Performance of sensitive Taranaki volcanic silts under cyclic loading

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ABSTRACT

WSP has been involved in a number of building projects in New Plymouth. Some of the buildings are founded on volcanic silts (Taranaki ash). Field and simplified laboratory tests often indicate that these materials are sensitive in nature. Most of cohesive soils, even if they are non-liquefiable, can be prone to 10-30% strength loss due to cyclic softening under seismic load. However, strength loss in sensitive materials can be 4 to more than 50 times in some cases. There are no commonly accepted methods to predict strength loss for sensitive cohesive soils under cyclic loading. It was therefore critical to carry out a study of dynamic response of the sensitive soils for one of building sites in New Plymouth. Triaxial tests (cyclic and monotonic) of undisturbed soil samples were carried out to confirm dynamic response of the sensitive soils. The tests provided valuable information on the dynamic behaviour of the soils and enabled the designers to assess stability of the building platform and develop reliable structural foundation options with higher level of reliability.

1 INTRODUCTION

Ground or foundation failures associated with liquefaction of saturated loose sands have been regularly observed in large earthquakes. In spite of its complexity, the phenomenon of sand liquefaction is now well understood and empirical liquefaction assessment methods are available.

In recent years, substantial progress has been made in terms of the development of methods for the assessment of liquefaction and cyclic softening of cohesive soils. However, most of the research that formed the basis for such methods was carried out on reconstituted specimens and it is not clear whether the methods are applicable to in-situ sensitive fine-grained soils such as Taranaki silts of volcanic origin.

Sensitivity is normally attributed to the nature of individual soil particles and interparticle forces and is a result of the destruction of soil structure and cementation bonds between soil particles as well as thixotropy effects. Many researchers believe that the strength loss is caused primarily by the destruction of the silt / clay particle structure that was developed during the original process of sedimentation and also due to disturbance to water molecules in adsorbed layers. Specifically, with respect to volcanic ash materials, Jacquet (1990) noted that sensitivity is likely associated with the minerals allophane and halloysite that are commonly found in these materials, and highlighted the disturbed structures of these minerals in highly remoulded volcanic ash materials.

Sensitivity of the natural cohesive soils can also be related to the soil's liquidity index (LI). Soft normally consolidated or lightly overconsolidated cohesive soils will generally have higher natural water contents, higher LI values, higher sensitives, and will therefore be most prone to strength loss during earthquakes.

The risks of using commonly acceptable liquefaction and cyclic softening assessment methods for sensitive materials without additional studies are high. In the worst case, sensitive soils can experience dramatic strength loss under cyclic loading which may result in failure of slopes, building platforms and building foundations. On the other hand, extensive earthquake shaking may be required to generate shear strain levels that cause substantial reduction in strength of sensitive soils; also, it is likely that most of the strong motion will have subsided by the time the shear deformation required to drop the soil strength to its residual level occurs. Therefore, detailed studies of the seismic performance of sensitive materials are normally required for practical engineering projects.

Sensitivity is a measure of the soil strength loss. Sensitive soils, upon large shear strain or remoulding, experience substantial loss of their shear strength. The sensitivity S_t is the relation between the undisturbed undrained shear strength S_u and the residual strength after large shear deformation or a complete remoulding of the material (when no further strength reduction can occur), S_{ur}, and is defined by the ratio S_u/S_{ur}. In New Zealand, sensitive soils are classified into *Moderately Sensitive* (S_t of 2 to 4), *Sensitive* (S_t of 4 to 8), *Extra Sensitive* (S_t of 8 to 16) and *Quick* (S_t of more than 16).

