Structural behaviour of residual soils of the continually wet Highlands of Papua New Guinea

WALLACE, K. B. (1973). *Géotechnique* **23**, No. 2, 203–218.
Discussion by De and Furdas (1973). *Géotechnique* **23**, No. 4, 601–603.

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Wallace has attributed the high shear strength and low compressibility of the residual soils of the Papua New Guinea highlands to the existence of a porous soil skeleton of rock-forming minerals enclosing a viscous gel of hydrated clay minerals. He also states that compression and shear tests show that the particles of the skeleton are cemented together at their contacts to form a structural framework. With this concept the hydrated clay minerals play only a secondary role in influencing the behaviour of the soil.

The evidence for this structure is indirect and not very convincing and is not supported by the behaviour of similar allophane and halloysite soils in Indonesia. The most obvious objection to the proposed structure is that in most of the allophane soils in Java there is insufficient coarse material for such a structure to exist. The clay fraction in these soils is generally between 60% and 80% and the proportion of sand-size material is usually only about 5%. Water contents vary from about 100% to 190% (the same range as in the soils described by Wallace) which correspond to void ratios from 2·7 to 5·1 or porosities from 0·73 to 0·84. Thus only 16% to 27% of the volume of the soil is made up of solid material. Of this solid material about 25% is likely to be silt-size and 5% sand-size so that the proportion of total volume occupied by material coarser than clay-size will be only about 6% to 9%. If rock-forming minerals are considered to be of sand-size, then less than 1% of the total volume will be made up of rock-forming minerals. Even in the Author's samples there do not appear to be sufficient rock-forming minerals to make possible the structure proposed. The Author's sample 1 for example has a water content of 130% and by weight contains 15% of rock-forming minerals. This means that 22% of the soil volume is occupied by solid material and only about 3% (22% × 15%) is occupied by rock-forming minerals. In the Author's Fig. 6 however, about 30% of the space appears to be occupied by rock-forming minerals.

The evidence for cemented bonds from consolidation tests is not at all definitive. As Wallace indicates, many residual soils display compressibility characteristics rather similar to overconsolidated soils but it does not follow that they all contain cemented bonds. There is no *a priori* reason why a residual soil should behave more like a normally consolidated than an overconsolidated soil. The compression characteristics in this case are dependent on the structure and composition resulting from the weathering process in the same way as the behaviour of a sedimentary soil is dependent on its stress history.

The importance of 'structure' in the residual soils under discussion here can be determined to some extent by carrying out œdometer tests on the undisturbed soil and on the same soil remoulded at its natural water content. The remoulded soil tested by Wallace appears to have had a water content much greater than the natural water content so that it is difficult to assess the real effect which remoulding has on the structure. This point is illustrated in Fig. 1 which shows the results of œdometer tests on three samples from allophane and halloysite groups in Java. Details of the samples are given in Table 1. In each case tests were carried out on the undisturbed soil, on the soil remoulded at natural water content, and on the soil

