

Geotechnical Properties of Two Volcanic Soils

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Abstract: Some of the factors which geotechnical engineers should be aware of in undertaking projects in volcanic country are described by giving an account of the behaviour and properties of two volcanic materials. The first is the allophane clay group; the unusual properties of allophane give these clays very distinctive characteristics. An account is given of their formation and geotechnical properties, and reference made to several projects undertaken involving allophane clays. The second is pumiceous sand, found in various parts of the North Island. The soft (or crushable) nature of the particles means the behaviour of these sands is significantly different from hard grained sands. An account is given of some research work carried out in an attempt to better understand their behaviour.

INTRODUCTION

Many engineering works of a geotechnical nature have been undertaken in the Central North Island volcanic region, the most impressive being the dams and canals constructed as part of major hydroelectric schemes. The Waikato River dams, the Tongariro scheme, and the Matahina dam are the best known examples. In general these projects have been very successful, though not all have been free of problems, either during construction, or after completion. The Arapuni scheme suffered some quite major construction problems, Matahina Dam experienced a significant “incident” during construction, and serious deficiencies in its design or construction became apparent after the Edgecombe earthquake. It has had major rebuilding and upgrading work done on it since then. Actual failures have not occurred on major schemes, but two spectacular failures occurred on small hydro schemes during the 1980s – the Ruahihi and Whaeo canal disasters.

SOME GENERAL COMMENTS

It is not the purpose of this paper to give an overview of the geotechnical properties of all the volcanic soils found in the volcanic areas of the North Island. That would be too big a task and beyond the competence of the writer. Rather the paper is restricted to two particular volcanic soil types, the study of which will hopefully highlight some of the general features of volcanic soils that geotechnical engineers should be aware of. These two types are allophane clays and pumice sands. Before proceeding with this some general observations may be useful.

Volcanic soils do not fit comfortably into the range of soils covered by most textbooks or conventional soil mechanics courses in universities. This is because their origin and formation processes are very different from those of “conventional” (ie sedimentary) soils. Fine grained materials of volcanic origin may contain clay minerals not found in other soils. In addition their formation process by situ weathering means that their properties are governed by this process, and not by stress history as with sedimentary soils. They cannot therefore be slotted into the familiar normally consolidated and over-consolidated categories. Coarse grained materials may be essentially the same as any other coarse materials, but they may also be very different. Some volcanic sands and gravels consist of quite weak particles, either because of their inherent nature or because weathering has altered and softened them.

It is important therefore to recognise these differences and not expect volcanic materials to conform to patterns of behaviour associated with conventional sedimentary deposits. In particular, empirical correlations applicable to sedimentary soils may be inappropriate and give misleading pictures when applied to volcanic soils.

ALLOPHANE CLAYS

Occurrence

There are substantial areas in the North Island where clays derived from the weathering of volcanic ash occur. These clays tend to be rich in the clay mineral allophane, which gives them rather unusual and unique properties. They are often referred to as "brown ash" by local engineers. Whether all clays referred to as brown ash contain allophane is not known to the writer; the term is used rather loosely and in some cases may be applied to clays that do not contain allophane. The clays described here are those whose properties are influenced primarily by their allophane content, and will be referred to as allophane clays. Similar clays occur in many parts of the world, including Indonesia, The Philippines, Japan, Central and South America, and Africa.

Formation

The formation and composition of allophane clay is complex, and most of the research and literature on the subject comes from the discipline of soil science rather than soil mechanics. This research and literature has grown enormously in the last two or three decades since the term allophane first found its way into geotechnical literature, and it shows a number of new and interesting findings. Firstly, it shows that allophane seldom occurs by itself. Instead, it is almost invariably found with other clay minerals, especially a mineral called imogolite. It seems to be almost inseparably linked to imogolite, and many papers on allophane are in fact on "allophane and imogolite" rather than on allophane alone. Secondly, it shows that allophane is not strictly amorphous, as early literature asserted. Both allophane and imogolite have some crystalline structure, albeit of a very different nature to other clay minerals.

Allophane clays are derived primarily from the in situ weathering of volcanic ash, although they may be derived from other volcanic material. This parent material may be either basic or acidic in nature. It appears that the primary condition for allophane formation is that the parent material be of non-crystalline (or poorly ordered structure) composition. Volcanic ash meets this criteria; it is formed by the rapid cooling of relatively fine-grained pyroclastic material, the cooling process being too rapid for the formation of well ordered crystalline structures. In the author's experience, the parent volcanic ash from which allophane clays are formed is generally in the coarse silt to fine sand particle size range.

In addition to the above requirement of non-crystalline parent material, it appears that the weathering environment must be well drained, with water seeping vertically downward through the ash deposit. High temperatures also appear to favour or accelerate the formation of allophane clays. Allophane clays may be very deep; in Indonesia the writer has encountered cuts in these materials up to about 30m deep, while site investigation drilling has shown depths of up to almost 40 metres. This thickness results from successive eruptions and associated ash showers, with weathering progressing as the thickness grows. Examination of cut exposures in West Java, Indonesia, shows the individual layer thickness to vary generally between about 100 and 300mm.

Structure

The precise structure of allophane clays is somewhat problematic. Their extraordinarily high natural water contents and void ratios (described in the next section) clearly indicate an unusual material, and call for an explanation in terms of either structure or chemical composition (or both). Various explanations have been offered over the years. As mentioned above, allophane has been described in the past as non-crystalline or amorphous, and "gell-like". However, electron microscopy studies over the past 10 years or so (Wada, 1989 and Jacquet, 1990) show that the material in its natural state does have an ordered structure – consisting of aggregations of spherical allophane particles with imogolite threads "weaving" among them, or forming "bridges" between them.

Figure 1 shows an electron micrograph of the material in its undisturbed state, (taken from Wada, 1989). The concept of approximately spherical particles with thread-like structures spanning between them appears to explain both the very high natural water content, and the changes the material undergoes on remoulding. Remoulding appears to break up the aggregations of particles and threads

spanning between them and turns the material into a homogeneous unstructured mass. This is generally accompanied by some loss of strength and an increase in compressibility, as well as a reduction in permeability.



Figure 1. Electron Micrograph of Allophane and Imogolite (after Wada, 1989).

General Comments on Engineering Properties

Before describing particular properties the point should be made that allophane clays are not problem soils. There is still a belief among some geotechnical engineers that the presence of allophane in a soil is something to fear or be concerned about. This should not be the case. Observation of these clays in their natural environment shows them to perform remarkably well. For example, terraced ricefields in Indonesia exist on slopes as steep as 35° and almost up to 40° . They are permanently saturated by irrigation water flowing from terrace to terrace. Many water retaining structures have been successfully constructed from allophane clays. While they ought not to be a cause for concern, it is important that their special properties be understood and taken account of in planning engineering projects involving allophane clays.

Natural Water Content, Void Ratio, and Atterberg Limits

The natural water content of allophane clay covers a very wide range, from about 50% to 300%. This corresponds to void ratios from about 1.5 to 8. It appears that water content is a reasonable indication of allophane content – the higher the water content the greater the allophane content. Atterberg Limits similarly cover a wide range, and when plotted on the conventional Plasticity Chart invariably lie well below the A-line. This means that according to the Unified Soil Classification System they are silts. However they do not display the characteristics normally associated with silt – the tendency to become “quick” when vibrated and to dilate when deformed. At the same time they are not highly plastic like true clays, so they do not fit comfortably into conventional classification systems. Figure 2 shows a plot of the Atterberg limits on the Plasticity Chart.

Influence of Drying

Drying has a very important effect on allophane clays. Frost (1967) gave the first systematic account of this effect for both air and oven drying on tropical soils belonging to the allophane and halloysite group. He showed that clays from the mountainous districts of Papua New Guinea with values of Plasticity Index ranging from about 30 to 80 in their natural state become non-plastic when air or oven

dried. Wesley (1973) describes similar effects from the allophane clays of Java, Indonesia. The properties of the clay described in this paper apply to the clay in its natural state, ie without air or oven drying, unless otherwise stated.

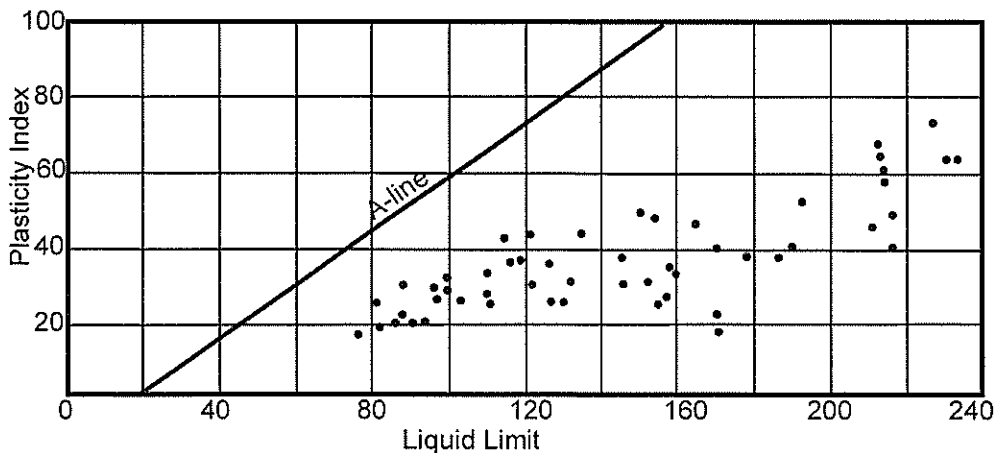


Figure 2. Atterberg Limits of Allophane Clays on the Plasticity Chart.