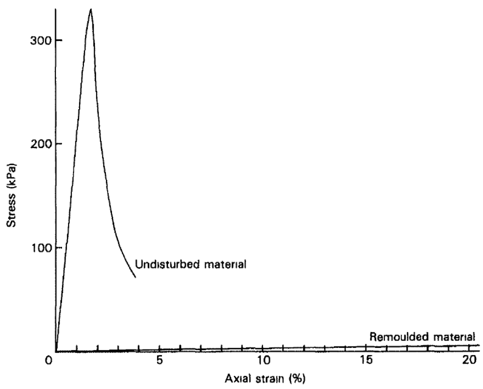

Figure 1: Typical stress-strain curves for highly sensitive volcanic ash soils (after Jacquet, 1990)

Typical stress-strain curves for undisturbed and remoulded extra sensitive soil are shown on Figure 1. The strain at failure can be quite low for undisturbed soils (1 % to 2%) and large for remoulded materials (6 % to 20%). As it can be seen from Figure 1, on remoulding, extra sensitive materials lose substantial part of their original strength. A significant strength loss of sensitive soils can be expected as a result of cyclic loading generated by earthquake shaking (Bisel et al, 2010). Previous studies of cyclic softening of sensitive cohesive materials were carried out by LeBoeuf et al (2016).

2 SITE MATERIALS

The natural ground profile at a building site in New Plymouth consists of Taranaki Ash materials overlying Lacustrine Deposits. The Taranaki ash can be classified as sandy silt, with fines content of up to 60% and sand particle size of 0.06 mm to 1 mm. The groundwater table was approximately 0.5 m to 1 m below the ground surface level. Previous publications in the literature such as Jacquet (1990) suggested that volcanic ash materials found in the New Plymouth areas could be sensitive, where the soil would exhibit a drastic loss in shear strength when subjected to very large deformation or remoulding. Sampling and Geonor Shear Vane tests were carried out in the four investigation boreholes (BH1 to BH4) drilled at the site. Our field Geonor Vane tests indicated that the residual strength of the Taranaki ash is quite low compared to its peak strength value. The measured range of S_t was 5 to 8, putting the site materials into the *Sensitive* to *Extra Sensitive* categories. Our laboratory tests on the site soils also showed that the site soils have high natural water content (equal to or greater than their liquid limit) and also high liquidity index values 0.6 to 1.1. The soil void ratios were in the range of 2.5 to 3.4. This indicates that the site soils are sensitive in nature and may experience poor seismic performance. The loss of strength due to sensitivity in such materials can potentially be very substantial. Therefore, the risk of slope instability and loss of bearing capacity of building foundations associated with the potential soil strength loss had to be addressed. To address this concern, three cyclic triaxial tests and one monotonic triaxial test were carried out.

3 TRIAXIAL TESTING PROGRAMME

The soil samples selected for the cyclic testing were derived from the push tube between depth 6.5 m and 7 m in BH3. From BH4, the depth between 5.5 m and 6.0 m were selected for monotonic triaxial test. Nominal specimen diameter and length were 60 mm and 120 mm respectively. The loading facility was a 250 kN MTS hydraulic loading frame. Figure 2 shows a typical triaxial test set-up used in University of Auckland. The soil specimen was back pressure saturated to achieve a Skempton B value of larger than 0.95. The initial effective consolidated pressure value was selected at 50 kPa.

Undrained cyclic loading was carried out at a loading frequency of 0.1 Hz. The first two cyclic tests were carried out under force-controlled conditions at a target value of CSR with stress-reversal, while the cyclic loading phase for the third test was carried out under displacement-controlled conditions. We did not adhere to a pre-determined schedule for these tests but instead developed the testing programme organically as new information came to light at the end of each preceding test. The details of each test are discussed in more detail below.

Figure 2: Cyclic triaxial test set-up

4 TEST RESULTS

4.1 Test 1

The method of Seed & Idriss (1971) was used to calculate the CSR value corresponding to a ULS PGA value of 0.33g applicable for the site. Since an earthquake in real-life has an irregular time-history and the PGA value is reached only momentarily throughout the shaking duration, Seed & Idriss (1982) proposed the use of a factor of 0.65 on the PGA value for use in a constant loading amplitude cyclic test. They also suggested that an earthquake magnitude of 6.5 (ULS magnitude for our building site) would be approximately equivalent to 10 cycles of loading in a constant-amplitude test (the magnitude is related to the duration of shaking as both are related to the size of fault rupture).

