

Behaviour and geotechnical properties of residual soils and allophane clays

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An overview of the properties of residual soils is given in the first part of the paper. The different processes by which residual and sedimentary soils are formed are described, and the need to be aware that procedures applicable to sedimentary soils do not necessarily apply to residual soils is emphasised. In particular, it is shown that the log scale normally used for presenting oedometer test results is not appropriate or relevant to residual soils. The second part of the paper gives an account of the special properties of allophane clays. Their abnormally high water content and Atterberg limits are described, and it is shown that despite this, their geotechnical properties are remarkably good. Methods for control of compaction of residual soils and allophane clays are also described.

Keywords: residual soils, volcanic, allophane clays, consolidation, shear strength, compaction

En la primera parte del artículo se entrega una descripción general de los suelos residuales. Se detallan los diferentes procesos en los cuales son formados los suelos residuales y sedimentarios, poniendo énfasis en la necesidad de estar atento a que los procedimientos aplicados a los suelos sedimentarios no son necesariamente aplicables a los suelos residuales. En particular, se muestra que la escala logarítmica generalmente usada para presentar resultados de ensayos edométricos no es apropiada o pertinente para suelos residuales. La segunda parte del artículo da cuenta de las propiedades especiales de arcillas alofánicas. Se describen sus altos valores de contenido de agua y límites de Atterberg y se muestra que a pesar de esto, sus propiedades geotécnicas son sorprendentemente buenas. También se describen métodos de control de compactación para suelos residuales y arcillas alofánicas.

Palabras clave: suelos residuales, volcánico, arcillas alofánicas, consolidación, resistencia al corte, compactación

Introduction

Soil mechanics grew up in northern Europe and North America, and most of its concepts regarding soil behaviour developed from the study of sedimentary soils. In fact, most of the early concepts came from the study of remoulded sedimentary soils and involved investigating the influence of stress history on their behaviour, in the belief that this was simulating the influence of stresses which soils may be subject to during their formation processes. Most text books on soil mechanics and university courses on the subject place considerable emphasis on stress history – soils tend to be divided into normally consolidated and over-consolidated on this basis, and behavioural frameworks are developed around this stress history concept. This might be all very well if all soils were sedimentary soils. This of course is clearly not the case. Large areas of the earth (including large areas in the North Island of New Zealand) consist of residual soils, and the application of concepts coming from sedimentary soils may or may not be relevant to these soils. It is

interesting to note that very few text books, and probably very few university courses on soil mechanics, even mention residual soils, let alone give an adequate account of their properties.

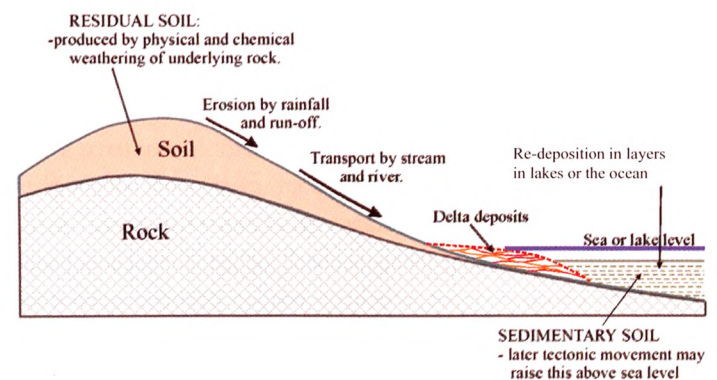


Figure 1 : Diagrammatic representation of soil formation processes

Formation processes

Figure 1 shows diagrammatically the physical processes that to the formation of sedimentary and residual soils.

Residual soils are formed directly from the physical and chemical weathering of the parent material, normally rock of some sort. Sedimentary soils are formed by a depositional process, normally in a marine or lake environment. Figure 2 is an attempt to summarise the factors involved in the formation processes that influence the properties of the two soil types. Sedimentary soils are seen to undergo a various additional processes beyond the initial physical and chemical weathering of the parent rock. It might appear from this diagram that the factors involved in the formation of sedimentary soils are more complex than those involved in forming residual soils. There is some truth in this, but in practice two important factors lead to a degree of homogeneity and predictability with sedimentary soils that is absent from residual soils. These factors are:

- The sorting process which take place during erosion, transportation and deposition of sedimentary soils tend to produce homogeneous deposits.
- Stress history is a prominent factor in determining the behavioural characteristics of sedimentary soils, and leads to the convenient division of these soils into normally and over consolidated materials.

The absence of these factors with residual soils means that they are generally more complex and less capable of being divided into tidy categories or groups.

It is perhaps helpful to consider that the behaviour of a soil, whether residual or sedimentary, is dependent on two factors, or two groups of factors. These are, firstly the nature of the soil particles themselves (*i.e.* their size, shape, and mineralogical composition) and secondly, the particular state in which these particles exist in the ground. For convenience, these factors can be referred to respectively as composition and structure. With sedimentary clays, the influence of composition is well known – kaolinite group clays are relatively “inert” with consequent low shrinkage/swell characteristics and relatively low compressibility, while montmorillonite clays are highly active and of opposite characteristics to the kaolinite group. Notwithstanding the influence of mineralogy, by far the most important “attribute” of sedimentary clays in their undisturbed state (at least according to conventional soil mechanics) is their stress history *i.e.* whether they are normally consolidated or over-consolidated. This is generally given greater importance in the literature than either mineralogy or structure.

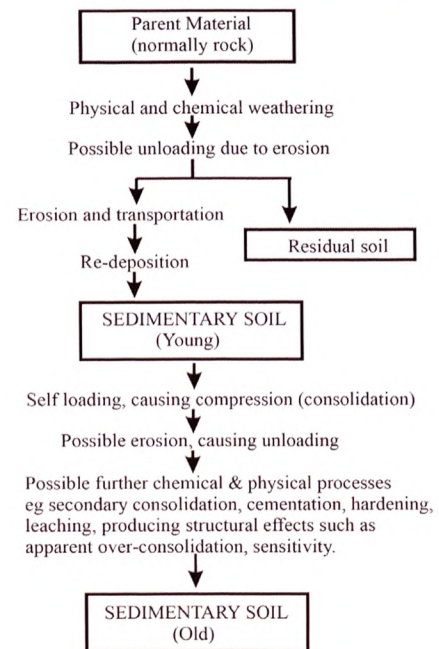


Figure 2: Soil formation factors influencing soil behaviour

With residual soils, mineralogy remains an important influence, but stress history is not a concept which has much if any relevance. The physical and chemical weathering processes that form these soils produce particular types of clay minerals, and particular “structures” *i.e.* particular arrangements of the particles, and possibly bonding or cementing effects between particles. These influences are infinitely more important than stress history. The terms normally consolidated and overconsolidated are therefore not directly relevant to residual soils.

Grouping and classification of residual soils

Various attempts have been made to group or classify residual soils, but none are particularly useful. Some, such as that of the British Geological Society (1990) make use of soil science classifications and are not very useful for engineering purposes. Terms such as vertisols, andosols, *etc.* are not normally meaningful to engineers, and the variation in properties within these groups is likely to be so large as to make the grouping of little relevance.