Identification of Allophane Clays

There are various techniques used by soil scientists to identify allophane: these are primarily X-ray diffraction and electron microscopy. Such methods are not readily available to geotechnical engineers. For engineering purposes, sufficient indicators of the presence of allophane are the following:

- 1) Volcanic parent material
- 2) Very high water contents
- 3) Very high liquid and plastic limits lying well below the A-line on the Plasticity Chart
- 4) Irreversible changes on air or oven drying - from a plastic to a non-plastic material.

If all of these are applicable then the soil almost certainly contains a significant proportion of allophane.

Stiffness and Compressibility

Typical results from oedometer tests on undisturbed samples from Indonesia and New Zealand are shown in Figures 3 and 4. Details of the samples are given in Table 1.

Sample Number	Location Information	Depth (m)	Void Ratio	Water Content (%)	Atterberg Limits		
					L.L.	P.L.	P.I.
I.3	Cipanunjang, Indonesia	1.0	4.76	161.2	187	149	38
I.4	Cipanunjang, Indonesia	3.0	4.72	156.1	179	139	40
I.6	Kamojang, Indonesia	12.2	5.34	190.7	*		
NZ.4	Omata, New Zealand	7.5	2.58	81.3	*		
NZ.6	Omata, New Zealand	7.0	3.20	114.1	*		
NZ.7	Omata, New Zealand	5.1	2.49	88.5	*		

* Atterberg limits not measured on these samples

Table 1. Details of Samples Used for Oedometer Tests.

Figure 3 shows the results as conventional e - $\log(p)$ graphs and Figure 4 as direct compression versus stress on a linear scale. The e - $\log(p)$ curves suggest that all the samples have similar compressibility characteristics with "pre-consolidation" pressures of varying magnitude. However, when plotted using a linear pressure scale this is no longer the case. Only some of the samples show an apparent pre-consolidation pressure; this is of course not related to stress history. It arises from the structure of the

soil created by the weathering process, and is perhaps better described as a vertical yield pressure rather than a pre-consolidation pressure. Why some samples show a yield pressure and others do not is not known, though it may be related to the original denseness of the parent material.

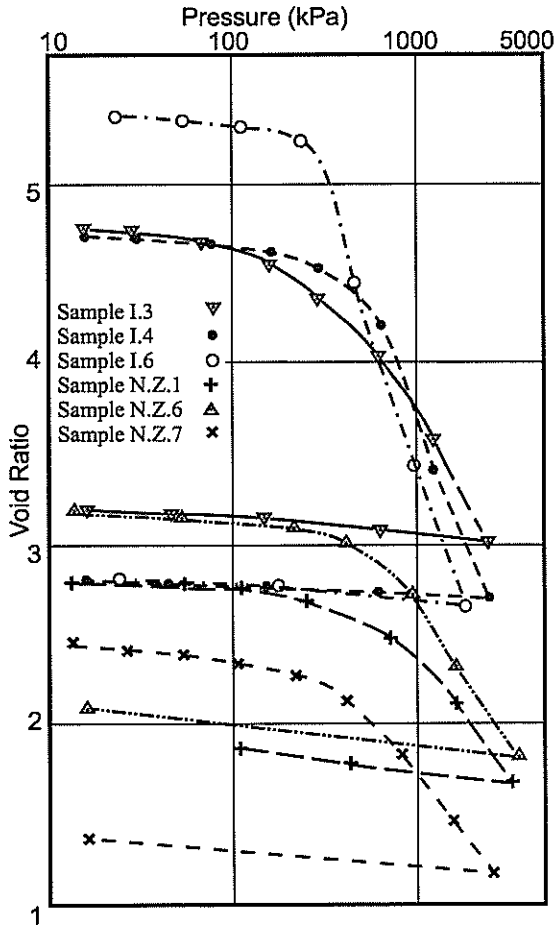


Figure 3. Oedometer Test Results as e-log(p) Plots.

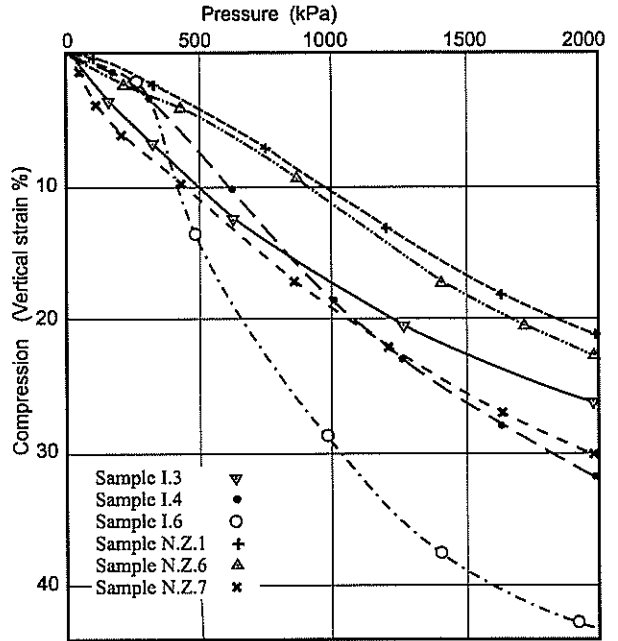


Figure 4. Oedometer Tests Showing Compression Versus Pressure on a Linear Scale.

These graphs illustrate two important points. Firstly, to gain a clear picture of the consolidation behaviour it is necessary to plot the results using a linear scale as well as a log scale. Secondly, the portion of the graph of interest in foundation design is often close to linear with respect to pressure, and favours the use of the linear parameter m_v (or constrained modulus D) for settlement calculations rather than the log parameters C_c and C_s .

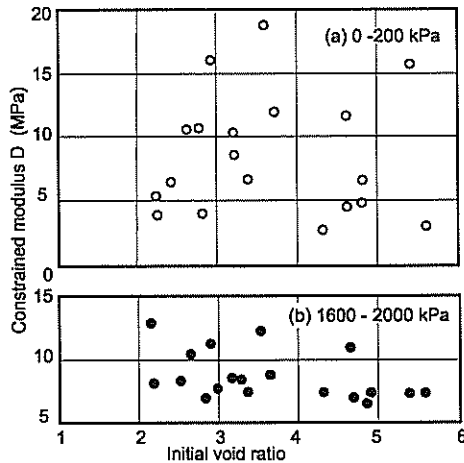


Figure 5. Constrained Modulus (D) Versus Initial Void Ratio.

It is of interest to note that for these clays there does not appear to be any relationship between the initial void ratio and compressibility. Fig. 5 shows the constrained modulus D measured when the sample is loaded from 0 to 200 kPa, and again between 1600 kPa and 2000 kPa, plotted against the initial void ratio. The data shows considerable scatter, but there is no clear trend towards higher compressibility with increase in void ratio from 2 to nearly 6.

Figure 6 shows typical root time plots from oedometer tests. At low stress increments the consolidation rate is clearly very rapid but becomes progressively slower as the stress level rises. To investigate this effect in more detail, pore pressure dissipation tests were carried out using a triaxial cell. Two samples from New Zealand and two from Indonesia were tested. A summary of the c_v values obtained from these dissipation tests is shown in Fig. 7. It is seen that the c_v value decreases by approximately four orders of magnitude as the stress increases from 50 to 1000 kPa. With the New Zealand samples, the tests were repeated after remoulding the soil. It is seen that the c_v value is then consistently low and close to the end value from the undisturbed samples. With the Indonesian samples, permeability measurements were also made between each consolidation stage; the results showed an identical trend to the c_v values. Figure 7 shows that remoulding the soil apparently destroys the open structure of the undisturbed soil, which is believed to account for the high permeability.

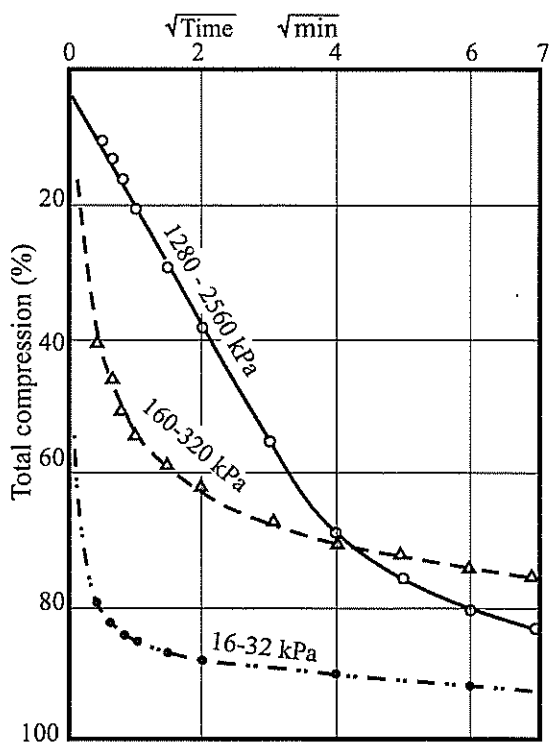


Figure 6. Typical Root Time Plots from Oedometer Tests.

