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25 **Abstract**

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

40

41 **Notation**

42	$C_c$	compression index
43	$C_s$	swelling index
44	$E$	Young's modulus
45	$e_L$	void ratio at liquid limit
46	$\varepsilon_a$	axial strain
47	$\varepsilon_{vol}$	volumetric strain
48	$\varphi_{crit}$	critical state angle of internal shearing resistance
49	$\varphi_e^*$	Hvorslev true angle of shearing resistance
50	$\varphi_e$	Hvorslev angle of shearing resistance for intact soil
51	$\varphi_{res}$	residual angle of internal shearing resistance
52	$\varphi_{peak}$	peak angle of internal shearing resistance

53	$G_s$	specific gravity
54	$\gamma_s$	Shear strain
55	$\kappa$	slope of a swelling line based on a natural log scale
56	$\lambda$	gradient of normal compression and critical state line based on a natural log scale
57	$M$	gradient of critical state line
58	$p'_0$	effective consolidation pressure
59	PI	plasticity index
60	$p'$	mean effective stress
61	$q$	deviatoric stress
62	$\sigma^*_{ve}$	equivalent stress on intrinsic compression line corresponding to void ratio of soil
63	$\sigma'_a$	effective axial stress
64	$\sigma'_r$	effective radial stress
65	$s'$	$(\sigma'_a + \sigma'_r)/2$
66	$t$	$(\sigma'_a - \sigma'_r)/2$
67	$\Delta u$	change in pore pressure
68	$v$	specific volume
69	$w_{PL}$	plastic limit
70	$w_{LL}$	liquid limit
71	$w_0$	natural water content
72	$\chi$	Hvorslev cohesion intercept
73	$\chi^*$	intrinsic Hvorslev cohesion intercept
74		

75 **Introduction**

76 Northland Allochthon residual clay is found in the northwest of the North Island of New Zealand. This soil  
77 forms via in situ weathering of parent mudstone to relatively shallow depths of between 1.5 m and 8 m  
78 (Winkler, 2003). Slope instability is very common within the saturated soil, at gradients as low as 8° (Glade,  
79 1998; Tonkin & Taylor, 2006; Harris et al. 2011). This results in many small, often shallow, landslides that  
80 particularly affect the road network (East & George, 2001), leading to significant regional economic losses  
81 in the order of several millions of dollars per year (Glade 1998; NIWA and GNS, 2009). With precipitation  
82 events that may exceed 40 mm/hour and several hundred mm per day (NIWA, 2015), instability is usually  
83 triggered by high rainfall (Glade, 1998), and may occur in the form of successive slides at low angles  
84 (O'Sullivan, 2009; Taterniuk, 2014). In spite of this, while there have been concerted efforts to understand  
85 and describe the geological provenance of the soils and their parent rocks (see Spörli & Hayward (2002)  
86 and references therein), only limited geotechnical laboratory data has been published on Northland  
87 Allochthon soils. This has been in part due to the difficulty of acquiring high quality undisturbed soil, which  
88 is friable in its natural state (O'Sullivan, 2009; Taterniuk, 2014) and in part because of an historically local-  
89 empirical approach to geotechnical design, in the main guided by in situ rather than laboratory testing (East  
90 & George, 2001).

91 It is difficult to establish clear mechanical-empirical relationships for many residual soils to assist in their  
92 engineering characterisation (Geological Society Working Party, 1990). In addition, in terms of qualitative  
93 mesoscale strength of in situ material, the presence of soil structure can produce greater or lesser strength  
94 in intact soils compared to soils reconstituted for routine laboratory testing. An increase in strength may be  
95 due to microstructural ageing and chemical weathering, while a decrease in strength in overconsolidated  
96 fissured soils may be due to the presence of such fissures at the mesoscale (Hosseini Kamal et al. 2014).  
97 As residual soils, Northland Allochthon soils are highly weathered (Class E "Residual" according to BS  
98 5930:2015). O'Sullivan (2009) has attested to the influence of fissuring on these soils, suggesting that in  
99 situ weathering may impart forms of structure in the transition zone from rock to soil, namely a decreased  
100 resistance to shearing and a change in the state boundary surface (Cafaro and Cotecchia; 2001).

101

## 102 **Soil testing framework**

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

108 As an alternative, Cotecchia & Chandler (2000) present a sensitivity framework for clays where either stress  
109 or strength sensitivity uniquely defines the yield stresses, providing a single parameter by which the clay  
110 structure may be represented. In order to use this framework, both reconstituted and intact oedometer tests  
111 must be available. However, the conventional oedometer test has been found to have limited usefulness in  
112 testing of stiff residual soils, such that for intact Auckland residual clay, Pender et al. (2003) found lower  
113 values of stiffness modulus from oedometer tests than from conventional triaxial tests while O'Sullivan  
114 (2009) found in difficulties interpreting the end of primary consolidation and atypical results in oedometer  
115 tests on intact Northland Allochthon residual clay. In addition to difficulties related to residual soil  
116 characteristics, other issues can arise. For instance, Gasparre (2005) found the calculation of stress  
117 sensitivity for London clay to be problematic because the presumed post yield behaviour diverged from the  
118 corresponding intrinsic compression curve at the end of her tests.

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

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

129 clays. The aim is to examine the potential for a framework that has been developed previously for  
130 sedimentary clays to be used for Northland Allochthon residual soil.

131

### 132 **Geology and context of Northland Allochthon residual soil**

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

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

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

145

146 Here we focus on the fersiallitic residual soils derived from Hukerenui mudstone in the Mangakahia  
147 complex, which typically comprise very soft to stiff, plastic, light coloured, clayey silts and silty clays with  
148 some sands or gravel sized clasts (Lentfer, 2007). Between the residual soil and the parent rock lies a  
149 transition zone (Figure 2), which tends to retain the sheared fabric of the underlying rock, and may contain  
150 gravel sized clasts of the parent rock. The residual zone is generally lighter in colour and has a lower  
151 permeability (Winkler, 2003) than this transition zone. The thickness of the residual soil layer varies between  
152 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  
153 that only the residual zone soil is discussed in detail in this paper.

154 The Northland region is classed as temperate, with no dry season and warm summers (Peel et al. 2007).  
155 Mean annual rainfall varies from 1000 – 2000 mm across the region, hence, this fine grained residual soil  
156 generally remains saturated, year round, to near the ground surface (Wesley, 2010). The Hukerenui

157 mudstone has been found to contain a significant amount of smectite (89%), with 8% illite, kaolinite < 3%  
158 and chlorite <1% (Lentfer, 2007), which can result in the clay being highly plastic.

159 In this paper, we examine data from five field sites. These are: Mountain Road, Kaeo, Ogles, Puhoi, and  
160 Silverdale. Figure 1 shows the five site locations, spread over a linear distance of nearly 250 km, and the  
161 geology at each site. Triaxial testing and detailed characterisation were carried out on soil from Mountain  
162 Road, while more limited classification testing was carried out on soil obtained from the Kaeo site,  
163 supplemented by data obtained by O’Sullivan (2009) for Ogles, Lentfer (2007) and Melrose and Willis  
164 (2010) for Silverdale, and the Further North Alliance (2013) for Puhoi. While the grouping of unpublished  
165 data in this way from different sites may not be considered “best practice”, here we take a pragmatic  
166 approach to provide as much information as possible, given the sparsity of research on this soil. In doing  
167 so, we show that while the region may be geologically complex, this does not necessarily translate to  
168 geotechnical complexity, at least in terms of physical properties of the soil.