Focussing on the two factors discussed above, namely mineralogical composition and structure, provides a basis for dividing residual soils into groups that can be expected to have fairly similar engineering properties. Starting with mineralogy, the following groups can be established:

- Soils without a strong mineralogical influence (those containing low activity clays): many residual soils

fall into this category, especially those derived from the weathering of sandstones, or igneous rocks such as granite. These soils are likely to be fairly coarse grained with a small clay fraction. Structure is likely to be an important concept in understanding the behaviour of these soils. The weathered granite soils of Hong Kong and Malaysia fall into this group.

(b) Soils with a strong mineralogical influence, from “conventional” clay minerals (*i.e.* those containing high activity clays): one very important worldwide group comes into this category – the “black cotton” soils or “vertisols”, also called Houston Black Clay in Texas, Tropical Black Earths of Australia, “Tirs” of Morocco *etc.* The predominant clay mineral is smectite, a group of which montmorillonite is a member. These black cotton soils are highly plastic, highly compressible and of high shrink/swell potential. Structural effects are almost zero with these soils. They normally form in poorly drained areas, and have poor engineering properties.

(c) Soils with a strong mineralogical influence, coming from special clay minerals not found in sedimentary clays: the two most important clay minerals found only in certain residual soils (especially tropical residual soils of volcanic origin) are halloysite and allophane. These are both silicate clay minerals. Apart from silicate minerals, tropical soils may contain non-silicate minerals (or “oxide” minerals), in particular the hydrated forms of aluminium and iron oxide, gibbsite and goethite. The most unusual of these minerals, in terms of understanding soil behaviour is allophane.

Soils of group (c) which contain these unusual minerals include:

(i) Tropical red clays – predominant mineral is halloysite but may also contain kaolinite, with gibbsite and goethite. Halloysite particles are generally very small in size but are of low activity, and soils containing halloysite as the predominant mineral generally have good engineering properties. Red clays generally form in well drained areas in a tropical climate having a wet and dry season. Red clays may be referred to as lateritic soils or as latosols. There is a wide range of engineering properties found in red clays, but they should not be confused with laterite itself.

(ii) Volcanic ash soils (or andosols or andisols): these are found in many tropical and sub-tropical countries (including New Zealand) and are formed by the weathering of volcanic “glass”. The predominant clay

mineral is allophane (frequently associated with another mineral called imogolite).

(iii) Laterites: the term laterite is used very loosely, but should refer to deposits in which weathering has reached an advanced stage and has resulted in a concentration of iron and aluminium oxides (the sesquioxides gibbsite and goethite), which act as cementing agents. Laterals therefore tend to consist of hard granules formed by this cementing action; they may range from sandy clays to gravels, and are used for road sub-bases or bases.

Table 1 shows this grouping system for residuals soils, and Table 2 attempts to list some of the more distinctive characteristics of these soil groups and indicates the means by which they may possibly be identified.

Following on from mineralogy, the next characteristic which should be considered is structure, which refers to specific characteristics of the soil in its undisturbed (*in situ*) state. Structure can be divided into two categories:

(a) Macro-structure, or discernible structure: this includes all features discernible to the naked eye, such as layering, discontinuities, fissures, pores, presence of unweathered or partially weathered rock and other relict structures inherited from the parent rock mass.

(b) Micro-structure, or non-discernible structure: this includes fabric, inter-particle bonding or cementation, aggregations of particles, pores, *etc.* Micro-structure is more difficult to identify than macro-structure, although it can be inferred indirectly from other behavioural characteristics such as sensitivity. High sensitivity indicates the presence of some form of bonds between particles which are destroyed by remoulding.

This grouping system is intended to help geotechnical engineers find their way around residual soils, and to draw attention to the properties likely to be of most significance for geotechnical engineering. It is not intended to perform a function as a rigorous classification system. Some comments on local or Southeast Asian soils may be helpful at this stage.

Weathered Waitemata clays (Auckland, NZ) : This is an example of a group which does not fit comfortably in any category and this in itself tells us something about these clays. Some Waitemata “clays” are essentially silts, and are not strongly influenced by clay minerals - they belong to Group A. Others are very highly plastic

Table 1: A classification or "grouping" system for residual soils

GROUPING SYSTEM		COMMON PEDOLOGICAL NAMES USED FOR GROUPS	DESCRIPTIVE INFORMATION ON IN SITU STATE	
MAJOR DIVISION	SUB-GROUP		PARENT ROCK	INFORMATION ON STRUCTURE
GROUP A Soils without a strong mineralogical influence.	(a) Strong macro-structure influence	Miscellaneous	Give details of type of rock from which the soil has been derived.	Describe nature of structure: - stratification - fractures, fissures, faults etc. - presence of partially weathered rock. (%?)
	(b) Strong micro-structure influence	Miscellaneous		Describe nature of micro- structure and/or evidence of it: - influence of remoulding - sensitivity - liquidity index.
	(c) Little or no structural influence	Miscellaneous		Indicate evidence for little or no structural effect.
GROUP B Soils strongly influenced by normal clay minerals	(a) Smectite (montmotillonite) group	Black cotton soils Black soils Tropical black earths Grumusols Vertisols		
	(b) Other clay minerals ?	?		?
GROUP C Soils strongly influenced by clay minerals essentially found only in residual soils	(a) Allophane sub-group	Volcanic ash soils Andosols or andisols Andepts		Give basis for inclusion in this group. Describe any structural influences, either macro- structure or micro-structure.
	(b) Halloysite sub-group	Tropical red clays Latosols Oxisols Ferralsols		As above
	(c) Sesquioxide sub-group - gibbsite, goethite, haematite.	Lateritic soils Laterites Ferrallitic soils Duricrusts.		Give basis for inclusion in this group. Describe any structural effects - especially cementation effects or the sesqui-oxides.