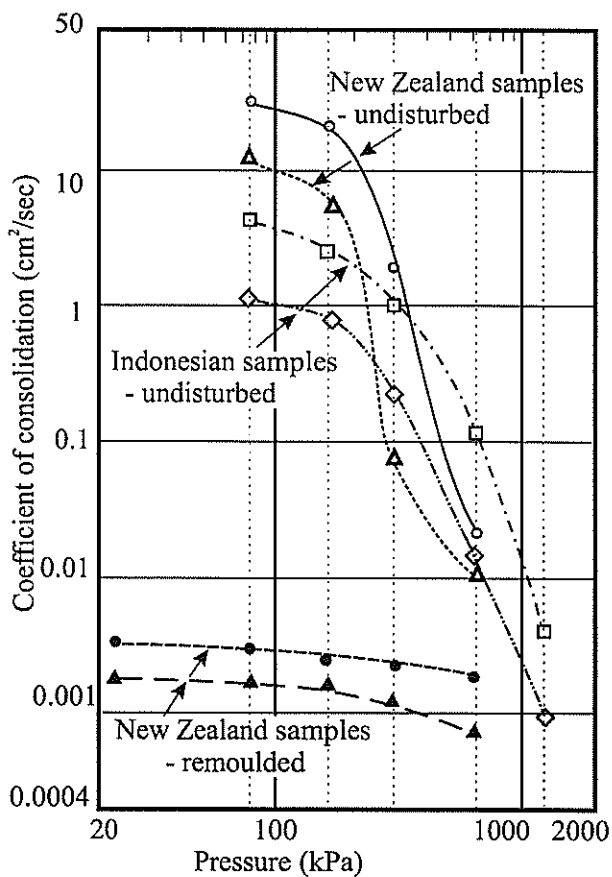


Figure 7. Summary of c_v Values from Pore Pressure Dissipation Tests.

It should be appreciated that with clays of this type it is not possible to determine reliable c_v values from conventional oedometer tests. The drainage path length is too short for pore pressure dissipation to control the deformation rate. The upper limit of the c_v value which can be measured with a conventional oedometer test with sample thickness of 20 mm is about $0.01 \text{ cm}^2/\text{sec}$. At the relatively low stress levels relevant to engineering situations, the c_v value of allophane clays is normally much higher than this.

The points made above should be kept in mind when dealing with any residual soil. Results of consolidation tests should be plotted using both log and linear scales in order to get a true picture of behaviour. Some samples within a particular soil type may show distinct “yield” pressures, while others may not. Consolidation rates can also vary over wide ranges within any given soil group.

Some layers within the Auckland Waitemata clays have permeability values at the lower end of the range expected for clays, while others may have permeability values normally associated with silts.

Undrained Strength

Figure 8 shows cone penetrometer test (CPT) results from two sites, one in Indonesia and one in New Zealand.

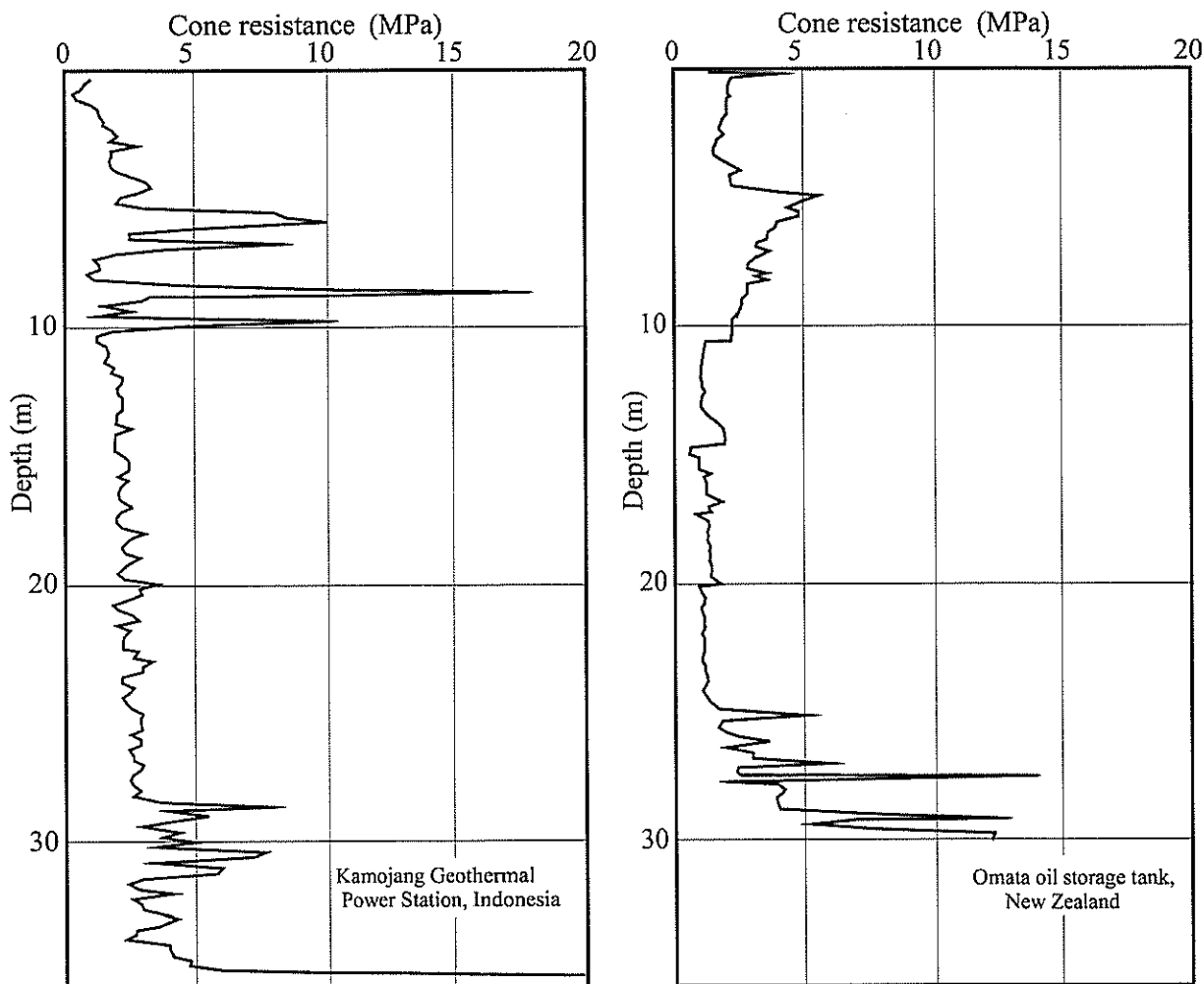


Figure 8. Cone penetrometer Tests from Sites in Indonesia and New Zealand.

These are fairly similar. They show that while the in situ strength is reasonably uniform, it does have small fluctuations over the full profile, and there are some zones with considerably higher values. These are believed to be zones of coarser material within the fine clay. The cone resistance varies between about 1 and 3 MPa. Using a correlation factor (N_k) of 15 this corresponds to an undrained shear strength range of about 65 kPa to 200 kPa. Values of undrained strength obtained from other methods at the Kamojang site ranged from about 50 kPa to 170 kPa, confirming the trend indicated by the CPT tests.

Effective Strength Parameters

The effective strength parameters c' and ϕ' are surprisingly high for a soil of such fine grained composition. This is perhaps not surprising; observation of field behaviour suggests that this must be the case. As mentioned earlier, in Indonesia and other tropical countries, terraced rice fields exist on remarkably steep slopes in areas of allophane clay. These slopes remain stable despite permanent saturation with irrigation water, which flows from terrace to terrace.

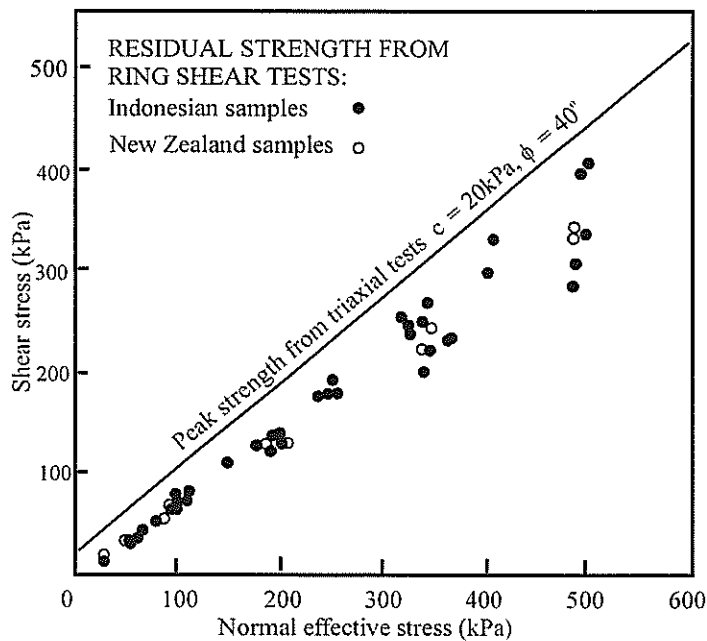


Figure 9. Effective Strength Parameters.

Figure 9 summarises results from laboratory tests on samples from Indonesia and New Zealand. Triaxial tests were carried out to obtain the peak values, and ring shear tests to obtain the residual values. Both values are remarkably high and there is surprisingly little difference between them. Rouse et al (1986) have obtained similar high values from allophane soils in Dominica.

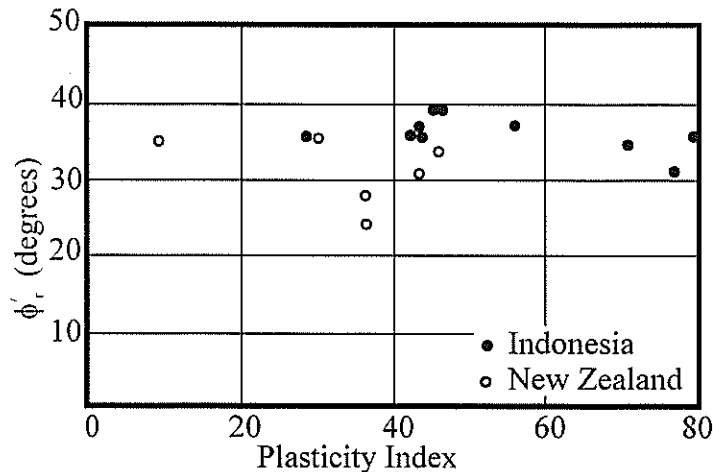


Figure 10. Residual Strength Friction Angle Versus Plasticity Index.