169

#### 170 **Atterberg limit tests and other soil properties**

171 Atterberg plastic and liquid limit index tests conducted on samples obtained from Mountain Road and Kaeo  
172 sites were combined with the data available from Ogles (O’Sullivan, 2009) as well as the consulting reports  
173 and unpublished theses (Lentfer, 2007; Melrose and Willis, 2010; Further North Alliance, 2013) – Figure 3.  
174 Collectively they show that the soils at the five sites range from medium to high plasticity and despite the  
175 large distances between sites, all plot on or near the A-line, indicating their relatedness (Muir-Wood, 1990).  
176 Particle size distributions (Figure 4), were determined for specimens from Mountain Road and Kaeo sites  
177 via sieve analysis (New Zealand Standard, 1986a) and hydrometer testing (New Zealand Standard, 1986b).  
178 Kaeo samples are seen to be more well-graded and have a significantly higher clay content than Mountain  
179 Road samples which are silt dominated and least plastic (Figure 3). Note that there is no relationship  
180 between index properties and geographical location within Northland (compare Figures 1 and 3) – that is,  
181 Mountain Road, located most centrally geographically but towards the south, has the lowest PI, while Puhoi,  
182 slightly further south again appears to be most weathered and most plastic.

183 Index properties for soil from the Mountain Road site are shown in Table 1, along with specific gravity and  
184 organic content. The data was obtained from laboratory testing (ASTM, 2010a; ASTM, 2010b; Germaine

185 and Germaine, 2009) namely, specific gravity ( $G_s$ ), natural water content ( $w_0$ ), and organic content by loss  
186 of ignition.

187

### 188 **Site Sampling**

189 To obtain high quality laboratory test samples, while it is widely reported that undisturbed samples cut from  
190 carefully extracted blocks provide the best results, the high potential for shrink-swell behaviour, large  
191 number of lithic fragments and delicate friable structure of Northland Allochthon residual soil make it difficult  
192 to trim them without excessive pitting. Therefore, a different approach was taken which is similar to that  
193 taken by O'Sullivan (2009). Samples were collected in 72 mm diameter thin-walled Shelby tubes so that  
194 the soil specimens could be extruded and placed directly into a triaxial cell with little additional trimming.  
195 The adopted sampling procedure proceeded as follows. A shallow pit with a level base was excavated to  
196 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  
197 level at Kaeo. The 450 mm long, 72 mm diameter stainless steel tubes were hydraulically jacked into the  
198 ground using a specially constructed rig and jacking frame at a controlled rate of around 20 mm / min with  
199 the excavator buck used to provide a vertical reaction (Figure 5). The tubes were then manually excavated  
200 with care, the ends sealed with wax then double wrapped in plastic and fully sealed for transport. In total,  
201 six tube samples were taken from Mountain Road and a further six from Kaeo. These included soils from  
202 both the residual and transition zones. This method of sampling eliminates the drilling or boring stage, which  
203 is recognized to result in the greatest disturbance (Ladd and DeGroot, 2003). Sample disturbance may  
204 nevertheless have impacted the behaviour of the intact soil: Lunne et al (1997) showed that use of tube  
205 samplers may result in a decreased peak and residual stress measurement in clay, due primarily to soil  
206 destructuring (DeGroot et al 2005). In the current study, it is recognized that the effects of this may result  
207 in a less pronounced difference between the intact and undisturbed properties of the soil.

208

### 209 **In Situ Flat Plate Dilatometer Testing**

210 In order to obtain data on the in situ stress history of the soil, Flat Plate Dilatometer tests (DMT) were  
211 conducted at Mountain Road (Marchetti, 1980; 2015). Three DMTs, denoted sDMT 5, 6, and 7 were located

212 approximately 5 m from the extraction point for the samples (other tests were carried out further away and  
213 results are not used here, although results produced similar trends).

214 The horizontal stress index,  $K_D$  is one of the key intermediate parameters obtained from the DMT (Marchetti,  
215 1980). The overconsolidation ratio (OCR), defined as the ratio of maximum vertical effective stress to  
216 current effective stress, may be estimated from  $K_D$  through Equation 1 (Marchetti, 2015):

$$217 \quad OCR = (0.5 K_D)^{1.56} \quad 1.$$

218  
219 As shown in Figure 6, despite there being nothing in the geologic history of this residual soil to suggest that  
220 the present overburden stress of the soil has been exceeded in the past (Isaac et al., 1994) it exhibits mean  
221 OCR's well above unity throughout its depth, with a particularly marked high value near the surface. The  
222 potential reasons for this are discussed further in the "preconsolidation" section at the end of this paper.

223

#### 224 **Laboratory mechanical testing**

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

228

#### 229 ***Triaxial testing***

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

240 Reconstituted specimens were prepared from a slurry of the Mountain Road soil at a water content of 1.25  
241 to 1.5 times its liquid limit (Burland, 1990). The soil was mixed with water for a minimum of 6 hours and  
242 then slowly poured into and consolidated over time in a 150 mm long and 50.7 mm internal diameter  
243 consolidometer constructed for the purpose of obtaining 50mm diameter samples. Two-way drainage was  
244 allowed via perforated discs, porous stones and filter paper applied to each end. The consolidation device  
245 utilized the application of pressure to the top of the sample through the addition of weights and pressure  
246 was increased in stages according to the method for consolidation in oedometer testing (Head & Epps,  
247 2011). The change in height was monitored and plotted versus the log of time to ensure end of primary  
248 consolidation was reached before next load was added. All specimens were consolidated to 150 kPa  
249 vertical effective stress before extrusion. The method used for trimming and extruding the samples was the  
250 same as that used for the intact samples. Trimmings were used to determine the water content of the  
251 specimen at the start of each test.

252 Triaxial testing was performed using a GDS triaxial stress path cell with computer control. The room  
253 temperature was well-controlled and continuously monitored to ensure it did not vary by more than 0.5°C.  
254 Reconstituted specimens were 50 mm in diameter and 100 mm long, whilst undisturbed specimens were  
255 72 mm in diameter and 130 mm long. Stress path cell instrumentation included pore pressure and cell  
256 pressure transducers, and an internal load cell. Local displacements were measured by using axial  
257 submersible on-sample LVDTs (linearly variable differential transformers) for all specimens. Radial on-  
258 sample LVDTs were also used on all reconstituted specimens. Pore pressure measurement was carried  
259 out at the top of the specimen and drainage was provided at the bottom. Undisturbed specimens were  
260 tested using enlarged lubricated ends in order to improve uniformity at all strain levels.

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

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

276

### 277 ***Oedometer testing***

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

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

287

### 288 **Results of laboratory tests**

#### 289 ***Oedometer tests on reconstituted soil***

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

293 It is now useful to put the Northland soil into the context of other clays, albeit sedimentary ones. The mean  
294 slope of the ICL,  $\lambda$  of 0.121, lies between that of London clay ( $\lambda \approx 0.1$ ), and pure kaolin ( $\lambda \approx 0.2$ ) (Atkinson  
295 and Evans, 1985). The ratio between PI and  $\lambda$  is 176, and compares well to the mean value of 170 given

296 by Schofield and Wroth (1968) based on five clay soils. The  $\lambda/k$  value is approximately 8.1, which is larger  
297 than the range of values (2-5) given by Schofield and Wroth (1968) but similar to the value of 7.2 reported  
298 by Allman and Atkinson (1992) for Bothkennar soil, which is often described as a clay but is predominantly  
299 comprised of silt sized particles.

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

$$303 \quad C_c = 0.253e_L - 0.04 \quad 2.$$

304  
305 Where  $e_L$  is the void ratio at the liquid limit. Using this equation gives a  $C_c$  of 0.29 ( $\lambda$  of 0.126) for two  
306 samples of residual clayey silt soil from Mountain Road, which is consistent with the average  $C_c$  of 0.278  
307 ( $\lambda$  of 0.121) attained from the oedometer tests on reconstituted specimens.