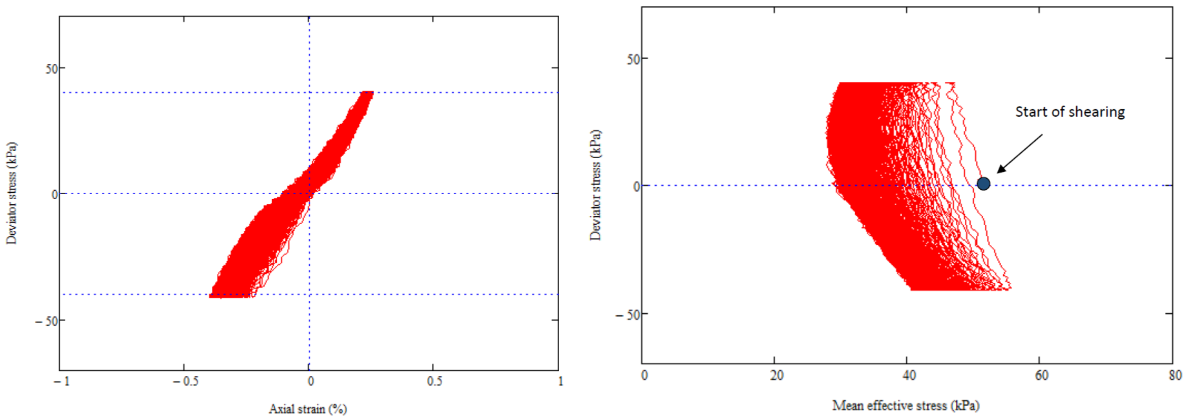

Figure 3: Test 1; deviator stress – axial strain loops (left); effective stress paths (right)

Using this methodology, the CSR thus calculated was 0.39 and this was chosen as the target CSR for Test 1. The specimen showed no sign of strength loss due to sensitivity nor stiffness degradation in Test Number 1 (Figure 3).

The stress-strain curve essentially retraced the same hysteresis loop throughout the entire test. The axial strain amplitude recorded was approximately 0.5%, and the test was terminated after 200 cycles.

4.2 Test 2

As the soil sample showed no sign of stiffness degradation in Test Number 1 under the target CSR as calculated based on Seed and Idriss (1971) method for the ULS PGA, it was decided to increase the target CSR value for Test Number 2 by 33% to 0.52 to investigate if adverse behaviour of the Taranaki ash material, such as strength loss due to soil sensitivity, might be triggered when the soil was subject to an elevated level of cyclic loading. The test results for Test 2 showed stiffness degradation as the number of loading cycles increased (Figures 4 and 5).