Fig. 10 shows values of the residual angle ϕ'_r plotted against Plasticity Index. It is seen that there is no relationship between the two; ϕ'_r does not steadily decrease with Plasticity Index as is the case with sedimentary clays. With PI values above about 80, sedimentary soils would be expected to have ϕ'_r values of around 10° , whereas the allophane clay has values between 30° and 40° .

Compaction Characteristics

The compaction behaviour of typical allophane clay is illustrated in Fig. 11. This shows a "natural" compaction curve together with curves obtained after drying to varying degrees. The natural water content was 166%. The natural curve was obtained by drying back the soil in steps from this initial water content. Fresh soil was used for each point. The test was then repeated three times, firstly after oven drying, secondly after air drying, and finally after limited air drying (to $w = 65\%$). This was done with large bulk samples. The material was then wetted up in stages, using fresh soil for each point. The results show the dramatic changes caused by drying. When dried from its natural water content

the compaction curve is almost flat, with only a very poorly defined optimum water content. On re-wetting, the behaviour becomes more conventional, with clearly defined optimum water contents and peak dry densities. It is evident from this that almost any result can be obtained if the test involves drying and re-wetting. This result is from an Indonesian allophane clay. Local (ie New Zealand) allophane clays may not show such a dramatic effect because of their lower allophane content.

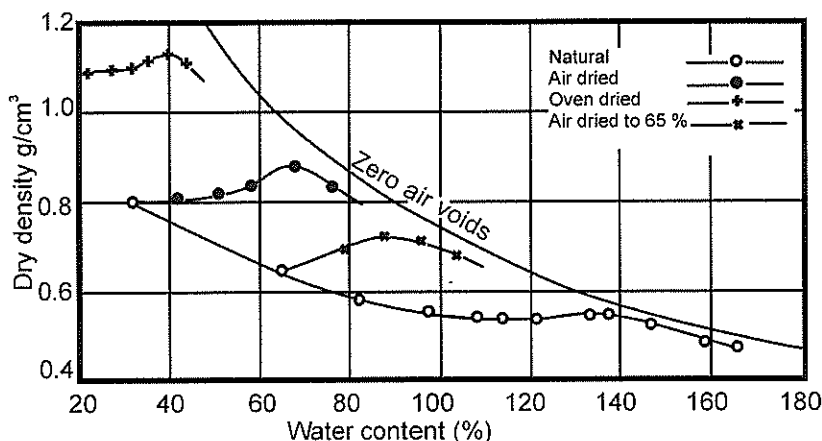


Figure 11. Typical Compaction Behaviour of an Allophane Clay.

Fig. 12 (after Kuno et al 1978) shows the effect of repeated compaction on allophane soils. Some allophane clays are of high sensitivity, and others are not: this is reflected in the curves in Fig. 12. The strength of the soil has been measured after compaction using a series of different (but known) compactive efforts. The compactive effort is indicated by the number of hammer blows. A cone has been pushed into the soil to obtain a measure of strength: this is the "cone index" value shown in the figure. The graphs show that in general there is a marked decrease in strength as the number of blows increases. Presumably the structure of the soil is being progressively destroyed, releasing water and softening the soil, an effect sometimes referred to as "over-compaction".

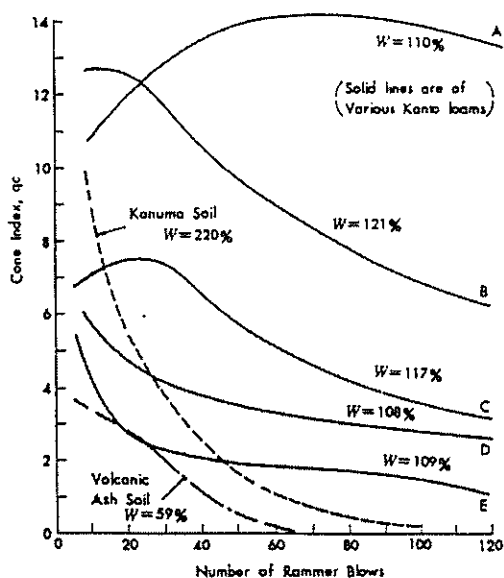


Figure 12. Influence of Compactive Effort on Strength of Compacted Allophane Clays.

The above behaviour illustrates that difficulties can arise in compacting allophane soils if their properties are not understood and taken account of in planning and executing earthworks operations. Specifications can be almost meaningless if excessive drying is allowed before testing is carried out. In countries like Papua-New Guinea and Indonesia the wet climate in which allophane clays occur means that significant drying during excavation and compaction is not very practical. Difficulties

during earthworks operations are described by Parton & Olsen (1980) and Moore & Styles (1988).

These problems can be overcome to some extent in several ways. The first is to recognise that soils can be satisfactorily compacted without recourse to the rigid control methods associated with water content and dry density values. The second is to be clear what objective is aimed for in compacting the soil. For example, the objective with a road embankment is very different from that with a water retaining embankment. With a road embankment it is preferable to keep the compactive effort to a minimum and “press” the soil together with quite light compaction. – enough to get rid of any large voids, but insufficient to destroy the natural “structure” of the soil and cause it to soften. In this way it is possible to retain much of the original strength of the material. With water retaining embankments a rather more rigorous approach is needed, but even for these it is desirable to carefully control the compactive effort. Compaction control, involving control of compactive effort, together with shear strength and air voids testing is generally a better approach than conventional water content and dry density methods.

The Cipanunjang dam in West Java (Wesley, 1974) is an example of successful compaction of allophane clay; compaction here was done using steel rimmed rollers. Some difficulties were encountered due to wet weather and softening of the soil, but the job was completed satisfactorily. The writer has been involved in the compaction of allophane clay at a geothermal site (Kamojang) in West Java, Indonesia. Difficulties were encountered because the very wet climate at the site made it difficult to dry the soil sufficiently to achieve the target undrained shear strength of 150 kPa. The fill was required to form a level platform for a switch yard. The strength requirement was lowered to 90 kPa and the job successfully completed. The fill appeared to “harden” with time, presumably due to the development of negative pore pressure in the soil.

Erosion Resistance.

It is an interesting observation that both in their undisturbed and re-compacted state, allophane clays are remarkably resistant to erosion. It is only when they are cultivated and allowed to partially dry at the surface that they become susceptible to erosion. Observation of road cuttings in Southeast Asia as well as in New Zealand (Taranaki and the central volcanic plateau) shows that negligible erosion occurs from the cut faces. In Indonesia, the drying of the face appears to result in the formation of a hard “crust” which is resistant to erosion. It is also evident in terraced rice-fields that negligible erosion takes place as the irrigation water flows from one terrace to the next terrace.

The report into the Ruahihi canal failure makes some rather sweeping statements about the properties of volcanic soils. Among these are allegations of dispersiveness and erodibility, and susceptibility to “degradation”. The statements are very sweeping and no attempt is made to differentiate between different types of volcanic soil. The report prompted the writer to investigate this question of the dispersivity of allophane clays. A final year student, Chan Siew Yan carried out pin-hole dispersion tests on soils from the Ruahihi area, as well as tests on other allophane clays. The results are described by Wesley and Chan (1991). None of these tests showed any evidence of erosion or dispersivity. For a more comprehensive account of allophane clays see Wesley (2002).

Significant Engineering Projects in Allophane Clays.

A number of dams and related water retaining structures have been successfully undertaken making use of allophane clays. An early example is the water supply dam Cipanunjang in West Java, Indonesia, built in 1928 during the Dutch colonial period. This is a homogeneous 30m high embankment with cut-off drains in the downstream slope. It is described in detail elsewhere (Wesley, 1974), and is still a vital part of Bandung’s municipal water supply. The Mangamahoe Dam in New Plymouth, and the embankment supporting the supply canal at the Kuratau power scheme (on the western shore of Lake Taupo) are further examples. The Kamojang geothermal power station in West Java, Indonesia, is supported by a raft foundation on about 35m of allophane clay (Figure 9). There have been no problems with its performance. Wesley and Matuschka (1988) describe these examples in greater detail.

PUMICEOUS SANDS

Occurrence and General Description

Pumice sands are found in various parts of the North Island of New Zealand, especially along the lower Waikato River valley and in parts of the Bay of Plenty. Although they do not cover wide areas, their places of concentration in river valleys and flood plains means they tend to coincide with areas of considerable human activity and development. Consequently they are not infrequently encountered in engineering projects and their evaluation is a matter of considerable interest to geotechnical engineers. Not much research has been carried out into the properties of these materials; in recent years the geotechnical group at Auckland University has made some progress towards filling the gap. Some of the results to date have been described by Wesley et al (1999), and Wesley (2000). In this paper the main focus will be on the issue of particle crushing, and the extent to which this influences the behaviour of the sand.

The sand is characterised by the vesicular nature of its particles; each particle contains a dense network of fine holes, some of which are inter-connected and open to the surface, while others appear to be entirely isolated inside the particles. The result is that the particles are light-weight, have very rough surfaces, and are easily crushed, especially when compared to more “normal” hard grained sands such as quartz sand.