308  
309 ***Triaxial tests on reconstituted soil***

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

321 Figure 9 shows corresponding triaxial data for the consolidated drained (CD) tests. The CD tests were  
322 performed on overconsolidated specimens and all displayed failure along shear bands after slight bulging.  
323 The specimens at OCR values of 5, 7 and 10 showed post-peak reductions in shear strength (Figure 9(a))

324 and corresponding contraction followed by dilation (Figure 9(b)), although OCR of 7 sample produced less  
325 dilation than the OCR of 5 sample, which was not expected. At the lowest OCR of 3, the behaviour was  
326 purely contractive and there was no peak in shear strength with increasing strain.

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

336 The intrinsic Hvorslev strength envelope represents the strength of the soil in the reconstituted state (Figure  
337 11). This envelope was obtained by normalizing the results from Figure 10 by using the void ratio of each  
338 sample at failure and the one-dimensional compression curves from test Oed2 (Figure 7). The Hvorslev  
339 true angle of shearing resistance ( $\phi^{*c}$ ) is  $32.6^\circ$ . The intrinsic Hvorslev cohesive intercept,  $\chi^*$ , is 0.04 and  
340 the value of  $s'/\sigma_{ve}^*$  at critical state is 1.42.

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

$$343 \quad q = Mp' \quad 3.$$

344

345 The value of  $\phi_{crit}$  in compression can be determined from Equation 4:

$$346 \quad \sin\phi'_{crit} = 3M/(6 + M) \quad 4.$$

347

348 Giving a value of  $34.2^\circ$  for  $\phi_{crit}$ , in line with the values of  $33.6^\circ$  to  $34.9^\circ$  from the intrinsic critical state line in  
349  $s'$ - $t$  space.

350

351 **Triaxial tests on undisturbed samples**

352 Three triaxial tests were performed on undisturbed specimens of Northland Allochthon residual clay from  
353 the Mountain Road site, at confining pressures of 110 kPa, 130 kPa and 190 kPa, respectively (Table 2).  
354 These confining pressures are representative of the in situ mean effective stress conditions of the soil  
355 specimens. As shown in Figure 13(a), the specimen tested at the lowest confining stress, 110 kPa, showed  
356 a gradual post peak reduction in strength, and developed a single failure surface, while the samples tested  
357 at 130 kPa and 190 kPa exhibited well-defined critical states, and failure of these samples took place after  
358 slight bulging. Although generally ductile, the test at 130 kPa exhibited a higher peak deviatoric stress than  
359 at 190kPa. Although this is not expected, this type of behaviour does occasionally occur in intact specimens  
360 due to their variability, and can be seen in the results on drained specimens of Todi clay in Burland et al.  
361 (1996). Despite this, all the samples exhibited positive pore pressure change (Figure 13(b)), with higher  
362 confining stresses resulting in higher development of pore pressure.

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

374

375 **Strength characteristics from the literature**

376 Little published data is available on the strength characteristics of Northland Allochthon residual soil.  
377 Available data from three field sites is summarized in Table 5. The data from Mountain Road is that obtained  
378 from this study. The values of the residual angle of shearing resistance ( $\phi_{res}$ ) for the Ogles and Silverdale

379 field sites were obtained from ring shear tests and peak and/or critical state parameters from triaxial testing  
380 (O'Sullivan, 2009; Harris, 2013).

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

387

## 388 **Discussion**

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

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

403

### 404 ***Variation in critical state parameters $\lambda$ and $\phi_{crit}$***

405 The CU tests on normally consolidated reconstituted specimens from Mountain Road reached a well-  
406 defined critical state. The mean critical state angle  $\phi'_{crit}$  of 34.2° is high for a clay, but is similar to that

407 attained for reconstituted Bothkennar soil (1.38) (Allman & Atkinson, 1992), which was attributed to its high  
408 silt content. The Northland Allochthon soil at Mountain Road has a silt content at over 60% (Figure 3).

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

413 Atkinson (1993) suggested that the intrinsic critical state parameters ( $\lambda$  and  $\phi_{crit}$ ) may vary due to differences  
414 in grading and mineralogy between samples, and depends primarily on the nature of the soil. The critical  
415 state parameters acquired by Hossain (2001) for tropical residual soil of Dhaka, Bangladesh show clear  
416 variability between the boreholes, which are considered to be in the same formation. For instance, samples  
417 of borehole 1 and 3 were highly oxidized and much more weathered compared to those of borehole 2.

418 Research by Rahardjo et al. (2004) and Rocchi & Coop (2015) on granitic residual soils provides further  
419 evidence for their properties varying geographically and with depth due to different degrees of weathering.  
420 In these cases, the peak and critical state angles of internal shearing resistance were found to reduce as  
421 the mean particle size decreased and PI increased. The variation in the values of  $\phi_{crit}$  found between Ogles  
422 (O'Sullivan, 2009) and Mountain Road are in line with these observations.

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

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

430 
$$\sin\phi' = 0.35 - 0.1\ln(PI) \quad 5.$$

431

432 And  $\phi'$  is related to  $M$  according to Equation 4.

433 Using Equation 5,  $\phi_{crit}$  should be around  $30^\circ$  for the Mountain Road site and around  $27^\circ$  for the Ogles No.3  
434 site (compared to  $34^\circ$  and  $26^\circ$  from triaxial tests, respectively). The correlation, though loose, demonstrates  
435 that a lower PI returns a higher  $\phi_{crit}$ .

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

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

456 Correlations between index properties and shear strength for many residual soils have been found to be  
457 difficult (Geological Society Working Party, 1990). However, the results from Lentfer (2007) on the  
458 Hukerenui mudstone mineralogy, and the clear reduction in internal angle of shearing resistance from  
459 critical state to residual for this soil is indicative of a dominance of platy clay minerals, indicating an  
460 absence of allophanes and halloysites. The presence of clay minerals may be responsible for the

461 particularly complex behaviour for which tropical residual soils are known apply (Geological Society  
462 Working Party, 1990). Hence as the mineralogy of Northland Allochthon residual clay soil is similar to many  
463 sedimentary soils, this may be a contributing factor as to why correlations for sedimentary clays are found  
464 to be applicable here.

465

#### 466 ***Preconsolidation***

467 There is nothing in the geologic history of the residual soil derived from the Northland Allochthon rock (Isaac  
468 et al. 1994) to suggest that the present overburden stress of the soil has been exceeded in the past. Wesley  
469 (1990) studied three different residual soils, and found that they all behaved as moderately or heavily  
470 overconsolidated soils, but similarly to the Northland Allochthon residual clay soil, at least two of them had  
471 never been subjected to overconsolidation by preloading. Since “preconsolidation” pressures of such soils  
472 do not bear relation to overburden history, it is therefore likely to be the result of weathering processes. For  
473 this reason, the preconsolidation pressure found for residual soils is often termed pseudo-preconsolidation  
474 pressure (Wesley, 1990). The mechanisms that could lead to pseudo-preconsolidation (OCR higher than  
475 1) include desiccation (especially near the surface), drained creep (long term secondary compression), and  
476 structuration which can be due to physiochemical processes (natural cementation due to carbonates and  
477 silica, bonding due to ion exchange) (Mitchell and Soga, 2005). The slightly dilatant behaviour of the intact  
478 specimens of Northland Allochthon residual clay during undrained loading shows that they behave as  
479 overconsolidated at these confining pressures, providing substantiation of a pseudo-preconsolidation  
480 pressure in this soil. Other authors (Futai et al. 2004; Wang & Yan, 2006) have also noted overconsolidated  
481 soil behaviour in residual soils at confining pressures of 400 kPa or less.