(a) sample B

(b) sample C

(c) sample D

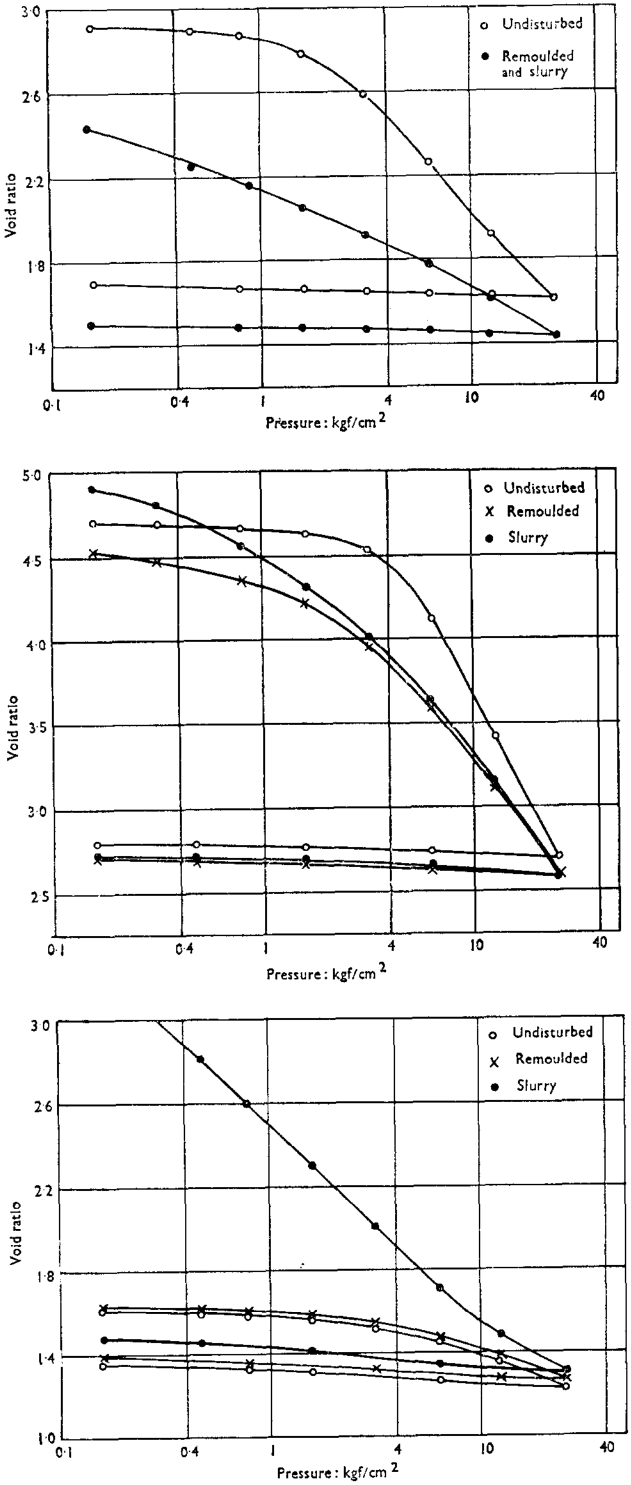

Fig. 1. Results of œdometer tests on three residual clays of volcanic origin

mixed to a slurry at a water content near the liquid limit. Fig. 1(a) shows the result obtained from a 'young' soil in which the texture of the parent rock (tuff or ash) was still visible. The natural water content was higher than the liquid limit so that remoulding the soil turned it to a slurry and the remoulded and slurry e–log p curves are identical. Remoulding this soil appears to destroy its structure and release a large amount of water held in the structure so that the remoulded e–log p curve lies at a much lower void ratio than the undisturbed soil.

Figure 1(b) shows a soil in which weathering was more advanced and almost no trace was left of the parent rock. This sample was predominantly allophane. In this case the effect of remoulding on the e–log p curve is much less pronounced suggesting that the structure of the undisturbed soil plays a less important role in determining its consolidation characteristics. The slurry curve now lies above the remoulded curve.

The third sample in Fig. 1(c) is from a predominantly halloysite clay representing the most advanced stage of weathering. With this soil remoulding has no significant effect on the e–log p curve, suggesting that the undisturbed structure has virtually no influence on the consolidation characteristics of the soil.

With each of the three samples the e–log p curve is similar to that of an overconsolidated sedimentary clay with an initial flat portion and apparent pre-consolidation pressure much higher than the existing in situ overburden pressure. However, this fact in itself provides very little information about the nature or importance of the in situ structure of the soil. With the first sample the in situ structure plays an important role but with the third sample the structure is of little or no importance as destruction of the structure by remoulding has no effect on the consolidation behaviour; this is in spite of the fact that it is this third sample which appears to be the most heavily overconsolidated.

Wallace has attempted to show that compressibility is linearly dependent on porosity. However he identifies compressibility with the compression index C measured over that part of the e–log p curve above the apparent critical pressure. The value of $C/(1+e_0)$ would be a better measure of compressibility, but in any case the compressibility which is of primary importance to the engineer is that part of the e–log p curve below the critical pressure since the bearing capacity is not likely to be in excess of the critical pressure.

There is no evidence from allophane and halloysite clays in Java of any relationship between porosity (or void ratio) and the compressibility of the soil below the critical pressure. Fig. 2 shows a plot of compressibility against void ratio in which the compressibility is the decrease in thickness per unit thickness of sample between 0 and 1·5 kgf/cm^2. The latter figure is an approximation of the mean bearing capacity of these soils. These results give no indication of an increase in compressibility of the undisturbed soil with increasing void ratio (or porosity).