One Dimensional Compression and Particle Crushing

To initiate this investigation of particle crushing the results of one dimensional compression (oedometer) tests are illustrated in Figures 13 and 14. Figure 13 shows the particle size curves for the four samples used in these tests. One of the samples is a natural sand obtained from the Waikato River. The other three are artificial sands obtained by crushing coarse pumice particles and then sieving. Figure 14 shows the results of the compression tests, which were carried out to a maximum stress of 8000 kPa. The reason for this high stress level will become apparent shortly – it is of a similar magnitude to the stress level in cone penetrometer tests. A curve from a typical hard grained (predominantly quartz) sand is also shown in this figure. It is immediately apparent that the pumice sands are much more compressible than the hard grained sand. The difference in compressibility is greatest at the lower stress levels, especially between zero and 2000 kPa. In order to investigate the effect of crushing, the tests were actually repeated a number of times to steadily increasing stress levels, and terminated at these levels so that particle size measurements could be made corresponding to each stress level. The stress levels involved were 500, 1500, 3000, 5000, and 8000 kPa.

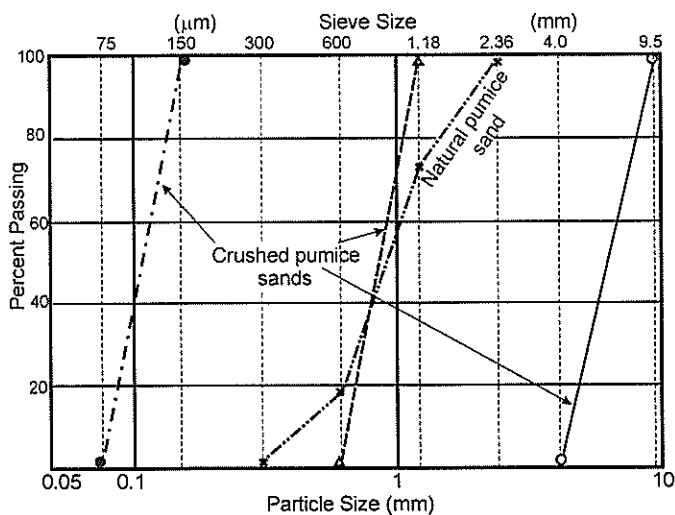


Figure 13. Particle Size Distribution Curves of the Pumice Sands.

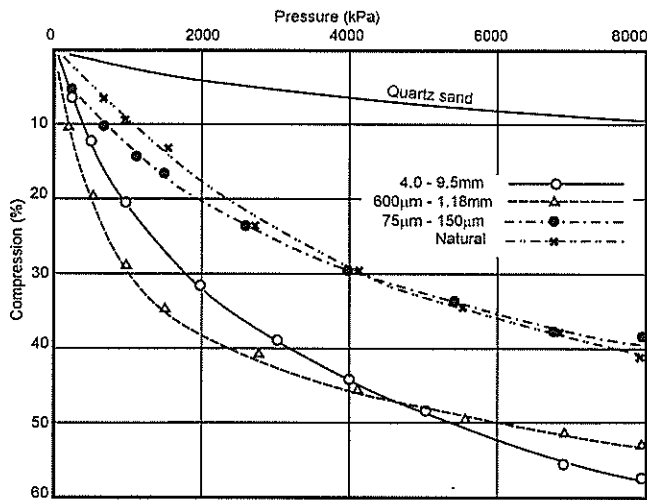


Figure 14. One Dimensional Compression Test Results.

The particle size measurements showing crushing are presented in Figure 15. It is immediately evident that very substantial particle crushing occurs, especially with the two coarser fractions. Crushing in the natural sand and the very fine fraction is much less than in the other fractions.

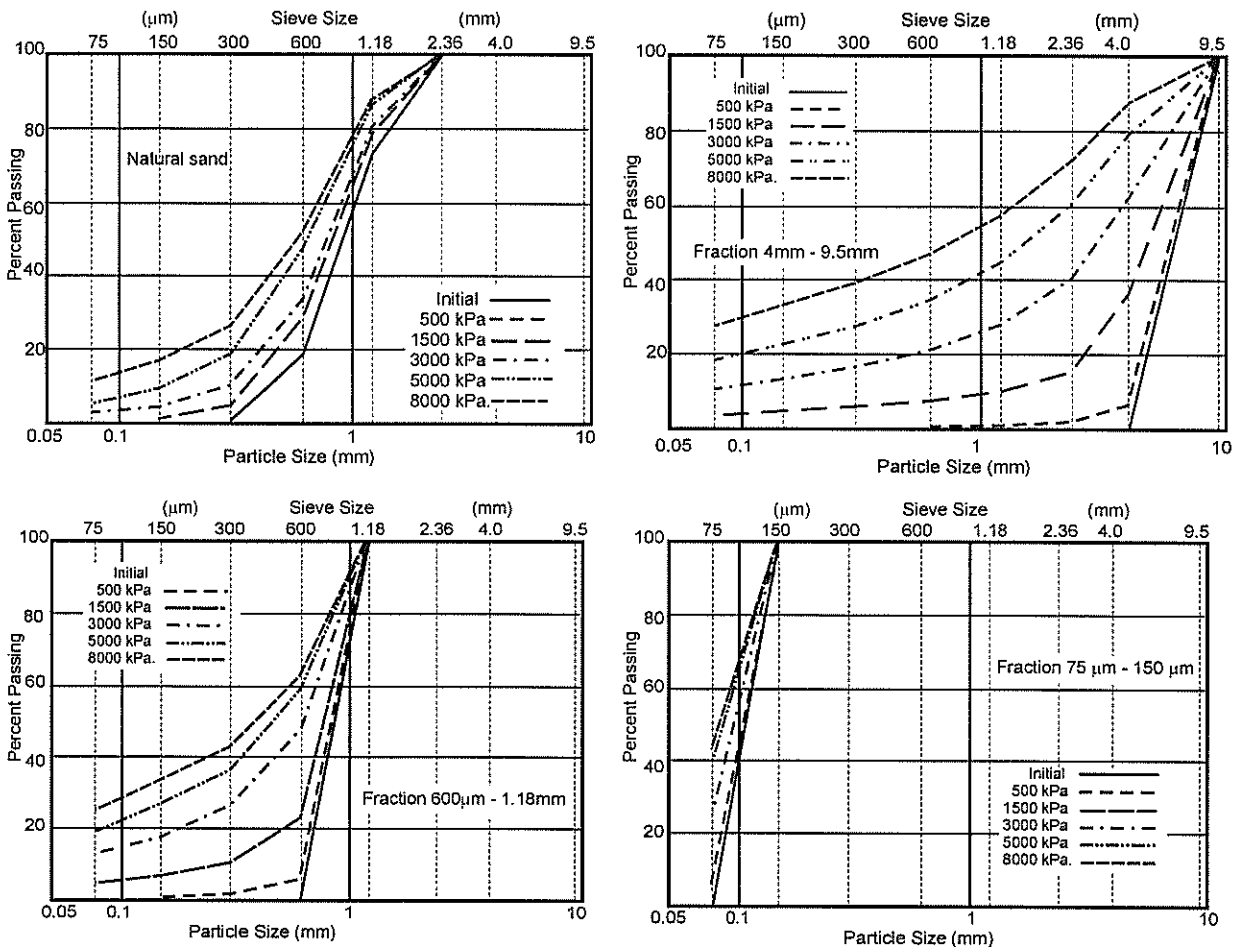


Figure 15. Particle Crushing During One Dimensional Compression.

To further illustrate the crushing effect, Figure 16 has been prepared. This shows a graph of percent crushed against the stress level for each of the four materials. The figure confirms the point mentioned above, namely that crushing is least in the natural sand and the fine fraction, and also suggests that crushing tends to “level off” at higher stress levels. The measure used here to express the degree of crushing (percent crushed) is a fairly elementary one, and more sophisticated measures may show

somewhat different trends. The measure used here has been calculated by simply comparing the amounts retained on each sieve; where reductions have occurred these have been calculated and added up to give the total weight of material crushed. This has then been expressed as a percentage of the total weight.

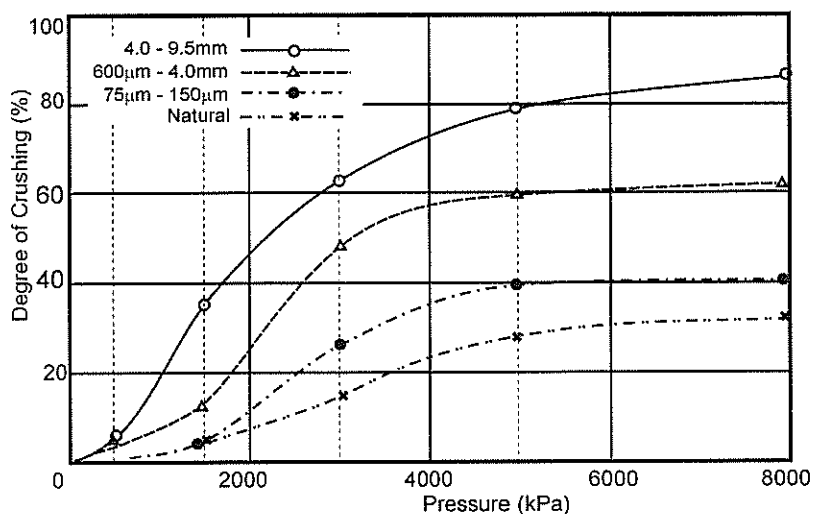


Figure 16. Degree of Crushing in Compression Tests.