482

#### 483 **Conclusions**

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

488 Oedometer and triaxial tests were carried out on reconstituted and intact soil from one site, Mountain Road,  
489 in order to determine how the soil behaved with respect to Burland et al's (1990) framework developed for

490 natural sedimentary clays. The results of these tests showed that this framework is appropriate for this soil,  
491 despite it being classed as a residual soil. The Hvorslev cohesive intercept for the intact material,  $\chi$ , was  
492 found to be 0.18, giving a ratio of  $\chi/\chi^*$  of 4.5. The ratio of the normalized strengths at intrinsic critical strength  
493 ( $T$ ) is 1.2, indicating that the intact soil is stronger than that of the reconstituted soil. The value of  $\phi_e$  of  $32.8^\circ$   
494 was only  $0.2^\circ$  more than  $\phi_e^*$ , however, suggesting that the difference in strength is cohesive rather than  
495 frictional in nature and hence, bonding plays a role in the strength of the intact soil. The undrained stress  
496 paths of the intact specimens of Northland Allochthon residual clay demonstrated slightly dilatant behaviour,  
497 suggesting a pseudo-preconsolidation pressure which is sometimes seen in residual soils.

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

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

518

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523

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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

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**Table 2: Summary of triaxial tests.**

Specimen Type	Name	CD/CU	Effective Stress (kPa)	Consolidation 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

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**Table 3: Parameters acquired from oedometer results.**

Test #	$C_c$	$C_s$	$\lambda$	$\kappa$
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

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**Table 4: Normalization parameters of in situ triaxial tests.**

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

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

Parameter	Units	Field Site	Mountain Road	Ogles <sup>1</sup>	Silverdale <sup>2</sup>	Silverdale <sup>3</sup>
		Complex/Formation	Undifferentiated Mélange, predominantly Mangakahia Complex Mudstones	Mangakahia Complex - Hukerenui Mudstone	Mangakahia Complex - Whangai Formation	Mangakahia Complex - Whangai Formation
$c'$	kN/m <sup>2</sup>		2.5	6	0	1
$\phi'_{peak}$	°		35.6	30	36	27
$\phi'_{crit}$	°		34.2	26	-	-
<b>M</b>			1.38	1.03	-	-
$\phi_{res}$	°		-	11 - 14	19	-

<sup>1</sup>O'Sullivan (2009)<sup>2</sup>Harris (2013)<sup>3</sup>Tilsley (1998) as cited in Harris (2013)

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

Soil Type	$\lambda$	<b>M</b>	$\phi_{crit}$ (°)
Glacial till <sup>1</sup>	0.09	1.18	29.5
London clay <sup>1</sup>	0.16	0.89	22.8
Kaolin clay <sup>1</sup>	0.19	1	25.4
Bothkennar clay <sup>2</sup>	0.18	1.38	34.1
Tropical residual clay (Dhaka- Borehole 1) <sup>3</sup>	0.07	1.05	25.8-26.5
Tropical residual clay (Dhaka- Borehole 2) <sup>3</sup>	0.06	0.96	24.2-24.4
Tropical residual clay (Dhaka- Borehole 3) <sup>3</sup>	0.05	0.84	21.6
Northland Allochthon residual clay (Mountain Road)	0.12	1.38	34.2

<sup>1</sup>Atkinson (1993)<sup>2</sup>Allman & Atkinson (1992)<sup>3</sup>Hossain (2001)

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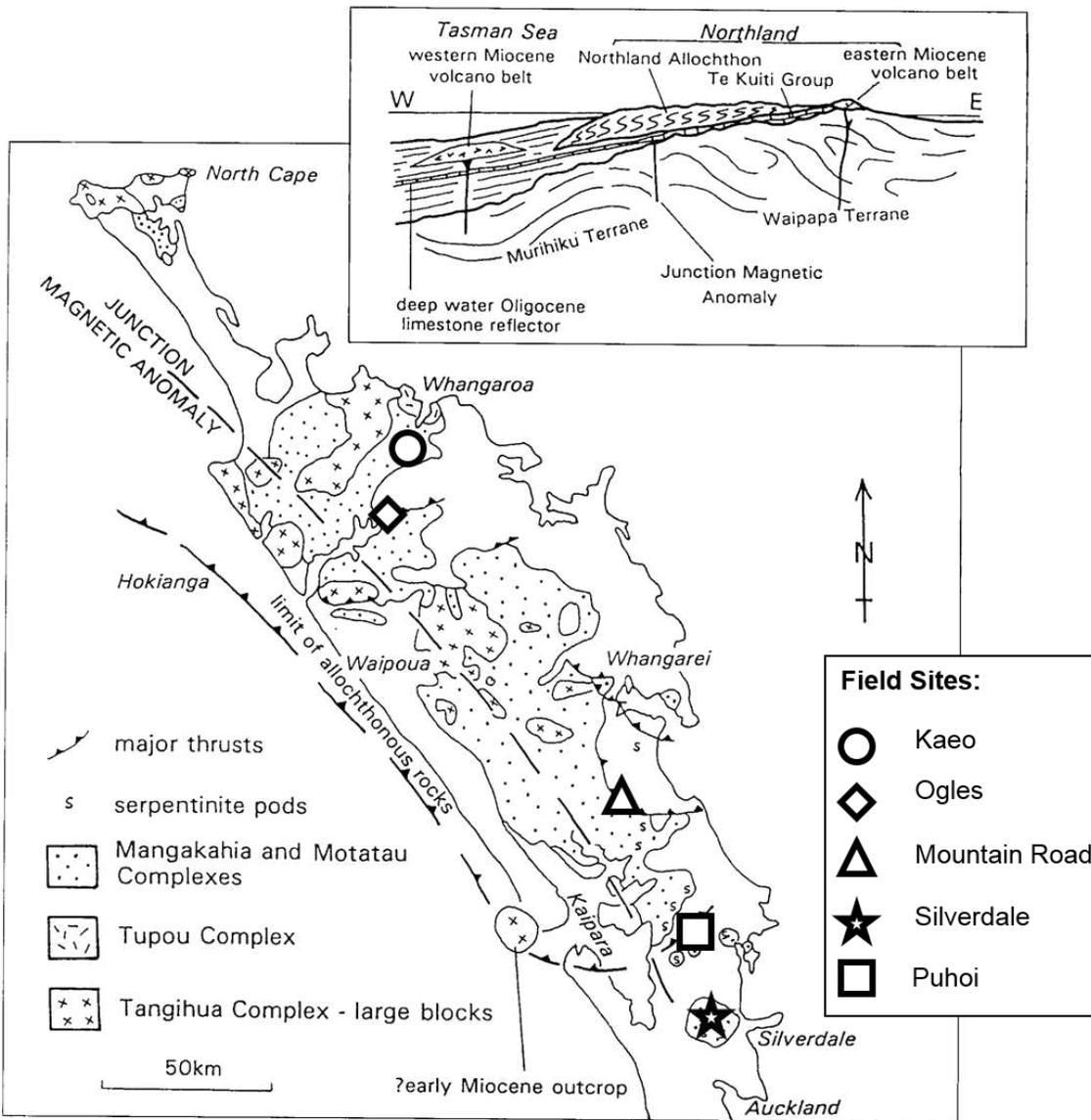
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 715 **Hayward et al. (1989).**