Table 1

Sample reference number	Liquid limit	Plasticity index	Natural water content, %	Passing sieve No. 200, %	Clay fraction, %	Principal clay minerals
A	88	19	79·2	88	58	—
TJ1*	165	46	128	96	76	Allophane
TJ2*	95	30	68	96	65	Halloysite
Y3*	213	46	181	97	—	Allophane
B	83	18	97·4	—	—	—
C	175	49	157·4	93	68	Allophane
D	104	32	56·9	98	77	Halloysite

* These samples have been described elsewhere (Wesley, 1973).

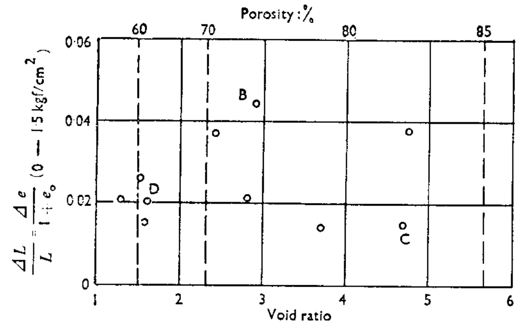

Fig. 2. Compressibility of undisturbed samples against void ratio

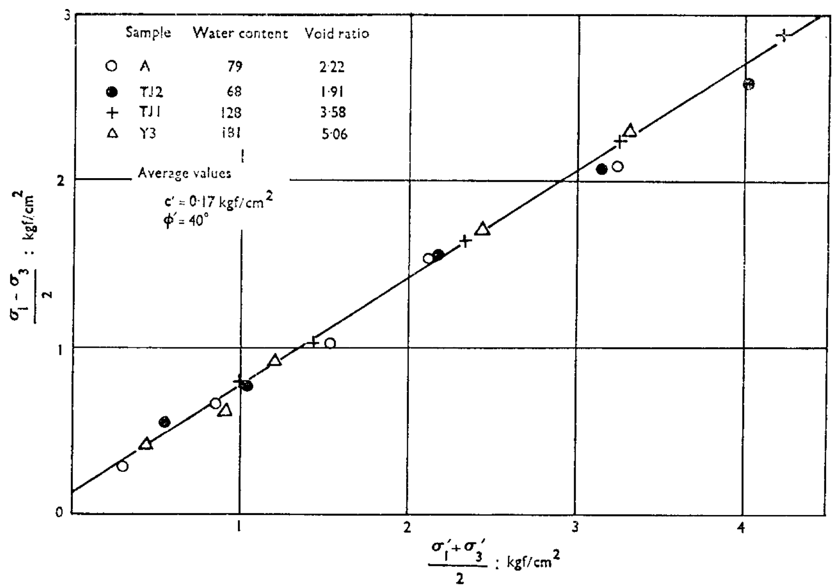

Fig. 3. Results of triaxial tests on halloysite and allophane samples

In my view, void ratio (or porosity) is not at all useful as a guide to the likely compressibility or shear strength of allophane and halloysite clays. This point is emphasized by the shear strength results shown in Fig. 3. This shows a plot of $(\sigma_1 - \sigma_3)/2$ against $(\sigma'_1 + \sigma'_3)/2$ at failure from consolidation, undrained triaxial tests on four samples. All four samples were taken from a plateau south of the city of Bandung in West Java. Details of the samples are given in Table 1. Tests on soil A were on undisturbed samples, but tests on TJ1, TJ2 and Y3 were on the soil compacted at its natural water content. In each case tests were carried out at five cell pressures. It is seen that the values from all four samples lie surprisingly close to the same straight line. This line represents a ϕ' value of 40°. The very close agreement may be to some extent fortuitous but the results emphasize two points. First, the in situ structure does not appear to be an important factor as destruction of the structure by remoulding and compacting does not substantially lower the shear strength. Second, the shear strength is not related to void ratio or porosity. The void ratio of the samples in Fig. 3 ranges from about 1·9 to 5·1.

It should be pointed out that the void ratio quoted above is an apparent void ratio measured in the conventional manner. With the allophane samples a certain proportion of the water may be bound up in the 'gel' in such a way that it may be better considered as part of the solid material than as free water occupying void space.

The foregoing results suggest that the properties of allophane and halloysite soils cannot be explained by some particular structure such as that proposed by Wallace (or by the cluster hypothesis put forward earlier by Terzaghi (1958)). They must be attributed to the unusual characteristics of the clay minerals themselves.

REFERENCES

Terzaghi, K. (1958). Design and performance of the Sasumua Dam. *Proc. Instn Civ. Engrs* **9**, 369–394.
Wesley, L. D. (1973). Some basic engineering properties of halloysite and allophane clays in Java, Indonesia. *Géotechnique* **23**, No. 4, 471–494.