Figure 16 suggest that very little crushing occurs at the stress levels involved in most foundation situations. Applied stresses from foundations on sand are unlikely to exceed 1000 kPa. The onset of major crushing appears to be at about 1500 kPa, and then levels off again after about 4000 kPa. At this latter stress level the particle size distribution has changed so much that the materials have become well graded and densely packed, so that contact stresses between particles are greatly reduced (and crushing declines). Despite the impression given by this figure it should not be imagined that no particle breakage occurs at low stress levels. Some crushing probably starts as soon as load is applied: the measured degree of crushing at 500 kPa was between about 2% and 5%. The large difference in compressibility between the pumice sand and conventional hard grained sand can only be attributed to the soft nature of the pumice particles and the consequent crushing, so that even if major breakage does not occur at low stress levels, some minor fracturing is still occurring, probably at the surface of the particles.

The information presented in Figs 15 and 16 is from tests on the sand in its loose state. Tests were also carried out on dense samples; these showed comparable results with somewhat less crushing.

Cone Penetration Tests in a Calibration Chamber

The prime interest in carrying out this research on pumice sands was in the interpretation of conventional cone penetrometer tests (CPTs). For this purpose a large calibration chamber (on loan from Monash University) was used. A diagrammatic view of the chamber is shown in Figure 17. Tests were carried out on the natural pumice sand, and also on a hard grained predominantly quartz sand. The tests were on the sands in both their dense and loose states, using applied vertical stresses of 50, 100, and 200 kPa. The results of these tests are summarised in Figures 18 to 21

Figs. 18 and 19 show typical results for the pumice and quartz sand respectively; these are for the loose and dense states at a vertical confining stress of 200 kPa. In Fig. 20, the cone resistance curves from Figs. 18 and 19 are shown on the same graph so that a direct comparison can be made between the behaviour of the two sands. Fig. 21 summarises all the test results in the form of the familiar graph of q_c versus vertical stress.

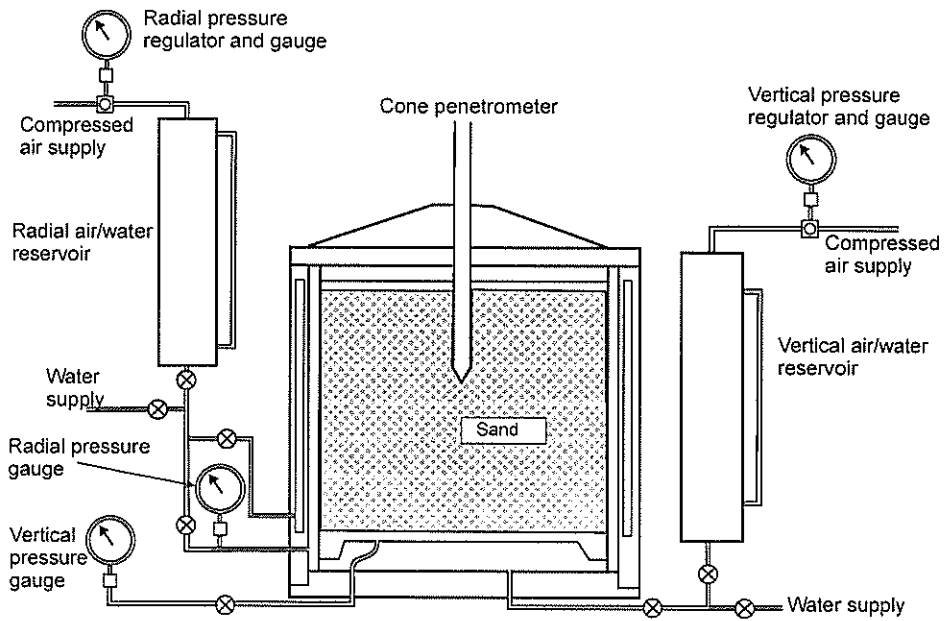


Figure 17. The Calibration Chamber Used for Cone Penetrometer Tests.

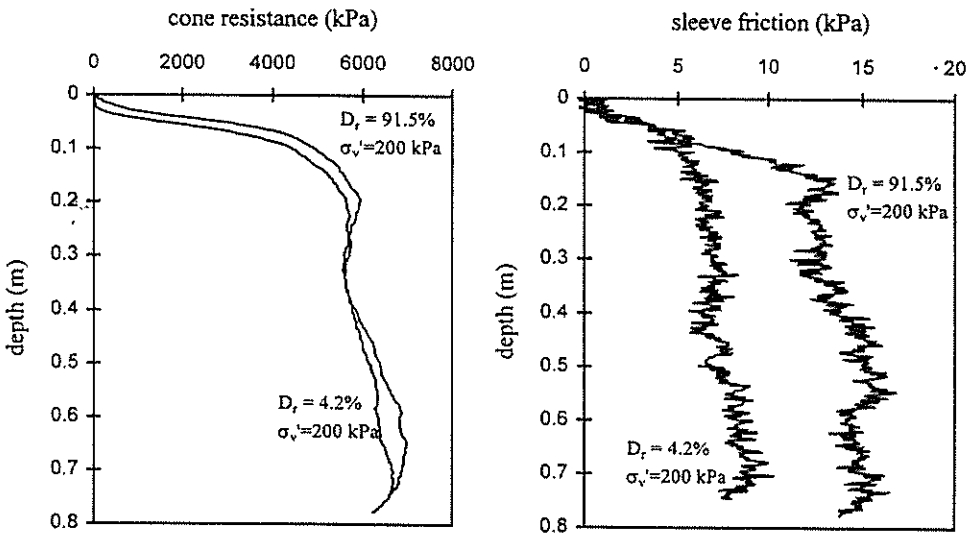


Figure 18. Typical Results from the Pumice Sand.

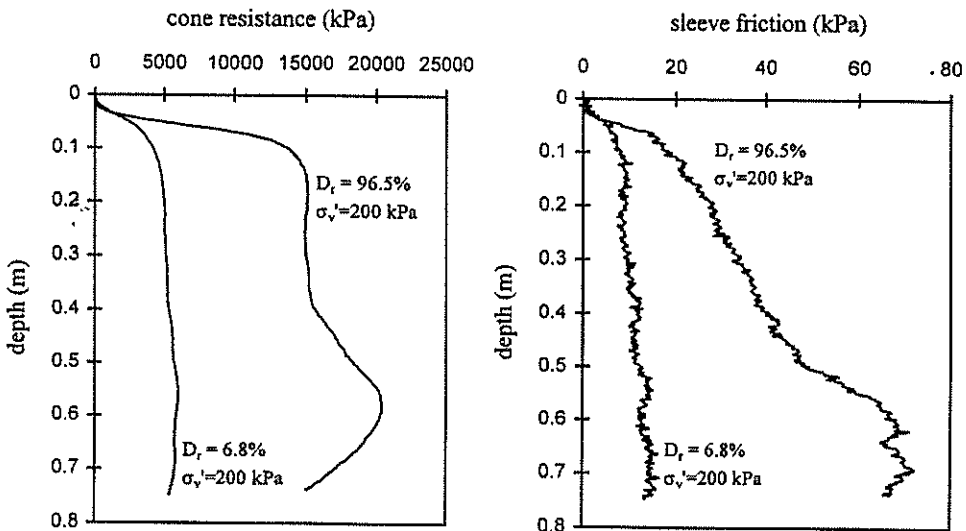


Figure 19. Typical Results from the Quartz Sand.

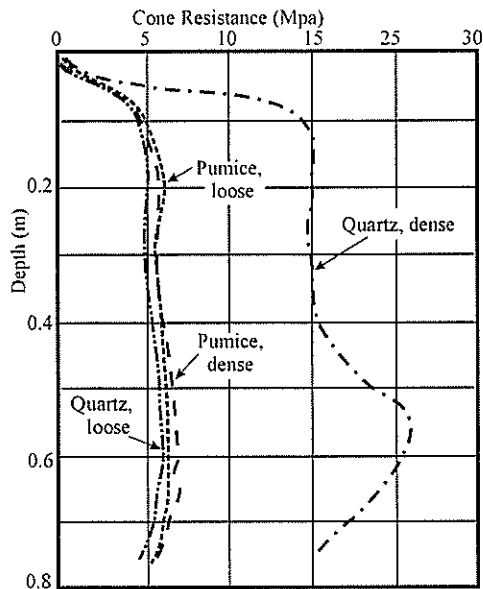


Figure 20. Comparison of Typical Results.

There is a surprising and dramatic difference in behaviour between the two sands as is readily apparent in these figures. The quartz sand behaves as expected, showing large differences in cone resistance between the loose and dense states, and steadily increasing values with confining stress. The pumice sand, on the other hand, shows quite startling behaviour, with the following rather extraordinary characteristics:

1. There is very little change in cone resistance between its loose and dense states, and the increase in resistance with confining stress is less pronounced than with the quartz sand.
2. The penetration resistance of the pumice sand is a little higher than that of the quartz sand in the loose state.
3. Despite showing very little difference in cone resistance, the pumice sand still shows a large difference in sleeve friction, comparable to that of the quartz sand.