Table 2: Characteristics of residual soils groups

GROUP		EXAMPLES	MEANS OF IDENTIFICATION	COMMENTS ON LIKELY ENGINEERING PROPERTIES AND BEHAVIOUR
MAJOR GROUP	SUB-GROUP			
GROUP A Soils without a strong mineralogical influence.	(a) Strong macro- structure influence	Highly weathered rocks from acidic or intermediate igneous or sedimentary rocks	Visual inspection	This is a very large group of soils (including the "saprolites") where behaviour (especially in slopes) is dominated by the influence of discontinuities, fissures etc.
	(b) Strong micro- structure influence	Completely weathered rock, formed from igneous or sedimentary rocks	Visual inspection, and evaluation of sensitivity, liquidity index etc.	These soils are essentially homogeneous and form a tidy group much more amenable to rigorous analysis than Group (a) above. Identification of nature and role of bonding (from relic primary bonds to weak secondary bonds) important to understanding behaviour.
	(c) Little or no structural influence	Soils formed from very homogeneous rocks	Little or no sensitivity, uniform appearance.	This is a relatively minor sub-group. Likely to behave similarly to moderately over-consolidated soils.
GROUP B Soils strongly influenced by normal clay minerals	(a) Smectite (montmotillonite) group	Black cotton soils, and many similar dark coloured soils formed in poorly drained conditions.	Dark colour (grey to black) and high plasticity suggest soils of this group.	These are normally problem soils, found in flat and low lying areas, having low strength, high compressibility, and high swelling and shrinkage characteristics.
	(b) Other clay minerals ?			Likely to be a very minor sub-group.
GROUP C Soils strongly influenced by clay minerals essentially found only in residual soils	(a) Allophane sub-group	Soils weathered from volcanic ash in the wet tropics and temperate climates	Position on Plasticity Chart, and irreversible changes on drying.	Characterised by very high natural water contents and Atterberg Limits. Engineering properties generally good, though in some cases high sensitivity may make earthworks difficult.
	(b) Halloysite sub-group	Soils often derived from volcanic material, especially tropical red clays.	Reddish colour, well drained topography, and volcanic origin are useful indicators.	These are generally very fine grained soils of low to medium plasticity, and low activity. Engineering properties generally good. (Note that there is often some overlap between halloysite and allophane clays).
	(c) Sesquioxides - gibbsite, goethite, haematite.	Laterites, or possibly some red clays referred to as "lateritic" clays.	Non-plastic or low plasticity materials, generally of granular, or nodular appearance.	This is a very wide poorly defined group, ranging from silty clay to coarse sand and gravel. Behaviour ranges from low plasticity silty clay to gravel. These materials are the end products of a very long weathering process.

clays, resulting from the presence of smectite (montmorillonite) minerals - and belong in Group B. The two types may occur in quite close proximity, *i.e.* in interbedded layers. It appears that the weathering process in this case is not actually creating the clay minerals; it is simply destroying the weak bonds which “lock” the clay minerals into the parent material. Waitemata clays may or not exhibit macro-structure as well as micro-structural effects.

Weathered greywacke soils (Wellington, NZ): These probably belong in Group A, as their properties are not strongly influenced by their mineralogical content. They are likely to exhibit significant macro-structure effects, dependent on their degree of weathering.

Weathered granite soils (worldwide): These also belong to Group A, and exhibit macro-structural effects - from joints and presence of “floating” un-weathered rock boulders.

Volcanic ash (allophane) soils (worldwide): These clearly belong to Group C. They are very strongly influenced by their mineral composition. They are unlikely to exhibit significant macro-structure, but may exhibit some micro-structure - significant sensitivity for example.

Tropical red clays (many tropical countries): These also belong to Group C. Those found in the island of Java, Indonesia (with which the author is familiar) are rather unusual in that they exhibit neither macro-structure nor micro-structure, except when the weathering is not far advanced. In this case they may show traces of the structure of their parent material.

Geotechnical engineering in residual soils

In the following sections some comments will be made on issues of direct relevance to geotechnical engineers, namely foundation design, slope stability and compaction. They are not comprehensive and should not be taken as generalisations applicable to all residual soils.

Foundation design

Consolidation behaviour

(a) Magnitude (stress/deformation curves). Figure 3 shows typical consolidation test results from one residual soil type - the tropical red clay found in Java, Indonesia. Although it is standard practice to plot consolidation test results as void ratio versus log

pressure graphs, it is often informative to also plot them as direct compression graphs using linear scales. The lower part of Figure 3 shows the linear plots. The results show the following points:

(i) Conventional graphs (e -log p) suggest the clays behave as moderately over-consolidated soils, although there is no clearly defined “pre-consolidation” or “vertical yield” pressure. It appears to be somewhere between 100 kPa and 500 kPa.

(ii) When plotted using a linear scale, the picture is quite different. The curves are reasonably close to linear, especially over the pressure range likely to be of engineering interest, generally about 0 to 200 kPa. The evidence of a “yield” stress has largely disappeared.

It is not suggested that the curves in Figure 3 are representative of residual soils in general. They are presented primarily to illustrate that the standard e -log p graph can be quite misleading and may imply the existence of “pre-consolidation” or “yield” pressures when no such pressure exists. With residual soils (and possibly also with sedimentary soils) it is generally desirable to plot consolidation test results using a linear scale for pressure as well as the normal log scale before drawing any conclusions about the behaviour of the soil. Some residual soils show quite distinct “yield” pressures, while others show steadily increasing stiffness with stress level, and some demonstrate almost linear behaviour.

Figure 4 is presented to show the influence of remoulding on compression behaviour for three different residual soils. These are respectively an allophane clay, a tropical red clay, and a silt derived from weathered Waitemata sandstone. Consolidation curves are given for the soil in its undisturbed state, its remoulded state, and after mixing it with water to form a slurry. These last curves can be regarded as the “virgin” consolidation lines for the soil in its completely remoulded state. It is seen that with the allophane clay and the Waitemata silt, remoulding results in a very significant change in the compression curve. These soils clearly have a relatively stiff structure in their undisturbed state which is destroyed by re-moulding (or “de-structuring” to use the in vogue term for this effect). The red clay on the other hand shows almost no change in behaviour after remoulding. This is often the case with red clays. They appear to exist naturally in a dense unstructured state close to their Plastic Limit, and remoulding thus has little or no effect on them.

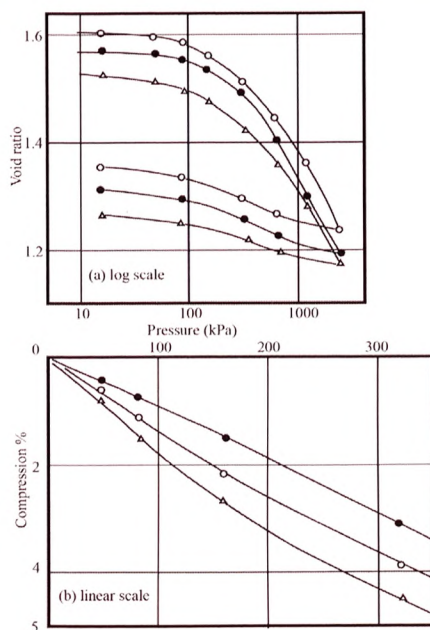


Figure 3: Oedometer test results from a tropical red clay

With regard to the estimation of settlement magnitude, there are two procedures commonly used in soil mechanics. The first is to use the parameters C_c and C_s which are obtained from the $e - \log p$ plot, and the second method is to use m_v values. For soils which give an approximately straight line on a linear stress/compression plot the use of m_v seems most appropriate. The choice of method is a matter for individual judgement, based primarily on the actual soil behaviour in consolidation tests. With residual soils the m_v parameter often seems more appropriate than the C_c or C_s parameters.

