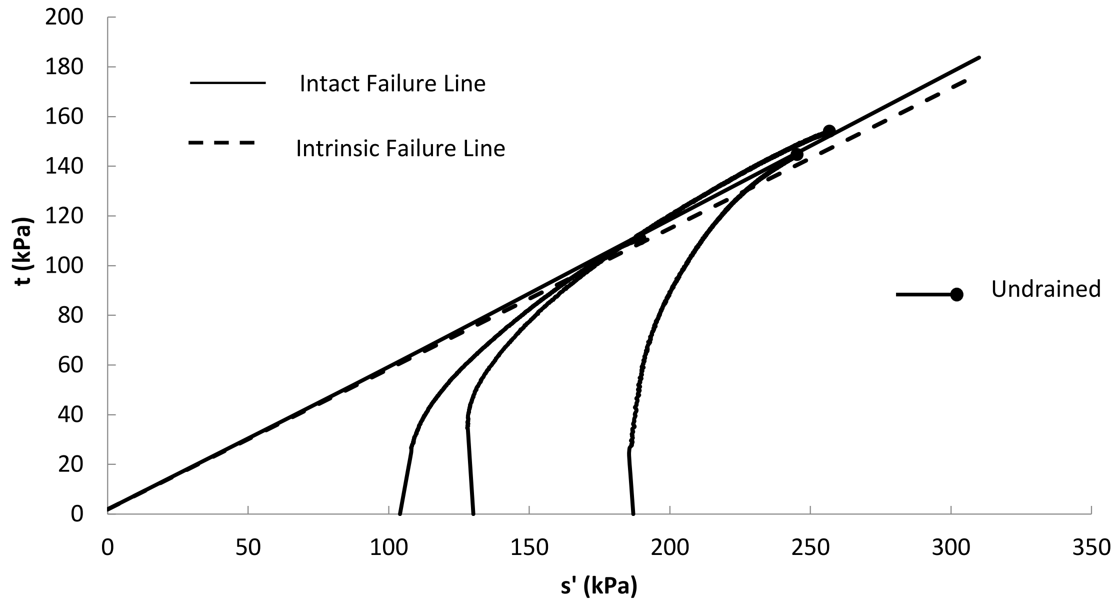


Figure 14: Peak strengths and undrained stress paths for the intact specimens.

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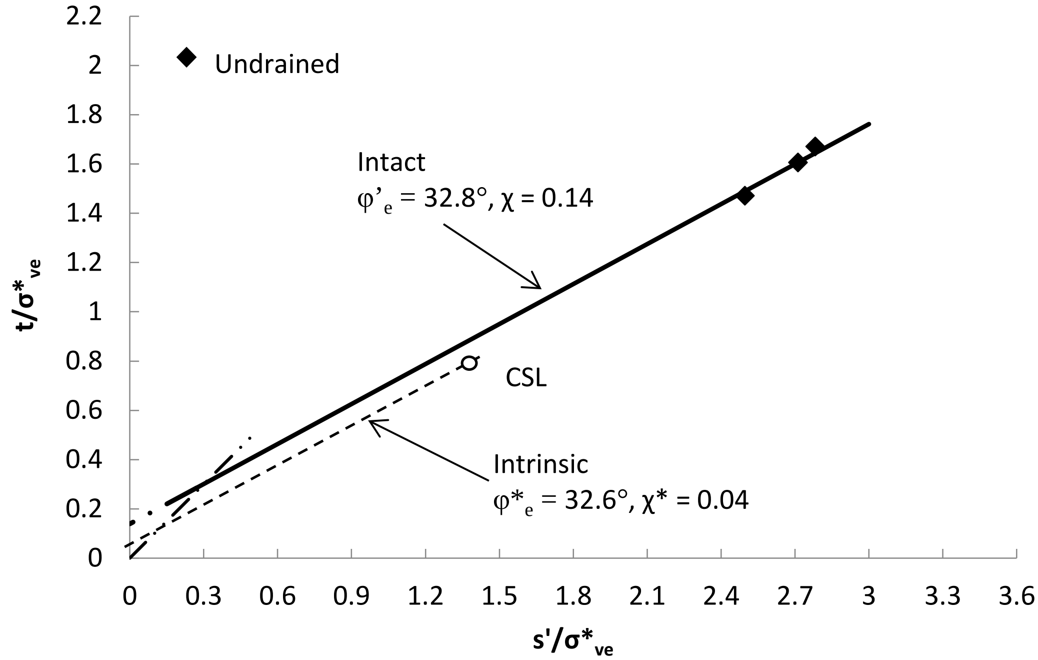

805 **Figure 15: Comparison of intact and intrinsic Hvorslev failure envelopes.**

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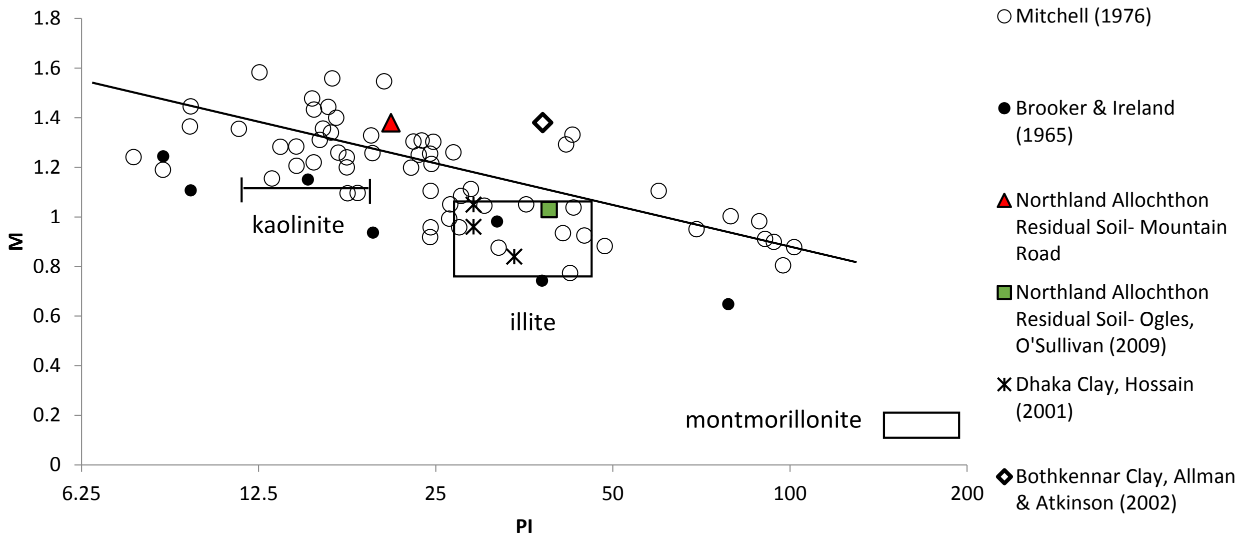

Figure 16: Relationship between M and plasticity index (PI) for different clays, modified after Muir-Wood (1990).