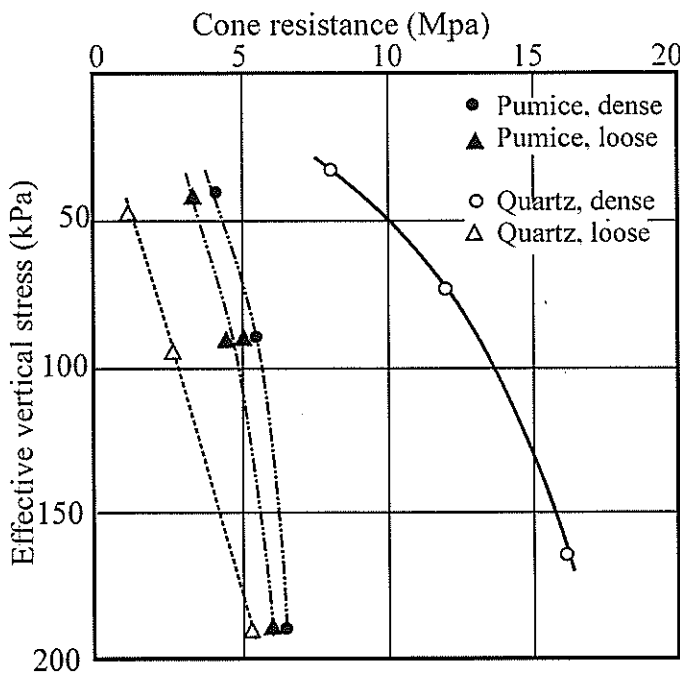


Figure 21. Chart Summarising All the CPT Results.

This dramatic difference in behaviour can only be attributed to the different in particle strength of the two sands. The initial interpretation put on the results was that with the quartz sand failure occurred by

shear displacement, while with the pumice sand failure occurred predominantly as a result of particle crushing. To investigate the extent of particle crushing, samples were taken from the immediate vicinity of the cone when emptying the chamber for the purpose of carrying out particle size measurements. "Immediate vicinity" in this context means a zone extending not more than 1 to 2 cm from the edge of the cone. These particle size measurements showed a surprising amount of crushing with both sands, especially in the dense states, and the crushing is only marginally higher with the pumice sand than the quartz sand. This comparison is purely qualitative, as it was not possible to be sure that the size and location of the samples was the same in each case. With the pumice sand the crushing was similar to that at a stress of 8000 kPa in the oedometer compression test (Figure 16).

Perhaps the explanation for the insensitivity of cone resistance to density with the pumice sand lies in the shear strength behaviour of the pumice sand revealed by drained triaxial tests. Typical triaxial test results, together with those from the quartz sand, are given in Figure 22. These show that with the pumice sand, in both the dense and loose state, the deviator stress slowly climbs towards a peak value, but there is no post peak decline in strength. The strain to reach the peak value is large, generally between 20% and 30%. This peak value is essentially the same regardless of whether the sand is initially in the dense or loose state. The failure values from triaxial tests on the two sands are illustrated in Figure 23.

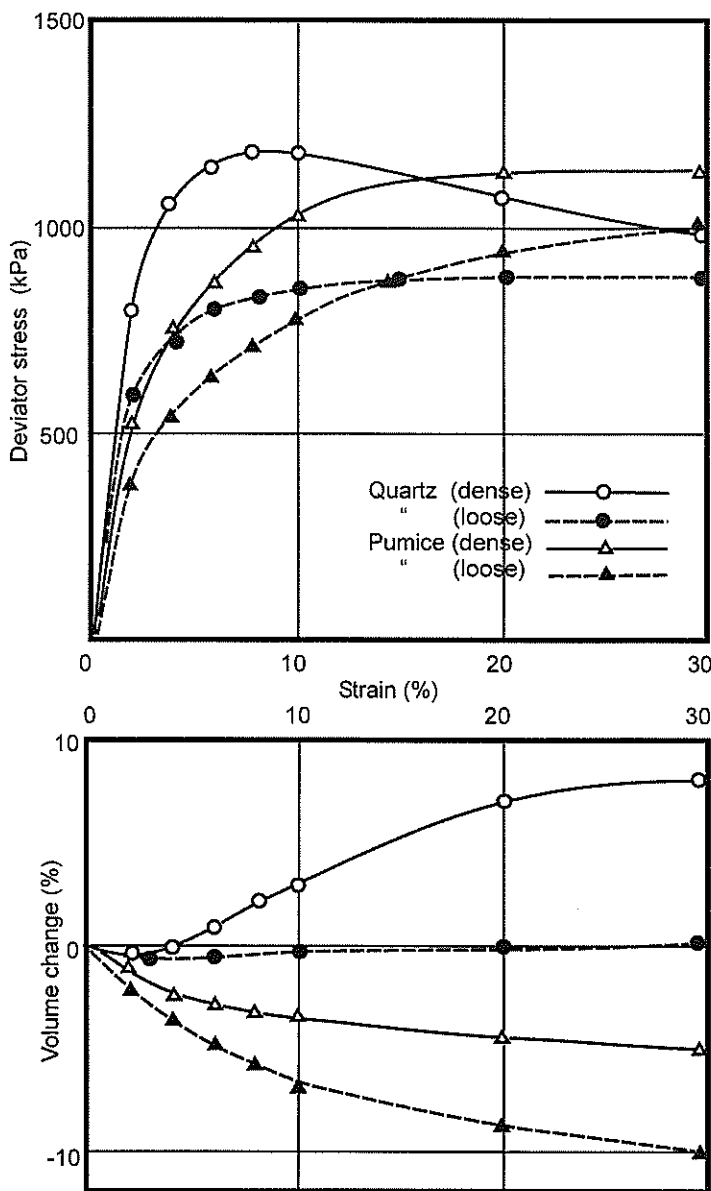


Figure 22. Typical Results of Triaxial Tests on Pumice and Quartz Sand.

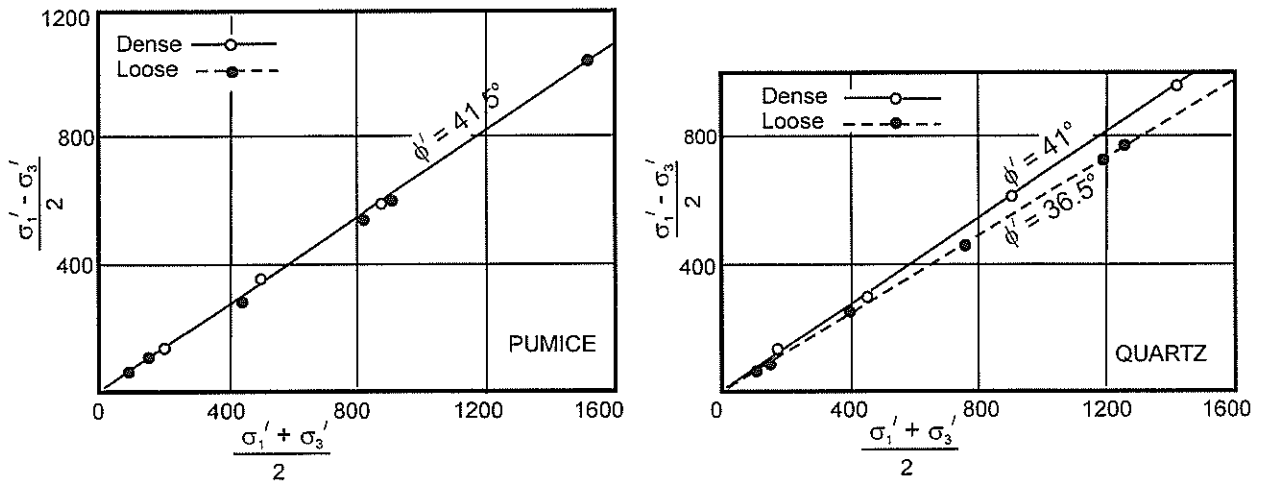


Figure 23. Results of Drained Triaxial Tests on the Two Sands.

The conclusion drawn from these results is that the resistance measured in the cone tests on the pumice sand simply reflects this common "ultimate" state revealed in triaxial testing, and which is little influenced by the initial density state.

HARDENING EFFECT IN COMPACTED PUMICEOUS MATERIALS

To close this paper, the results of another small piece of research are presented. Some materials, especially those of volcanic origin, tend to stiffen up or "harden" quite rapidly after compaction, even though they may be quite soft and displaying substantial "weaving" during compaction. With some of these materials, leaving them undisturbed for only a day or two is sufficient for them to become quite firm or hard. The question investigated here is whether this hardening is the result of some form of chemical reaction creating bonds or cementation between particles, or purely a pore pressure effect. The investigation was carried out by a post-graduate (M.Eng. Studs) student, who has described the work in detail (Kiryakos, 1997). Only the essential details and results are presented here.

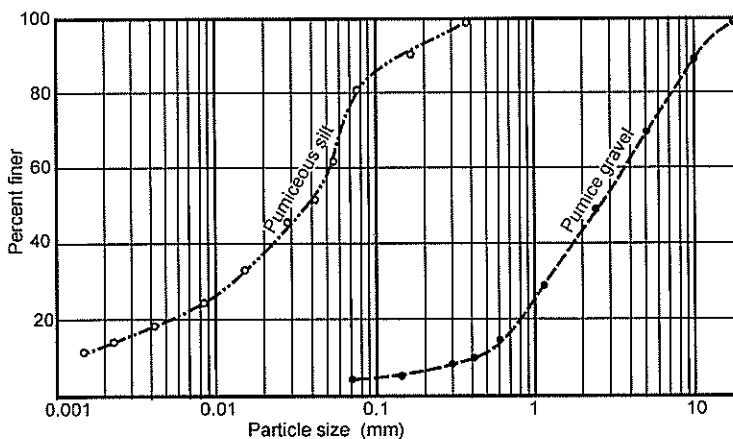


Figure 24. Particle Size Distribution Curves of the Two Materials.