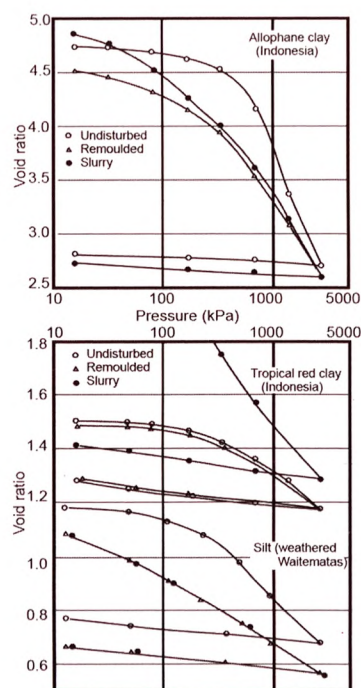


Figure 4: Influence of remoulding on $e - \log p$ graphs

It should be noted that for settlement estimates with sedimentary soils, there are various empirical constructions or corrections for improving the accuracy of estimates. The best known are probably the Schmertman construction and the Skempton and Bjerrum method. Both these methods are based primarily on stress history concepts and are not intended for residual soils. Therefore the use of these methods with residual soils is highly questionable. There are no established procedures available for correcting consolidation curves for residual soils to allow for sample disturbance (such as the Schmertman method for sedimentary soils) and hence it is very important to obtain good quality undisturbed samples for consolidation tests.

One further factor which should be appreciated when attempting to predict settlement magnitudes of foundations on residual soils is that the initial stress state in the ground is likely to be unknown if the water table is at some depth below the surface. The pore pressures above the water table will be negative (*i.e.* in a suction or “tension” state), and likely to vary between winter and summer. During prolonged dry periods the suction value may be quite large. This means that the initial effective stress in the ground is not known and likely to vary between winter and summer. This is a fact commonly ignored in routing settlement effects. This situation is illustrated in Figure 5.

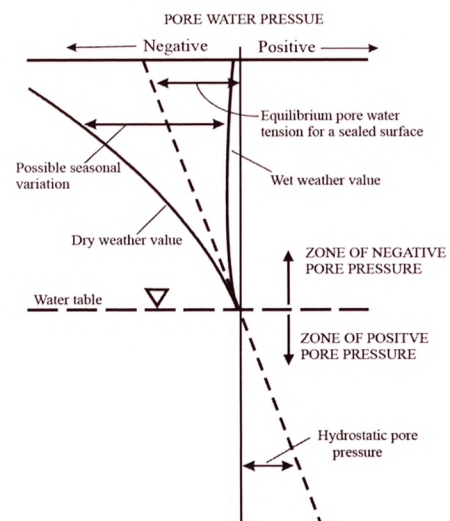


Figure 5: Pore water pressure state above and below the water table

(b) Consolidation rate: consolidation rates with residual soils tend to be rather faster than with sedimentary soils; as evidenced by their behaviour, both in the laboratory and in the field. This appears to be due to higher permeability associated with their undisturbed

structure. In consolidation tests the rate of pore pressure dissipation may be too fast to allow reliable determination of the coefficient of consolidation. This is demonstrated in Figure 6 which shows standard graphs of compression versus root time for the loading increment 100 kPa to 200 kPa for three residual soils. The normal straight line section, which is used to determine t_{90} is not clearly defined. Hence, the estimation of c_v is problematical. It is usually found that at higher stresses the graphs become more linear; the higher stress tends to destroy the original structure and lower the permeability.

It should be appreciated that there is an upper limit to the value of coefficient of consolidation which can be measured in a conventional consolidation test. Analysis shows that the highest value of c_v which can be reliably measured with a 19 mm thick sample is about 0.1 m²/day (=0.012 cm²/s). Soils with c_v values greater than this will not show distinct straight lines on a conventional compression versus root time plot. If reliable values of c_v are required for soils which behave in this way, it is probably best to use a different method of measurement, such as a pore pressure dissipation test in a triaxial cell.

Table 3 shows the wide range of c_v values covered for the three soils of Figure 6.

Table 3: Values of c_v for the three soil types

Soil	c_v m ² /day
Waitemata silts and clays	0.01 to 10
Indonesian red clays	0.07 to 0.7
Volcanic ash soils	0.01 to 200

These values lie above and below the value of 0.1 m²/day that can be measured in the standard consolidation test.

Figure 7 illustrates the influence which remoulding may have on consolidation rate. The two curves are for the same stress increment, from 100 kPa to 200 kPa. Remoulding destroys the soil structure responsible for its high permeability and the much slower rate of consolidation produces the normal straight line on the root time plot.

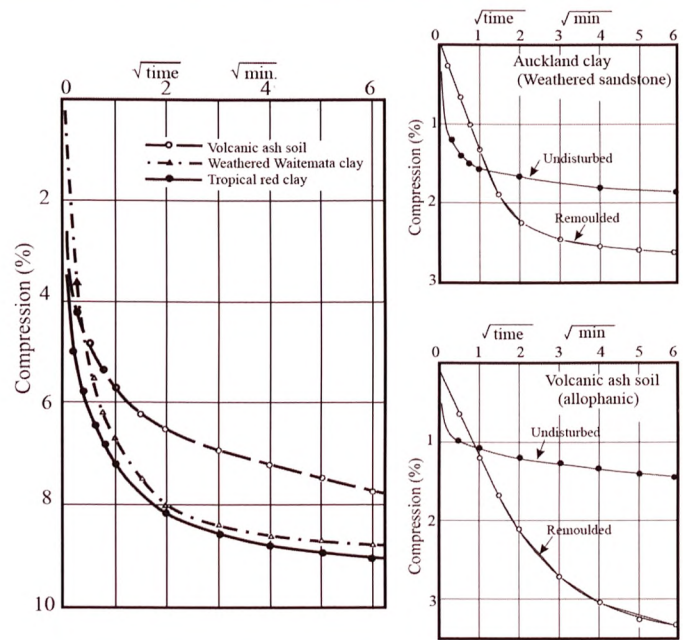


Figure 6: Typical root time graphs from residual soils

Figure 7: Influence of remoulding on consolidation rate

Shear strength

It is not possible to make many categorical statements regarding the shear strength of residual soils; the following observations are generalisations and should be treated with some caution. It is reasonably true to assert (excluding montmorillonite “black cotton” soils) that the shear strength of residual soils, whether expressed as undrained shear strength or effective strength parameters, is generally higher than that of sedimentary soils. It is rare for the undrained strength to be less than about 75 kPa, and is generally between 100 and 200 kPa. Their ϕ' values are generally above 30°, and they have significant values of the cohesion intercept c' . In the case of some allophane rich volcanic ash soils both the peak ϕ'_p and residual ϕ'_r values may be higher than 35°. Figure 8 shows the results of triaxial tests on two residual soils; the first is for volcanic ash soils and the second for a clay (known as Middle clay) derived from weathered sandstone.

The results from volcanic ash soil in the upper figure show a relatively small variation in the shear strength; this is not surprising since volcanic ash soils are generally free of discontinuities and are of reasonably uniform composition. The lower figure shows the influence of structural defects (macro-structure) in the parent rock that are still present in the soil. It is clear that in the latter case it would be almost impossible to infer reasonable design parameters from results of this sort.