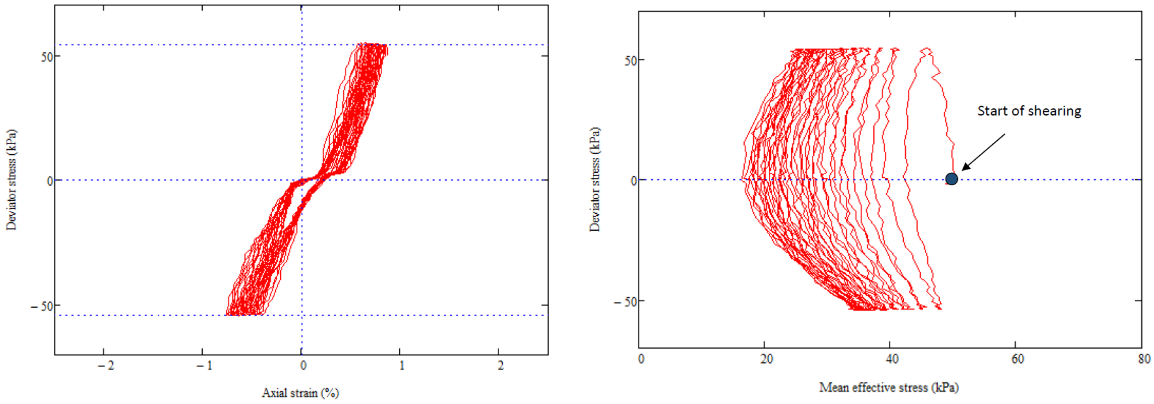

Figure 4: First 20 load cycles in Test 2; deviator stress – axial strain loops (left); effective stress paths (right)

The axial strain amplitude reached about 5% at about 50 cycles (Figure 5). However, the soil showed no sign of strength loss due to sensitivity and was able to maintain the same level of shear strength albeit requiring further deformation to mobilise the strength as the duration of loading increased throughout the test.

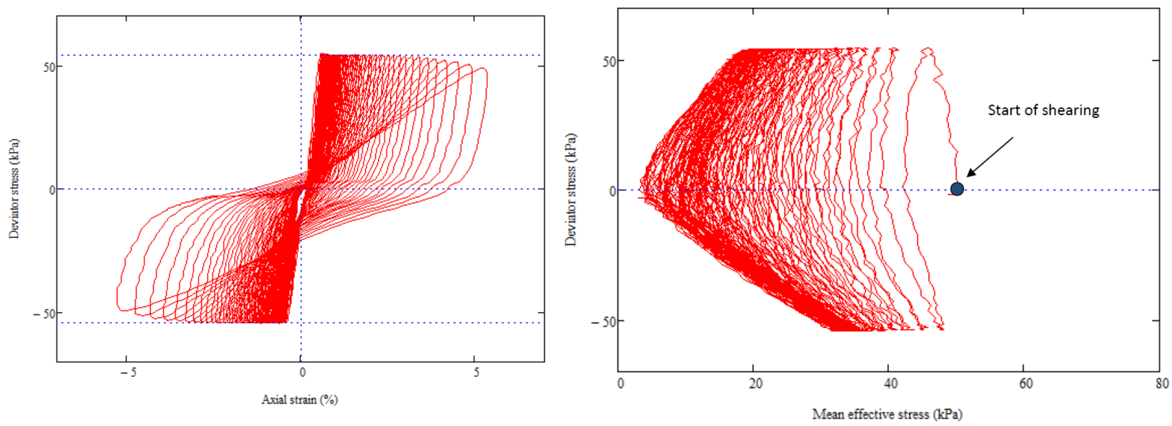

Figure 5: 65 load cycles in Test 2; deviator stress – axial strain loops (left); effective stress paths (right)

While some excess pore pressure build-up was recorded, no full liquefaction condition (r_u = 1) was achieved (Figure 6).