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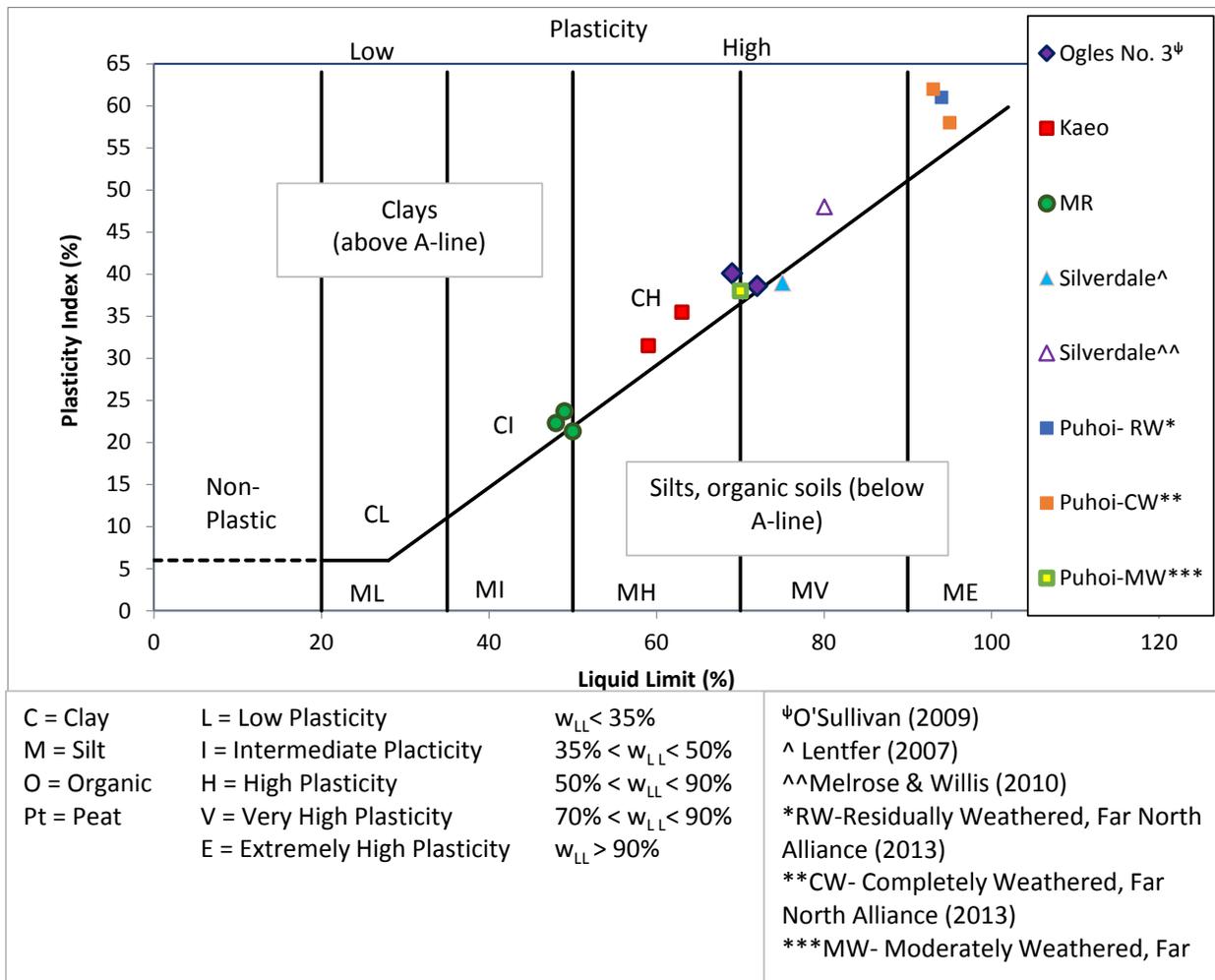


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**Figure 2: Upper (residual) and lower (transition) soil zones at Mountain Road.**

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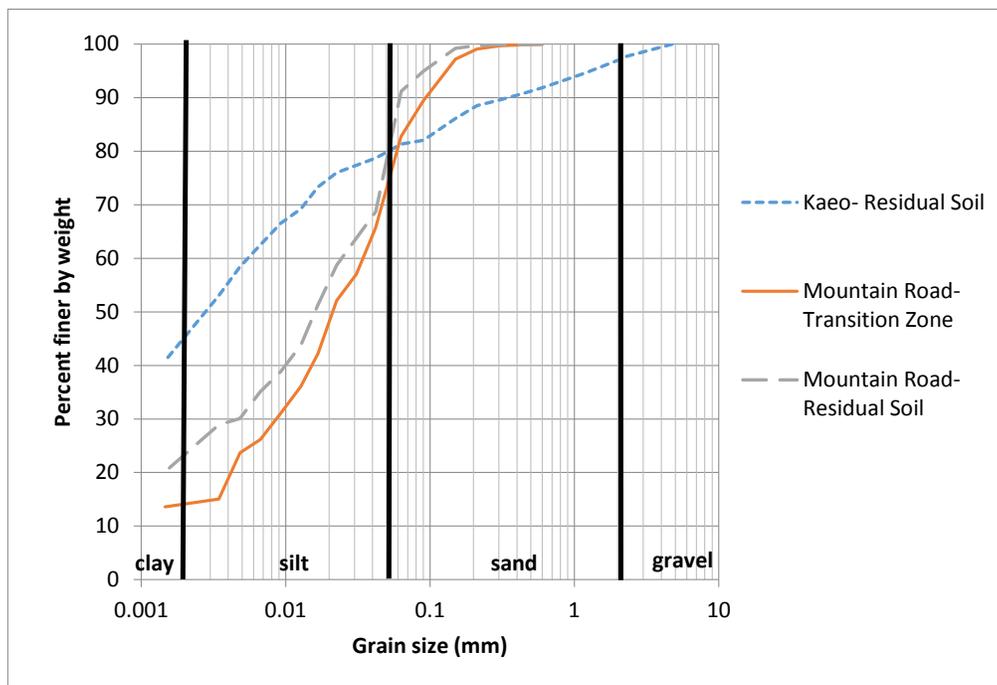
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**Figure 3: Results of Atterberg limits on Northland Allochthon residual clay soil.**