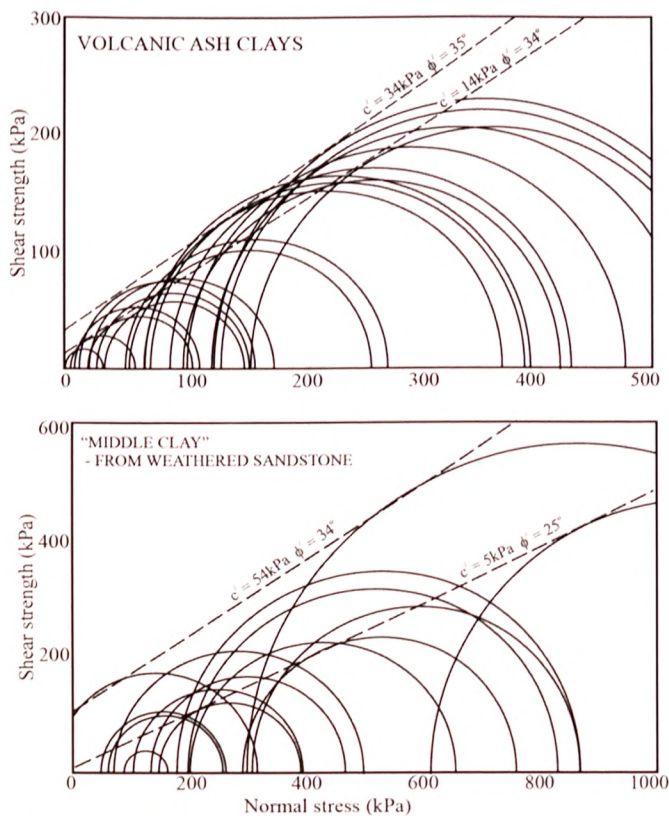


Figure 8: Triaxial test results from two types of residual soils

Bearing capacity

As mentioned above, the permeability and consolidation rates with residual soils are generally high, and in situations where residual soils are subject to external loading by the construction of foundations it is likely that generated pore pressures will dissipate almost immediately and the soils will remain in the drained state. This means that design using undrained strength will be conservative, as there will be some increase in strength as the load on the foundation increases.

However, this is not an argument against the use of undrained strength to estimate the bearing capacity of the soil for foundation design purposes. During rapid load application, such as during seismic loading, the soil will still behave in an undrained manner, and for this reason especially, design should be based on undrained strength. There are also strong practical arguments in favour of using undrained strength, as this can be measured relatively easily and reliably. Both field methods (*e.g.* Dutch penetrometer) and laboratory methods (unconfined compression or vane test) can be used to obtain reliable undrained strength values, whereas the measurement of drained strength parameters c' and ϕ' is more difficult and less certain.

Slope stability

There are several aspects of the stability of residual soil slopes that are of particular interest to the geotechnical engineer. These include the following:

- (a) slopes in residual soils (excluding “black cotton” soils) generally remain stable at much steeper angles than those in most sedimentary soils. Slopes of 45° or steeper are not uncommon, and cuts can often be made as steep as 60° without danger of slip failure,
- (b) slope failures in residual soils, especially when steep slopes are involved are unlikely to be deep seated circular failures. They are more likely to be relatively shallow, with fairly planar failure surfaces. However, the volume of material involved may still be very large,
- (c) slips and landslides in residual soils generally occur during periods of heavy rainfall, and are the result of temporary increases in the pore water pressure in the slope,
- (d) the value of c' is usually significant and is considered to be due to some form of weak bonds between particles,
- (e) the residual strength is likely to be closer to the peak strength than is the case with many sedimentary soils, especially in clays containing allophane or halloysite,
- (f) with some (possibly the majority) residual soils, the presence of discontinuities may be the governing factor.

Factors (c) and (f) are very important with respect to the use of analytical (slip circle) methods for assessing stability. Factor (c) is particularly important; with sedimentary clays of low permeability the pore pressures can be measured and the assumption made that they will remain approximately the same for a long time. With residual soils, any measurement of pore water pressure in the slope is valid only at the time it is made and is not relevant to long term stability estimates. For such estimates it is the worst condition likely to occur in the future which is of importance. Factor (f) is likely to dominate the behaviour of many cut slopes in residual soils, and rule out the use of analytical methods. Figure 8 shows an example of such a soil. Only in very rare situations is it likely to be possible to determine the location, orientation, and strength of discontinuities with the degree of reliability needed for the use of analytical methods.

The rapid changes in pore water pressure that occur with residual soils mean that stability analysis must be carried out in terms of effective stresses. The only exception to this might be when an embankment is constructed on a residual soil; this situation is similar to a foundation situation and undrained strength could be used.

It is worth noting that there is some evidence that pore water pressure in a slope will only change significantly as a result of periods of heavy rainfall if the c_v value is greater than about 0.1 m²/day, see Kenney and Lau (1984).

Compaction of residual soils

One last property of residual soils that has caused difficulties to engineers relates to their compaction behaviour. There are two problems, as follows:

- The variability of residual soils may mean a large and rapid variation in optimum water content within short distances in any borrow pit.
- Some compaction curves for residual soils, notably volcanic ash soils do not show peaks indicating maximum dry densities and optimum water contents.

Neither of the above “problems” are real problems in the sense of indicating that residual soils are more difficult to compact than sedimentary soils. If there is a problem, it is only in the evaluation of the soils and the method to be adopted for specifying and controlling the compaction. Many volcanic ash soils can be effectively compacted at water contents in the range of 100% to 180%, a fact which geotechnical engineers are often reluctant to accept.

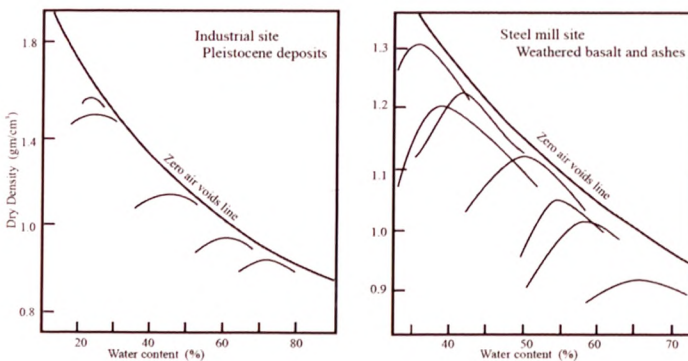


Figure 9: Compaction curves from residual soils on two sites near Auckland

Figure 9 shows the results of compaction tests carried out on a number of different samples from two sites involving residual soils. It is evident that there is a very

wide range of optimum water contents and maximum dry densities. Figure 10 shows the result of a compaction test on a volcanic ash sample from Java, Indonesia. The test has first been carried out by drying the soil in stages from its natural water content. The soil has then had water added to it after various degrees of drying, and further compaction tests carried out. The results show the very flat compaction curve obtained from the natural soil, and also the very significant influence which drying has on the soil properties. Any value of optimum water content can be obtained by varying the extent of pre-drying.