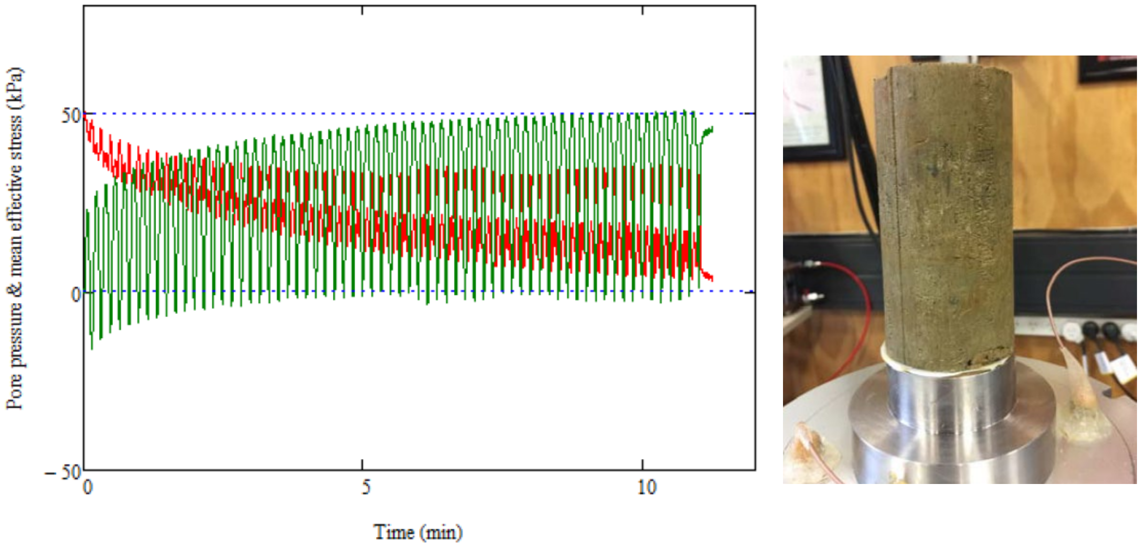

Figure 6: Pore water pressure response – green and mean effective stress - red (left); typical specimen set up in triaxial cell (right)

5 TEST 3

At our site, one of the structures was located on a fill embankment underlain by the saturated Taranaki ash, and slope stability of the sides of the fill embankment was one of the design considerations.

The methodology of Seed and Idriss (1971) was technically based on a flat ground surface. There is no simple way to convert a PGA value which, by definition, is for applications on level ground, to terrains such as a raised embankment. For design situation such as an embankment, an approximation can be found in Kramer (1996), who suggested that a pseudo-static force corresponding to 0.5PGA may be appropriate.

Given that for that structure, the Taranaki ash material is overlain by the fill embankment, the ash (sandy silt) would be subjected to a certain level of shear stress for resisting potential sliding of the embankment. There is a possibility that the initial level of shear stress could influence the material's cyclic behaviour relative to that observed when the material was initially consolidated at an isotropic state before cyclic loading commences.

To investigate the behaviour of the soil under such conditions, the sample in Test 3 was first loaded monotonically under undrained conditions to a deviator stress value of approximately 70 kPa (equivalent to a maximum shear stress of 35 kPa). The sample was then loaded under cyclic displacement-controlled conditions with a displacement amplitude corresponding to 0.5% axial strain approximately, the same value that had been recorded in Test 1. The purpose of this test was to investigate how the Taranaki ash sample would respond when cyclic loading corresponding to that of an ULS event commencing under an initial stress condition that a soil element may likely experience underneath the embankment.

Ideally, the cyclic loading phase of Test 3 would have been done at a target CSR value of 0.39. Unfortunately, since we could not be sure of the actual shear strength of the sample, the test could become uncontrolled if the sum of the cyclic shear stress and the initial static shear stress exceeded that of the shear strength of the sample. For this reason, it was decided that the cyclic phase of Test 3 would be carried out under displacement-controlled conditions. According to the test results, the sample showed a small degree of cyclic softening as the number of loading cycles increased (Figure 7).

If there were significant cyclic softening, each successive hysteresis loop would result in a flatter slope relative to the previous one. After about 5 loading cycles, the stress-strain curve began to retrace the same hysteresis loop.