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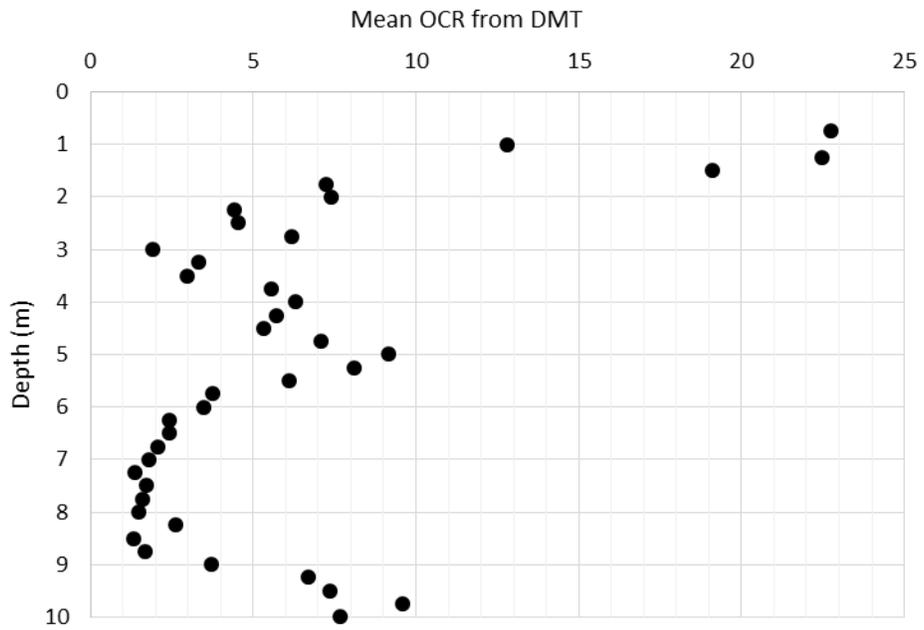
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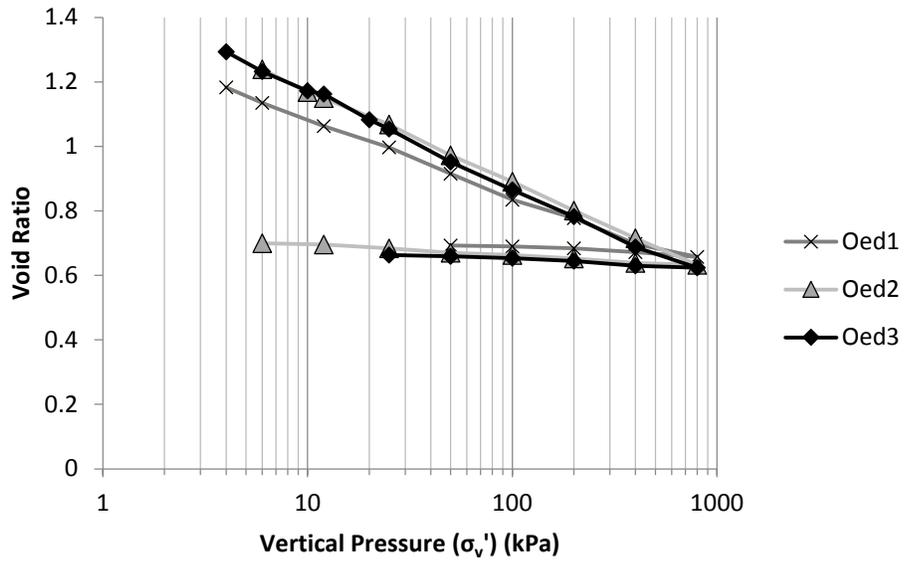
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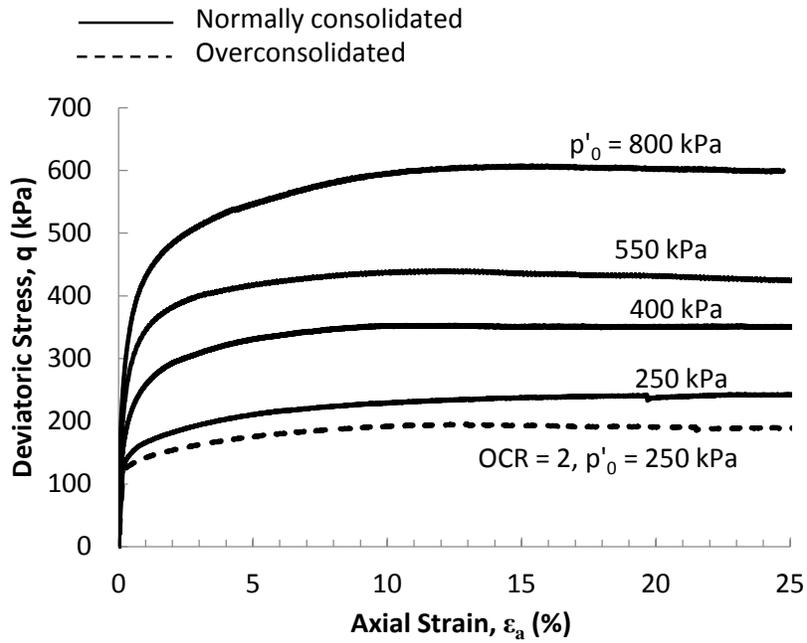
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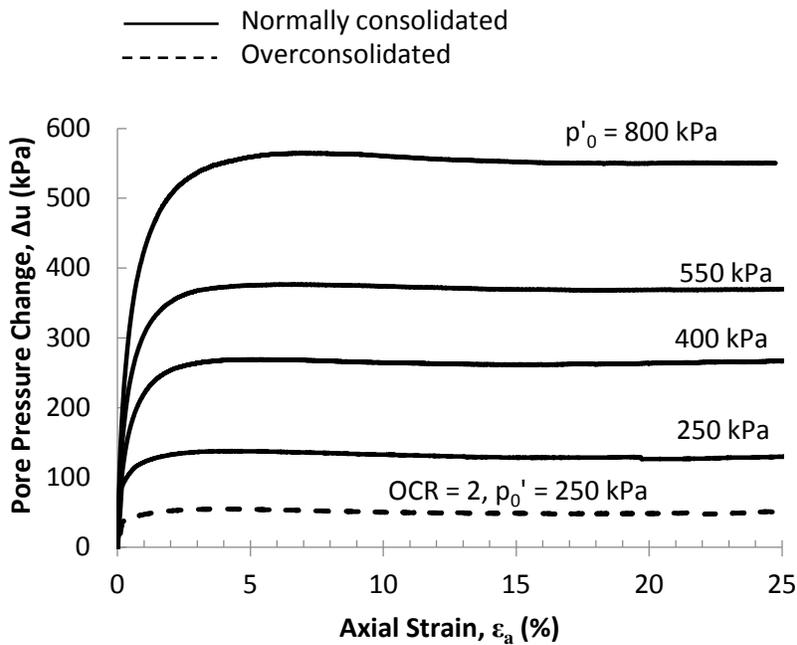


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**Figure 7: Intrinsic one-dimensional compression and swelling curves from three oedometer tests on reconstituted specimens.**



(a)



(b)

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

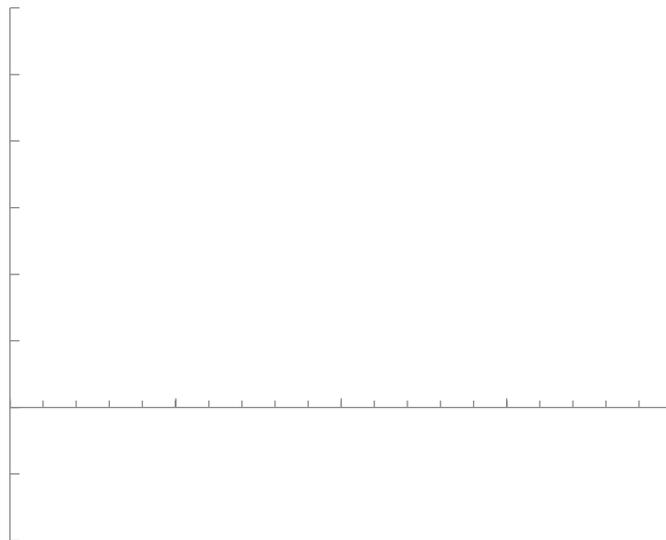




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(a)