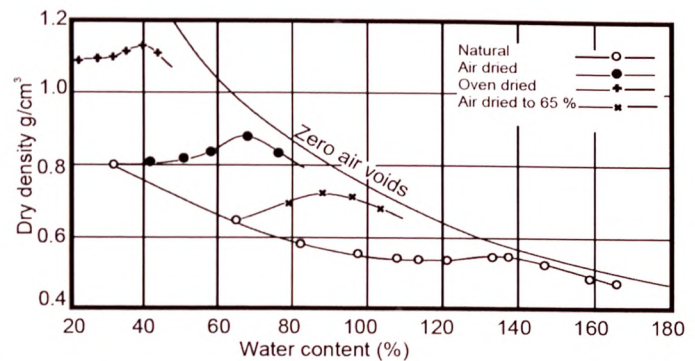


Figure 10: Compaction test result from a volcanic ash soil (Indonesia)

The behaviour illustrated in Figures 9 and 10 means that the control of compaction by the conventional method of specifying dry density and water content limits based on standard compaction tests is very difficult. Alternative methods of compaction control have been developed for such soils which overcome the above difficulties. The simplest method is that which is based on undrained strength and air voids criteria and is described by Pickens (1980).

The principle of the method is to specify a minimum value of shear strength (commonly 100 kPa to 150 kPa) and a maximum value of air voids (commonly 8 to 12%) for the compacted soil. These values can be varied according to the nature of the job and the soil or weather conditions at the site.

Figure 11 illustrates the principle of the method in relation to the conventional method based on water content and maximum dry density. The requirement of a minimum strength means that the soil must not be too wet, and the requirement that the air voids not exceed a certain value means that the soil must not be too dry.

The method is easy to use and control testing involves density and water content measurements in the usual way.

The values obtained are not significant in themselves; they are simply used to calculate the value of the air voids. At each control point, measurements are also made of shear strength. The simplest method of doing this is by using a hand operated shear vane, such as the "Pilcon" vane.

The actual values of optimum water content and maximum dry density of the soil do not need to be known, and it is not essential to carry out normal compaction tests at all. Such tests may however be useful in order to know whether much drying of the soil will be needed in order to be able to effectively compact it.

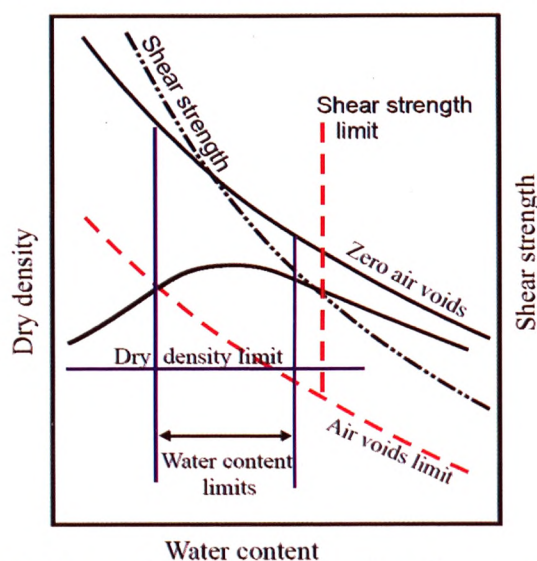


Figure 11: Compaction control limits using shear strength and air voids criteria

Comments on normal identification and classification tests

The tests normally used as a starting point in the evaluation and classification of soils are particle size measurement and the Atterberg limits. The applicability of such tests to residual soils is a matter of some contention within the profession; it is useful therefore, to examine the arguments put forward to suggest that these tests are of less relevance to residual soils than to sedimentary soils. The arguments are as follows:

- (a) Classification tests are carried out on the remoulded soil, and since remoulding destroys the important structural features which dominate the behaviour of many residual soils the tests indicate very little about undisturbed behaviour.
- (b) Some residual soils contain a large proportion of coarse particles, and since Atterberg limits are carried

out on the fine fraction only, they do not give a good indication of the properties of the soil as a whole.

- (c) The particles of some residual soils are of a weak and fragile nature and are broken down into smaller particles during testing.
- (d) The results of these tests are influenced by pre-drying the soil, and the plasticity limits are also dependent on the amount of mixing carried out prior to testing.
- (e) Empirical relationships between either particle size or Atterberg limits and other engineering properties have been developed from sedimentary soils and are not necessarily valid for residual soils.

There is some validity in all of these arguments, but we should be careful in our evaluation of them; they are certainly not valid for all residual soils on a general basis. In the case of one important residual soil group, namely the "vertisols" (or Black Cotton soils) it is likely that none of these arguments is of any relevance at all.

Arguments (a) and (b) above are not peculiar to residual soils; they frequently apply also to sedimentary soils, and in any case classification tests are frequently used for the evaluation of fill materials in which case it is the properties of the remoulded soil which are required.

Argument (d), at least with respect to the influence of pre-drying the soil, is not a valid argument against the use of classification tests, since there is no difficulty at all in carrying out the tests without pre-drying the soil.

Argument (e) above is perhaps the most important question to be considered, especially with respect to the Atterberg limits. It has been the author's experience that with residual soils the position which a soil occupies on the conventional Plasticity Chart provides a good indication of properties - probably just as good as with sedimentary soils. Soils which plot well below the A-line behave as silts while those which plot well above the A-line behave as clays. Figure 12 shows the position on the Plasticity Chart of the three most distinctive residual soils - the "Black Cotton" soils, the tropical red clays, and the allophane clays.

Problems arise when attempts are made to relate specific soil properties, or classification boundaries to one or other of the liquid and plastic limits. For example, the British classification system (BS 5930: 1981) divides soils up into a number of categories based on the liquid limit.

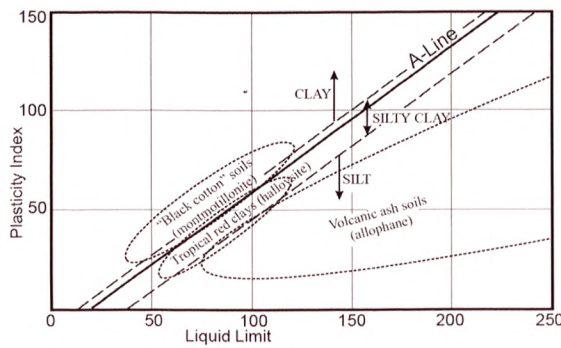


Figure 12: The Plasticity Chart and residual soils

Such a division is not very relevant to residual soils. It is the position above or below the A-line which is of most significance, especially with tropical residual soils.

Rather than a subdivision based on the liquid limit, a subdivision along the lines shown in Figure 12 would be most relevant to residual soils. The lines drawn parallel to the A-line divide soils into three types labelled clay, silty clay, and silt. Many residual soils behave as silty clays for engineering purposes, and rightly fall into the category of silty clay on this chart. The more distinctive residual soil types, such as “Black Cotton” soils, and allophane clays, would rightly be classified as clays and silts respectively.