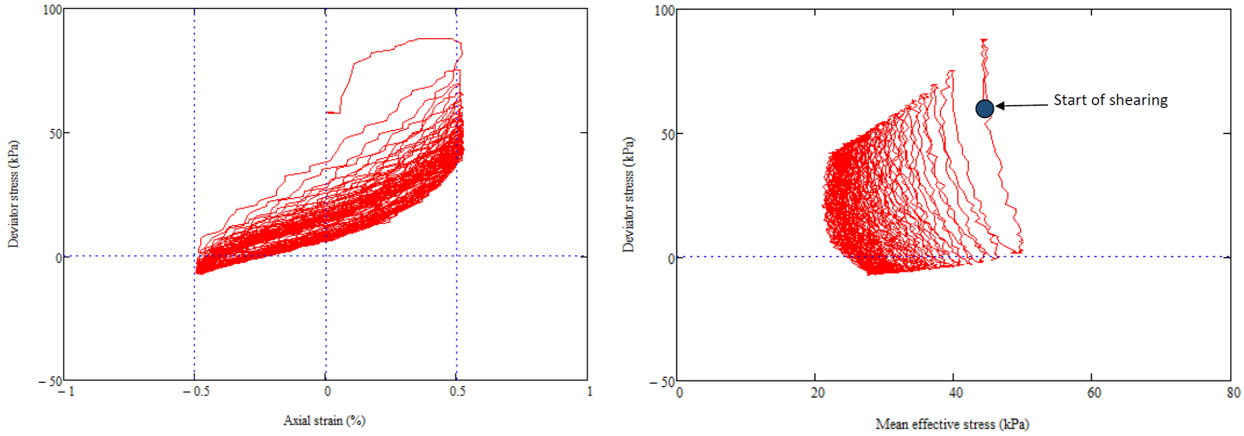

Figure 7: Test 3 – Phase 1; deviator stress – axial strain loops (left); effective stress paths (right)

We note that the sample reached a maximum deviator stress of approximately 85 kPa under the first cycle of loading before plateauing, indicating an undrained shear strength of about 42 kPa.

After about 45 loading cycles, the displacement amplitude was increased to an axial strain value of 1% and Test 3 entered cyclic loading phase 2 (Figure 8).

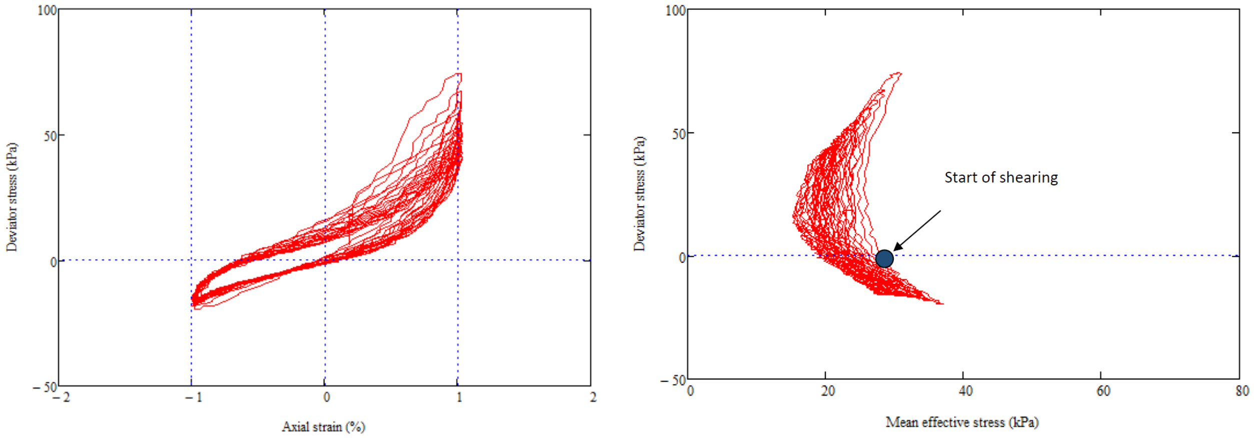

Figure 8: Test 3 – Phase 2; deviator stress – axial strain loops (left); effective stress paths (right)

At the first loading cycle of phase 2, the maximum deviator stress reached about 75% of the undrained shear strength shown in phase 1 while maintaining a concave-up stress-strain curve. This indicated that the sample would have likely reached its previous maximum shear strength if a higher value of axial strain was allowed to mobilise. There was also a larger degree of cyclic softening at the beginning of phase 2 although, similar to phase 1, the stress-strain curve eventually retraced the same hysteresis loop after about 5 loading cycles.