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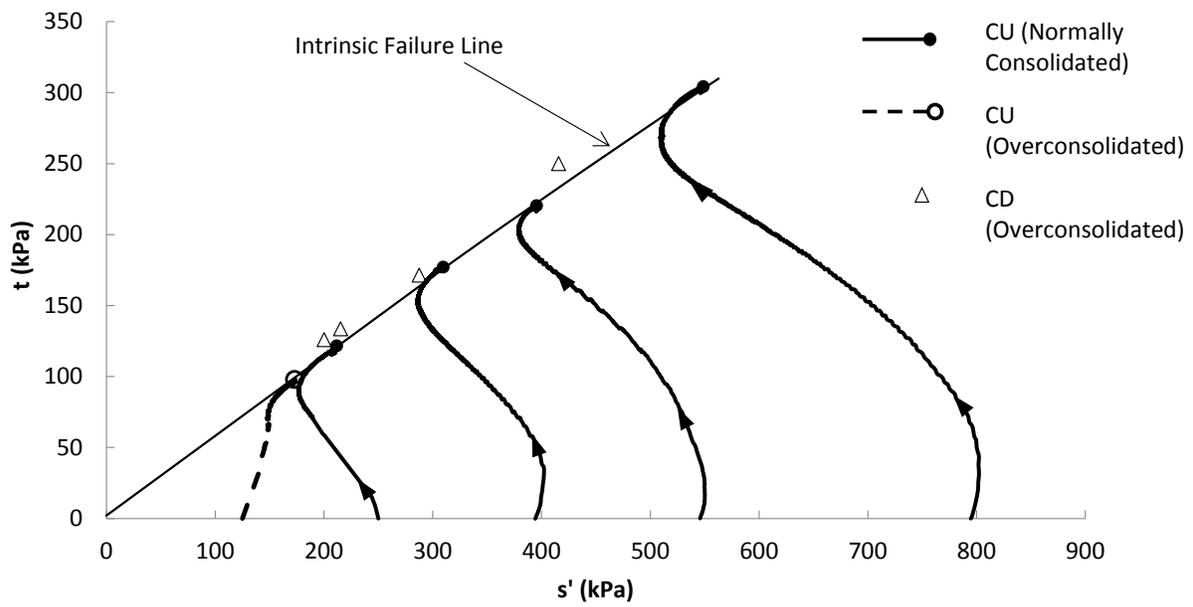
(b)

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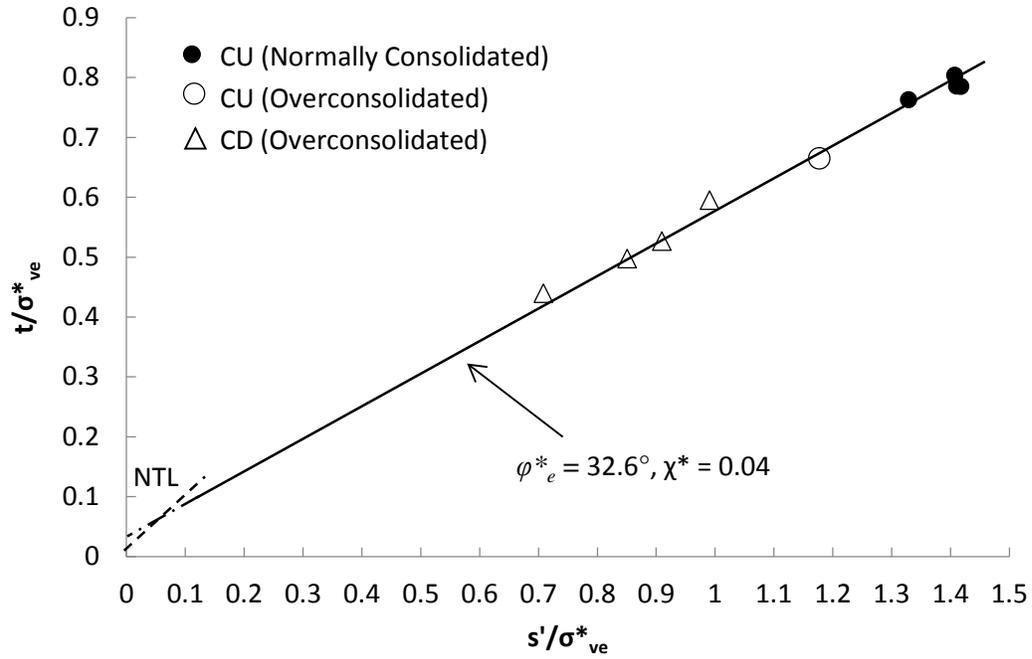


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**Figure 10: Peak strengths and undrained stress paths for the reconstituted specimens.**

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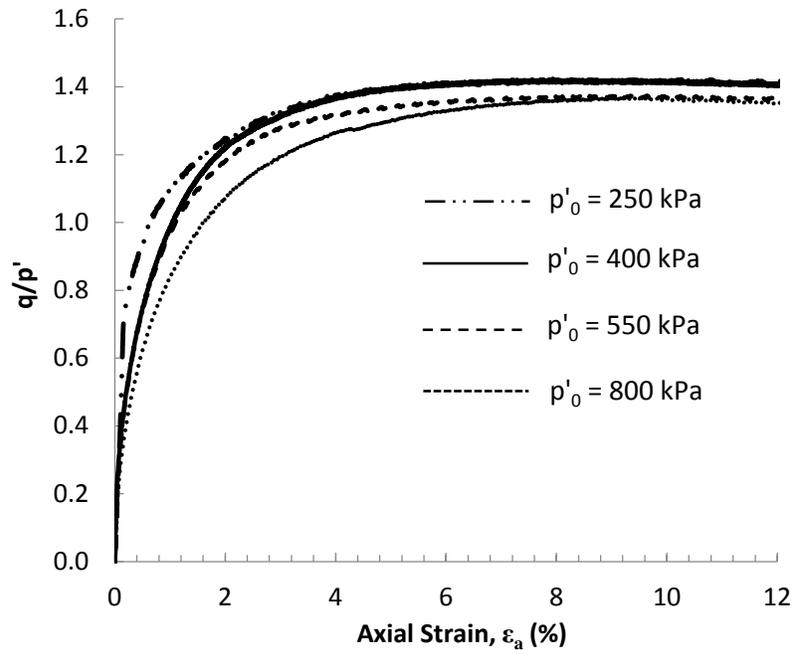
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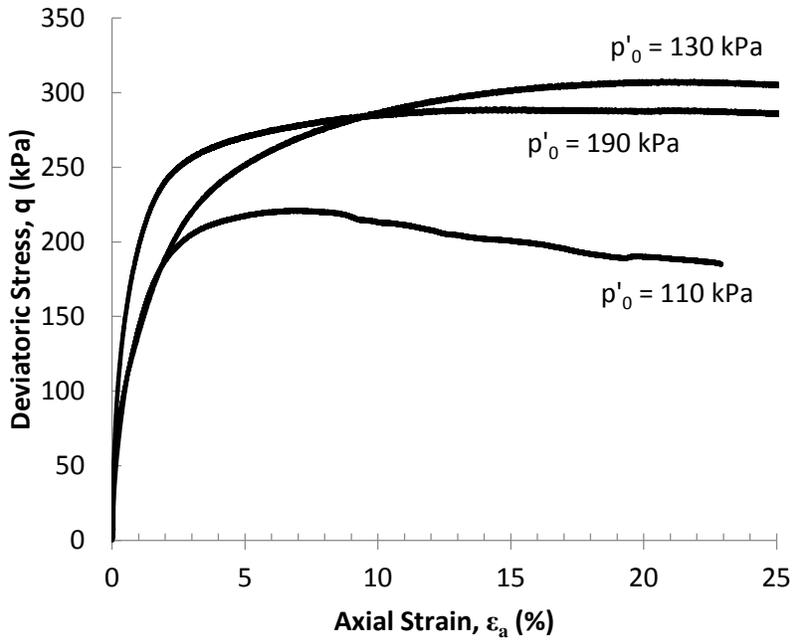
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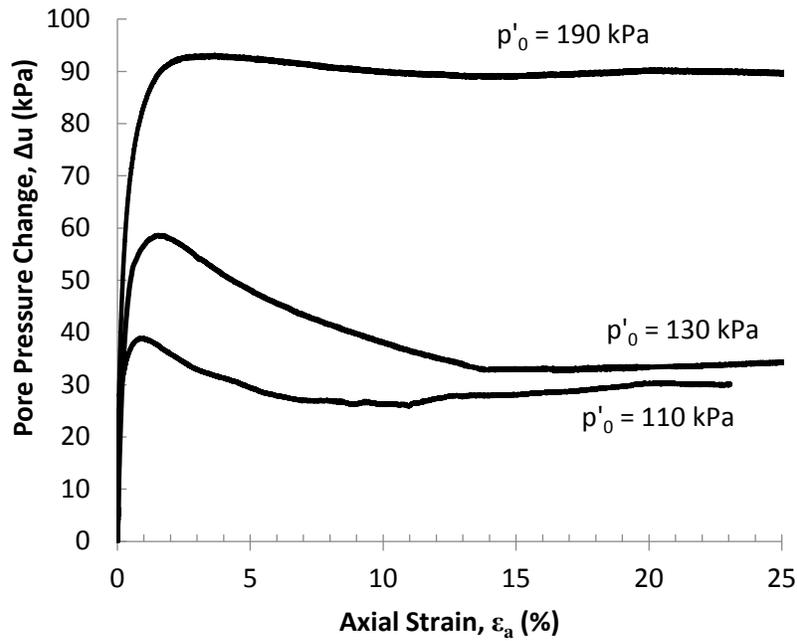
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(a)