It should be noted that the influence of increased mixing (or even drying) of the soil on the Atterberg limits is to move the point on the plasticity chart parallel to the A-line; hence if we use distance above or below the A-line as our main criteria for evaluating soils this movement is not of great significance. Hence argument (d) above is not very important.

Empirical relationships based on particle size or Atterberg limits

There are some rather vague general relationships involving particle size and Atterberg limits, and there are specific empirical relationships.

Among the general relationships is the understanding that as particle size decreases (or possibly as Liquid Limit L.L. increases) the properties of a soil become less favourable for engineering purposes. This is generally true (or held to be true) if a particular soil type is being considered. This understanding may well apply to many residual soils, but there is very considerable evidence that it does not apply to halloysite or allophane soils. Especially with allophane soils, there is no evidence of decrease in strength or increase in compressibility with either decreasing particle size or increasing L.L.

Specific empirical relationships would be those such as:

$$C_c = 0.009 (L.L. - 10) \quad (1)$$

This relationship is for remoulded N.C. soils and thus has no relevance to engineering situations in residual soils. In general, these types of relationships should hold for materials of conventional clay mineralogy. For residual soils containing allophane or even halloysite they may not be valid.

General remarks on residual soils

If there are lessons to be learnt from geotechnical engineering in residual soils, they are probably the following:

- Geotechnical engineers ought to have open minds about how soils may behave, and not assume they will conform to preconceived patterns, especially when working with residual soils.
- 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.
- 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.
- Some well established procedures, such as the use of the e -log p plot for analysing consolidation behaviour, are not necessarily appropriate for all soils, especially residual soils.
- With residual soils, the mode of formation is so varied that it is unrealistic to expect them to fit into a single behavioural pattern.

The special properties of allophane (volcanic ash) clays

Occurrence

There are substantial areas in the New Zealand 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 30 m deep, while site investigation drilling has shown depths of up to almost 40 m. 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 300 mm.

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.

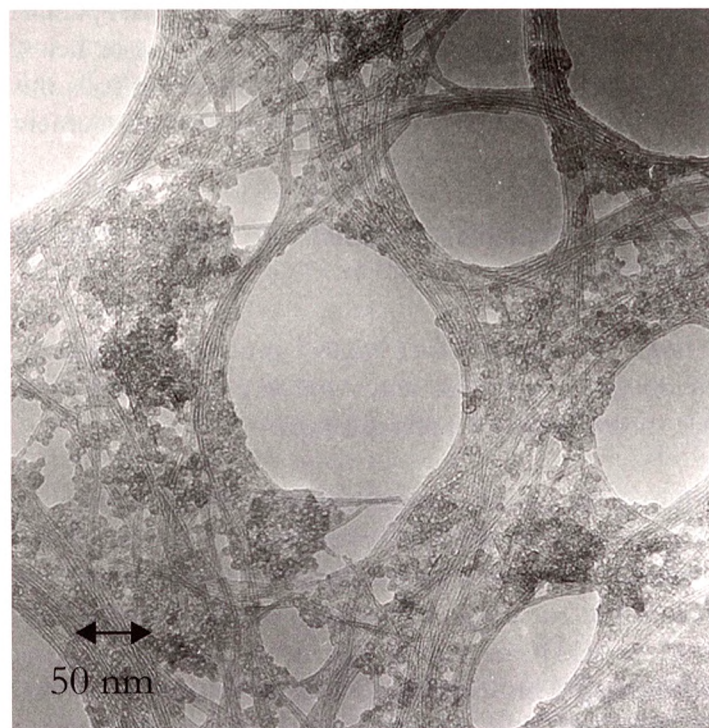


Figure 13: Electron micrograph of allophane and immogolite (after Wada, 1989).

Figure 13 shows an electron micrograph of the material in its undisturbed state. 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.

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 allophane clay areas in many countries 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.

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 14 shows a plot of the Atterberg limits on the Plasticity Chart.

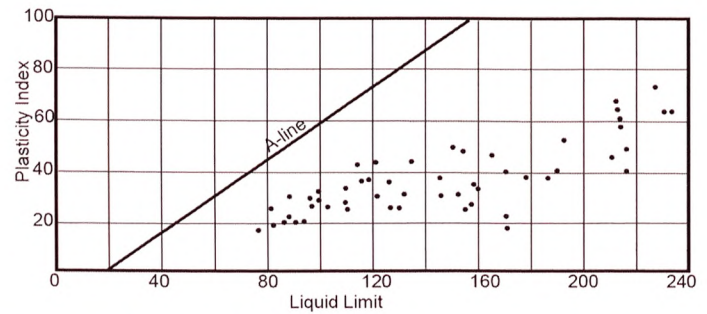


Figure 14. Atterberg limits of Allophane clays on the Plasticity Chart.

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, *i.e.* without air or oven drying, unless otherwise stated.

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:

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

If all of these apply then the soil almost certainly contains a significant allophane content.

Stiffness and compressibility

Typical results from oedometer tests on undisturbed samples from Indonesia and New Zealand are shown

in Figures 15 and 16. Details of the samples are given in Table 4.

Table 4: Details of samples used for oedometer tests.

Sample Number	Location Information	Depth m	Void Ratio <i>e</i>	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

Figure 15 shows the results as conventional *e-log p* graphs and Figure 16 as 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 arises from the structure of the soil created by the weathering process, and is perhaps best described as a vertical yield 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.

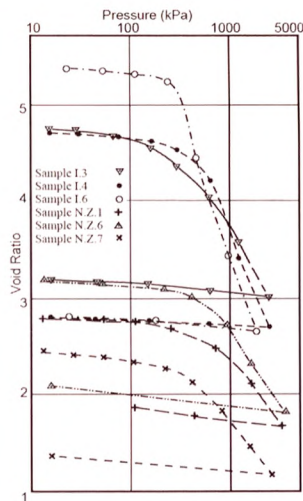


Figure 15. Oedometer test results as *e-log p* plots

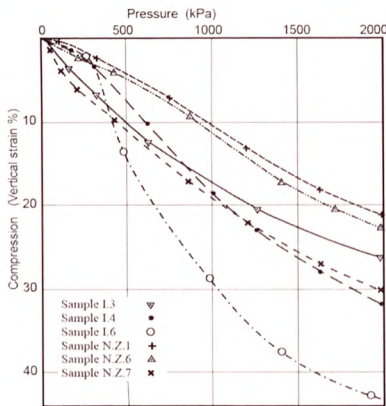


Figure 16. Oedometer tests showing compression versus pressure on a linear scale

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*.

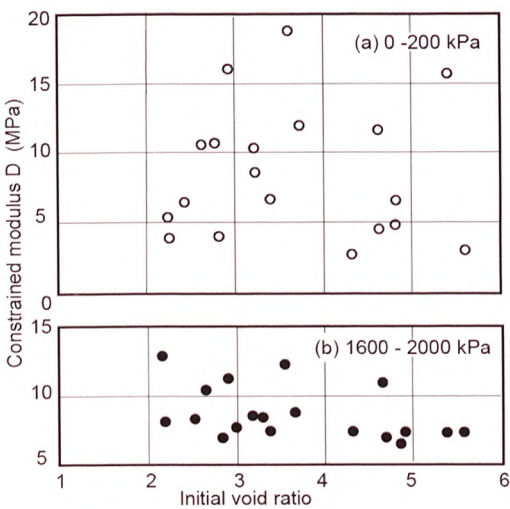


Figure 17: Constrained modulus (*D*) versus initial void ratio

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. Figure 17 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.