No sign of strength reduction associated with soil sensitivity has been observed at any stage of Test 3.

5.1 Test 4

Monotonic Consolidation Undrained Triaxial Test (CU) with pore pressure measurements was carried out. Regular data loggings of axial loads, pore pressure, axial displacement was measured for these tests, as well as recordings of initial specimen dimensions, as per the standard testing procedures of a regular CU test. The test was carried out to the axial strain value of 28%. This was to investigate the degree of soil strength decrease caused by the suspected sensitivity present in these samples under monotonic loading condition. The test data is shown on Figure 9.

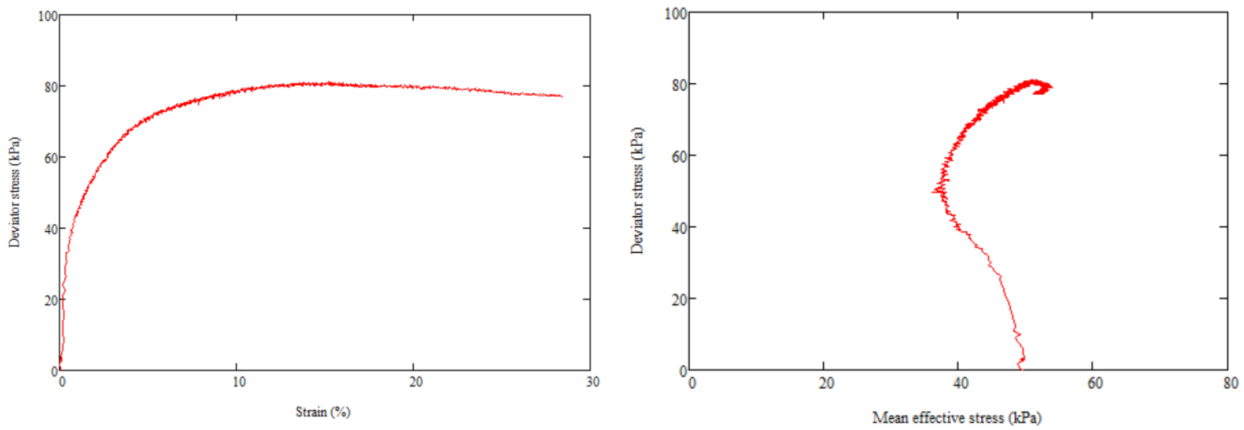

Figure 9: Test 4; stress-strain curve (left) and stress path (right)

No sign of substantial strength loss has been observed under monotonic loading condition in Test 4.

6 CONCLUSIONS

Simplified field and laboratory tests (Geonor Vane and Pilcon Vane tests) quite often indicate that the volcanic silts can be classified as sensitive and extra sensitive soils, indicating a potential for substantial strength loss under cyclic loading.

A study of seismic performance of a sensitive Taranaki ash (sandy silt) for a site in New Plymouth was carried to assess the effect of the soil's sensitivity on its strength loss under cyclic load. The study included field and laboratory testing. As part of the laboratory testing, three cyclic triaxial tests and one monotonic triaxial test were carried out.

The results of the cyclic triaxial tests showed that for the soil samples tested, no strength loss associated with soil sensitivity was observed under cyclic loading condition. Increased number of load cycles did not result in substantial strength decrease. The tests also indicated that the site soil is not prone to liquefaction.

The soil showed a slight degree of softening at cyclic load level corresponding to the ULS design event. However, it should be noted that such behaviour is commonly observed for typical non-sensitive cohesive soils.

The test results indicate that strength loss due to soil sensitivity and associated adverse effect on the soil bearing capacity and embankment slope stability at the site is not a risk.

The commonly accepted classification of soil sensitivity based on the peak and residual undrained shear strength measurements is not necessarily applicable to the assessment of the seismic behaviour of sensitive soils. Therefore, more detailed studies of sensitive soils for practical engineering projects in New Zealand, and especially in the Taranaki Region, can be justified.

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