Two pumice material were tested, one a silt and the other a gravel. Particle size and compaction test results are shown in Figure 24 and 25. A number of compacted samples of each material were then prepared by compacting them at identical water contents. The water content was about 2% wet of optimum in each case, this value being chosen so as to produce samples of relatively low strength comparable with those often produced during compaction in the field. The samples were compacted in brass tubes. Strength measurements were made immediately on two samples of each material to determine the initial, or "as-compacted" strength. The strength measurements were made using

unconfined compression tests for the gravel, and undrained triaxial tests (without a confining stress) for the silt.

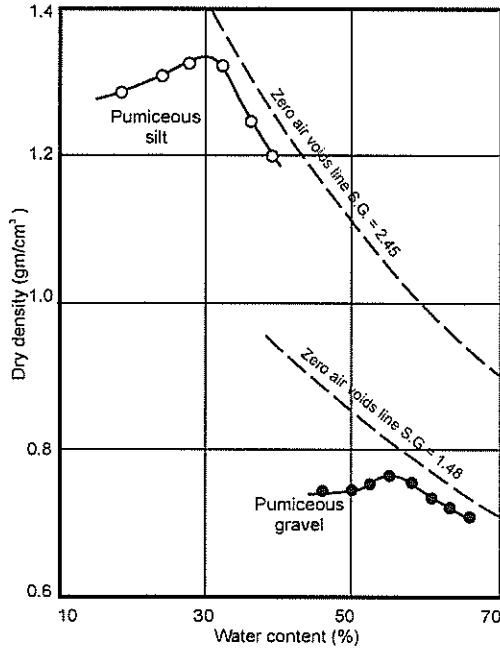


Figure 25. Compaction Test Results for the Two Pumiceous Materials.

One set of samples was then sealed by carefully waxing the tubes, and stored for testing later at specific time intervals. In this way any changes in strength in these samples could only result from chemical effects. The second set of samples was used to investigate changes in pore pressure. With the silt samples this was done by setting up the samples in a triaxial cell and connecting the porous

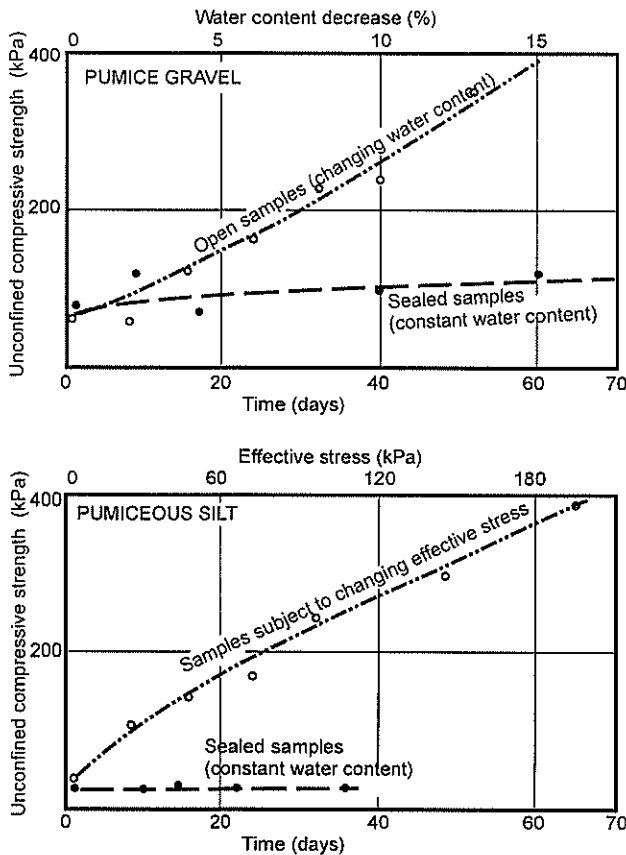


Figure 26. Changes in Strength with Time for the Two Pumiceous Materials.

stone at the base of the sample to a suction pump via a water/air interface. This induced negative pore water pressure (pore water tension) within the samples. Tests were done at pore water tensions from 25 kPa to nearly 100 kPa, this being the maximum value feasible with this technique. Ignoring a small correction because of incomplete saturation, this produced effective stresses in the samples equal to the applied suction. This procedure was used as it was considered to most closely replicate the situation in the field. With a low water table, as is often the case in well drained pumiceous areas, the material compacted at the surface will be subjected to a negative pore water pressure governed by the depth of the water table. To continue the investigation to higher stress levels, several more tests were carried out using cell pressure in the usual way to consolidate the samples.

The compacted gravel samples were very coarse, open materials, containing considerable air, and the technique used with the silt samples was not considered appropriate for the gravel samples. Instead these samples were simply extruded and left open to the air, so that evaporation took place and presumably induced increasing pore water tension within the gravel. Unconfined strength measurements were then made when weight measurements indicated moisture content reductions at 2% intervals. The two sets of sealed samples of both materials were opened at specific time intervals, and tested immediately after opening to ensure no loss of moisture. The results of the tests are shown in Fig 26 in the form of graphs of strength versus time and strength versus either pore water tension (for the silt) or water content reduction (for the gravel).

The results clearly show that strength changes only occur as a result of change in pore water tension or change in water content. The sealed samples tested at time intervals of up to 60 days show no significant or consistent change in strength with time. Hence the hardening characteristic is clearly a purely physical one resulting from changes in effective stress and not from chemical processes. In practice, it is likely that quite high pore water tension develops in both the silt and the gravel when they are left alone after compaction. The silt is frequently a subgrade material lying above a deep water table, so that the pore water tension will be approximately equal to the depth of the water table. The gravel, on the other hand, is often used on forest roads as the main pavement material, and is thus open to the atmosphere. Hence evaporation can occur which would be expected to create high pore water tension within the material.

CONCLUSION

Two particular volcanic soils have been described here, both of which show unusual properties. The clay minerals, allophane and imogolite, impart these distinctive properties to the allophane clay, and the soft grains impart the unusual properties to the pumice sand. The lessons from this study relevant to other volcanic soils are probably the following:

1. Volcanic soils are likely to behave somewhat differently from sedimentary soils. Geotechnical engineers should recognise this and not assume they will conform to preconceived patterns, especially those associated with sedimentary soils.
2. While every effort should be made to develop theoretical or behavioural frameworks to assist us in understanding and interpreting soil behaviour, we ought to recognise the limitations of such frameworks, and not seek to make all soils fit into these frameworks.
3. In evaluating the engineering properties of soils we ought to first observe carefully their behaviour in the field, before looking at their behaviour in laboratory tests
4. Some well established concepts, especially stress history and the use of the e -log p plot for analysing consolidation behaviour, are not necessarily appropriate for all soils, especially volcanic and residual soils.
5. Volcanic soils are so varied that it is unrealistic to expect them to fit into any single behavioural pattern.

REFERENCES

- Frost, R.J. (1967). "Importance of correct pre-testing preparation of some tropical soils," *Proc. First Southeast Asian Regional Conf. on Soil Engineering*, Bangkok, pp. 44-53.
- Jacquet, D. (1990). "Sensitivity to remoulding of some volcanic ash soils," In: *New Zealand Engineering Geology* 28(1) pp1-25.
- Kiryakos, M.D. (1997). "Investigation of the time hardening effect with two compacted pumiceous materials," *M.E. Project*, University of Auckland.
- Kuno, G., Shinoki, R., Kondo, T. & Tsuchiya, C. (1978). "On the construction methods of a motorway embankment by a sensitive volcanic clay," *Proc. Conf. on Clay Fills*, London, pp. 149-156.
- Moore, P.J., & Styles, J.R. (1988). "Some characteristics of volcanic ash soil," *Proc. Second Int. Conf. on Geomechanics in Tropical Soils*. Singapore, pp. 161-166.
- Parton, I. M. & Olsen, A.J. (1980). "Properties of Bay of Plenty Volcanic Soils," *Proc. 3rd Australia New Zealand Conference on Geomechanics*, Wellington. Vol.1, pp, 165-169
- Wada, K. (1989). "Allophane and imogolite". Chapter 21 of *Minerals in Soil Environments* (2nd Edition) SSSA Book Series No 1, pp. 1051-1087.
- Wesley, L.D. (1973). "Some basic engineering properties of halloysite and allophane clays in Java, Indonesia," *Geotechnique* 23, No 4: 471-494.
- Wesley, L.D. (1974). "Tjipanundjang Dam in West Java, Indonesia," *Journal of the Geotechnical Division ASCE* 100/GT5, pp. 503-522.
- Wesley, L.D. (1977). "Shear strength properties of halloysite and allophane clays in Java, Indonesia," *Geotechnique* 27, No2, pp. 125-136.
- Wesley, L.D. & Matuschka, T. (1988). "Geotechnical engineering in volcanic ash soils," *Proc. Second Int. Conf. on Geomechanics in Tropical Soils*, Singapore Dec. 1988. Vol.1. pp. 333-340.
- Wesley, L.D. (1998). "Some lessons from geotechnical engineering in volcanic soils," *Proc. Int. Symposium on Problematic Soils*. Sendai, Japan, Vol.1, pp. 851-863.
- Wesley, L.D. and Chan S.Y. (1990) "The dispersivity of volcanic ash soils," *Proc. IPENZ Conference* 1991. Vol. 1, pp.67-76.
- Wesley, L.D., Meyer, V.D., Satyawan Pranyoto, Pender, M.J., Larkin, T.J., and Duske, G.C. (1999). "Engineering Properties of a Pumice Sand," *Proc. Eighth Australia New Zealand Conference on Geomechanics*, Hobart.
- Wesley, L.D. (2002). "Geotechnical characterisation and properties of allophane clays". *Proc. International Workshop on Characterisation and Engineering Properties of Natural Soils*. Singapore, Dec. 2002.