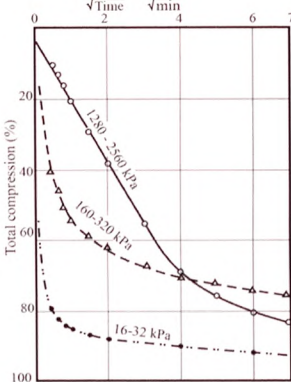


Figure 18: Typical root time plots from oedometer tests

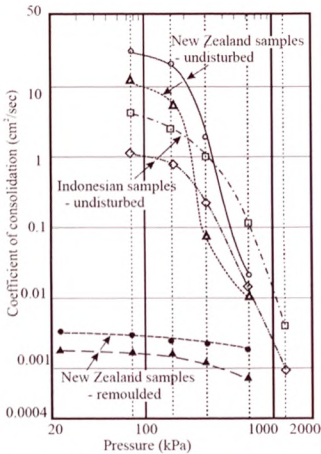


Figure 19: Summary of *c_v* values from pore pressure dissipation tests

Figure 18 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 Figure 19. 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 19 shows that remoulding the soil apparently destroys the open structure of the undisturbed soil, which is believed to account for the high permeability.

As noted earlier, 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 is about 0.01 cm²/s. At the relatively low stress levels relevant to engineering situations, the c_v value of allophane clays is normally much higher than this.

Undrained strength

Figure 20 shows cone penetrometer test (CPT) results from two sites, one in Indonesia and one in New Zealand.

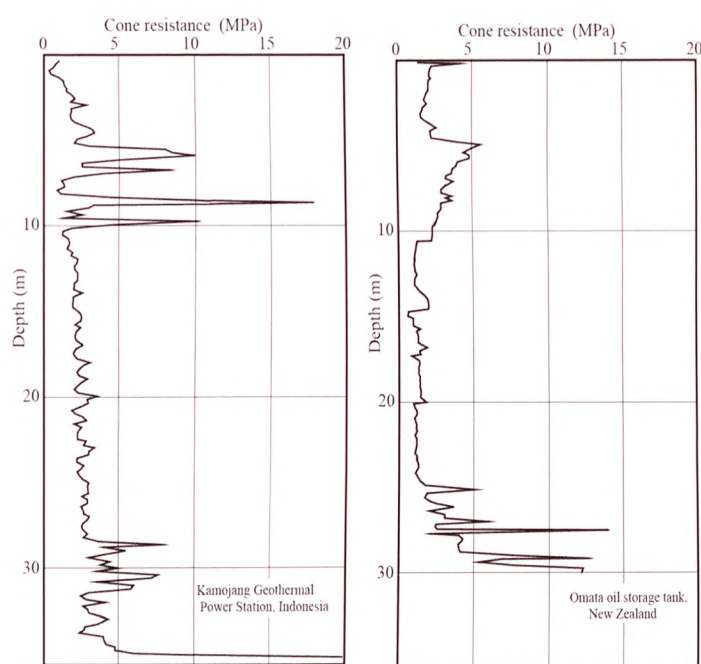


Figure 20: Cone penetrometer tests from allophane clay sites in Indonesia and New Zealand

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 ϕ' and c' 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.

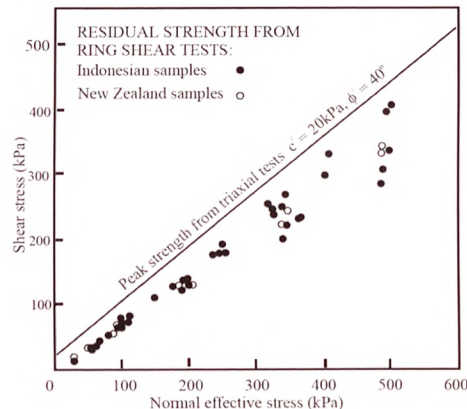


Figure 21: Effective strength parameters for allophane clays

Figure 21 summarises results from laboratory tests on samples of allophane clay 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 22 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° .

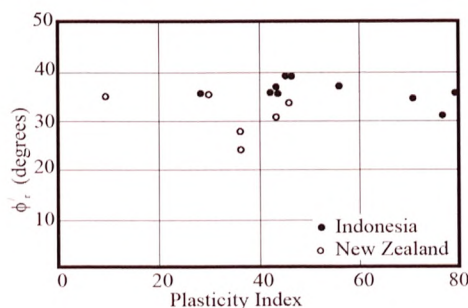


Figure 22: Residual strength friction angle from allophane clays versus Plasticity Index.

Compaction characteristics

The compaction behaviour of typical allophane clay was illustrated earlier in Figure 10. The natural water content was 166%, and 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 $w = 65\%$). 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. New Zealand allophane clays may not show such a dramatic effect because of their lower allophane content.

Figure 23 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 Figure 23. 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”.

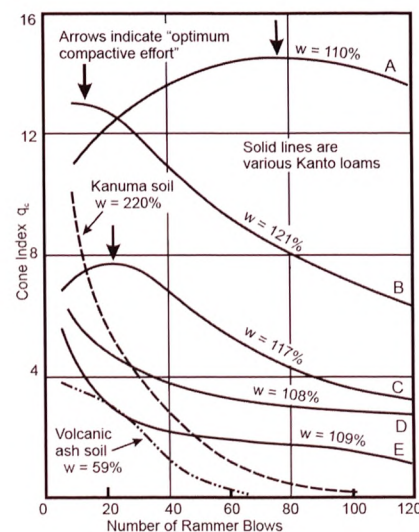


Figure 23: Influence of compactive effort on strength of compacted allophane clays (after Kuno *et al.*, 1978)

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 and Olsen (1980), and Moore and 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 power station 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 an electrical transformer and 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.

In relation to erodibility, the writer has investigated the question of the dispersivity of allophane clays by carrying out pin-hole dispersion tests on allophane clays from Indonesia and New Zealand. The results are described by Wesley and Chan (1991). None of these tests showed any evidence of erosion or dispersivity.

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 (formerly spelt Tjipanundjang) in West Java, Indonesia, built in 1928 during the Dutch colonial period. This is a homogeneous 30 m 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 the municipal water supply of the city of Bandung, the capital city of West Java. The Mangamahoe Dam in New Plymouth, New Zealand, and the embankment supporting the supply canal at the Kuratau power scheme (on the western shore of Lake Taupo, New Zealand) are further examples of embankments of allophane clay forming water retaining structures. The Kamojang geothermal power station in West Java, Indonesia, is supported by a raft foundation on about 35 m of allophane clay (Figure 20). There have been no problems with its performance. Wesley and Matuschka (1988) describe these examples in greater detail.

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