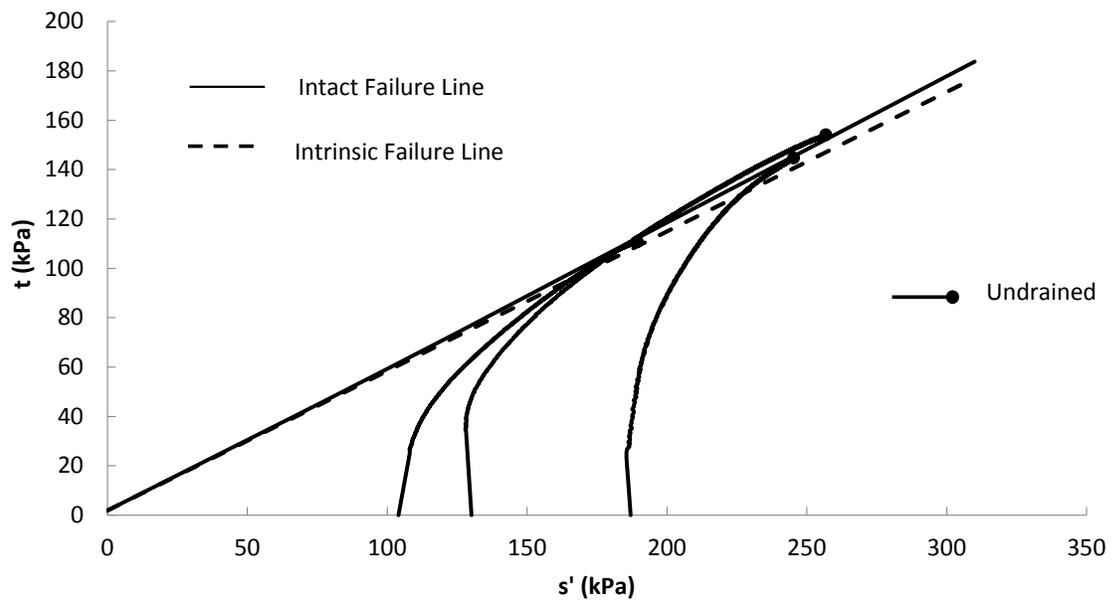
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(b)

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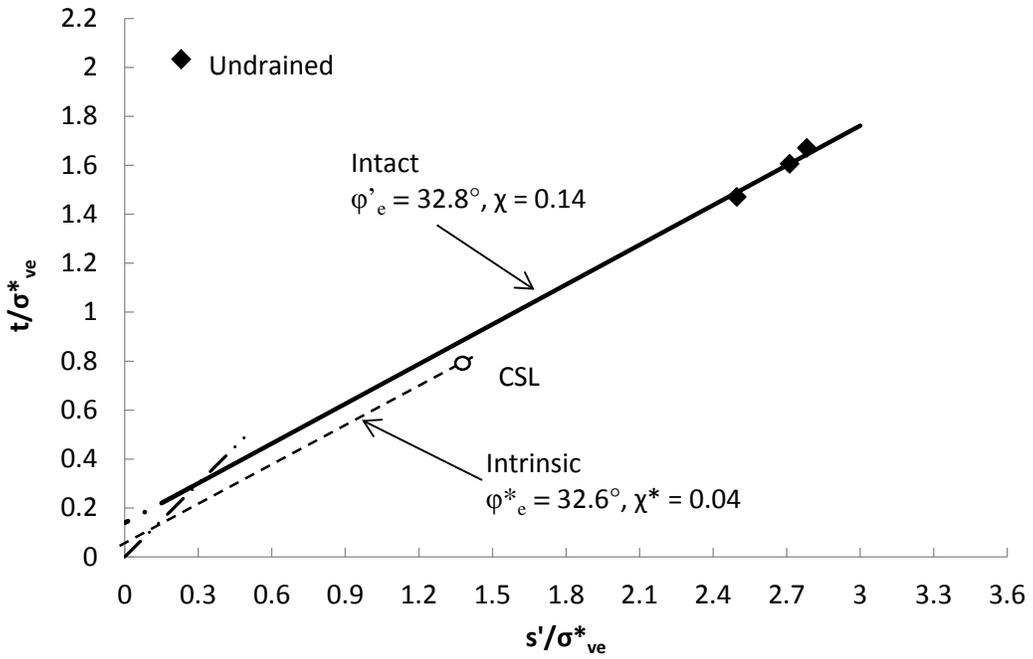
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**Figure 14: Peak strengths and undrained stress paths for the intact specimens.**

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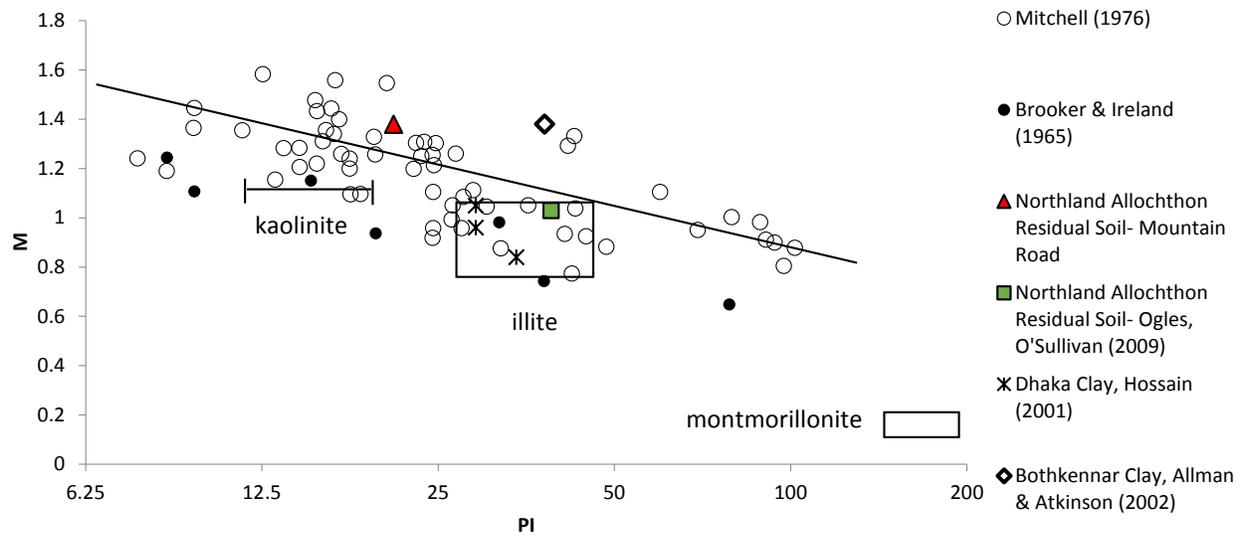


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