Author's reply to Wesley

Wesley's data for similar Indonesian soils is a stimulating contribution. A constructive comparison shows many basic similarities between the Indonesian and New Guinean soils. Both have high porosity and highly hydrated kandite clay minerals although the Indonesian samples were less sandy and more weathered. The œdometer compression and rebound characteristics are similar with obvious critical pressures within the ranges I quoted. Remoulding eliminated the critical pressure for Indonesian sample B and this effect was less obvious with sample C. Remoulding was not effective in breaking down the much stronger structure of the less compressible sample D.

The point of departure arises in Wesley's rejection of the concept of a cemented structure. He indicates that the volume of coarse material that is obtained by mechanical analysis is insufficient for such a continuous three-dimensional structural framework. The idealized structure sketched in my Fig. 5 is a three-dimensional conception containing both rock-forming minerals and aggregated clay minerals and it can occupy much less than 30% of the *volume* of the soil. Whether such a structural arrangement can be sustained by 6–9% of silt and sand plus weak clusters of clay particles is open to conjecture. During my studies I was surprised by the high stiffness and critical pressure of samples which had been dried and resaturated, and it seemed that after dehydration of the clay minerals there was still a strong intact structure. For samples in this state the water was no longer bound up in the gel and many of the characteristics of the clay would have been altered irreversibly. Another point which drew my attention to probably structural influences was that in shear box tests the soils exhibited drained strength characteristics at quite high rates of shear.

On the subject of compressibility correlations, Wesley has misinterpreted my work. Equations (2)–(4) of the Paper show that compressibility has in fact been measured in terms of $C/1+e_0$. The linear relationship with porosity is well defined and qualified in this section of the Paper. Wesley's Fig. 2 is quite a different matter; it refers to compressibility below the critical pressure and is in complete agreement with my results. Compressibility below the critical pressure was similar to the rebound value and, as can be read from Table 3, it was low and showed no pattern of variation with porosity.

My point on moisture content against porosity was a simple one. For description of these soils, porosity is a better basis for comparison than is moisture content. Certainly there must be more relevant parameters than either of these but if these parameters are to be used (as is the case in Wesley's Table 1 and Fig. 3), the porosity should be given.

The few shear strength results presented for the Indonesian and New Guinean soils show that shear strength can be high, even in a highly remoulded state. I attribute this to 'considerable interlocking of the coarse particles with only a small proportion of the total load being carried by the 'gel' whereas from the Indonesian soils it is attributed to the unusual characteristics of the clay minerals. What are these characteristics? Further work is required but because of the inherent variability of residual soil properties this is not an easy task.

Author's reply to De and Furdas

I am grateful for the constructive comments and accept the modified model of an idealized residual soil structure. Such models can be a useful conceptual aid in the interpretation and application of geotechnical test results even though they may never be completely justified by introspection. For example, with an overconsolidated soil, a continuously curved *e*–log *p* curve will most likely be the result of sample disturbance but the same characteristic for a residual soil may be due to variable bond strength and therefore would be much closer to the true field characteristic than would be interpreted according to experience with overconsolidated soils.

I was satisfied that, by examination through a low-powered stereo microscope, the fraction that was retained after wet sieving on a 75 μm sieve could be positively identified as described in Table 1. However with respect to the probable existence of clusters of silt and clay size particles which were broken down during preparation for particle size analysis, I agree that these clusters almost certainly existed.

While the high permeability of the dried, remoulded and saturated soil was definitely not the result of piping, it was observed that some clay material was removed from the samples during the permeability tests. Susceptibility to piping was not really studied but it is thought that it could be an important consideration with dried soil.

A note on the interpretation of Coulomb's analysis of the thrust on a rough retaining wall in terms of the limit theorems of plasticity theory

COLLINS, I. F. (1973). *Géotechnique* **23**, No. 3, 442–447.

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Collins' Paper is an excellent contribution to our understanding of earth pressure theories. I should like to make the following comments.

Much earlier than Heyman, Davis (1968), after performing an upper bound calculation for the passive thrust of a smooth wall on a cohesive frictional soil states 'The same result is obtained by a Coulomb wedge analysis, which is really only another way of performing the same calculation.'

Both Coulomb and Collins *assume* that the angle of wall friction is fully developed in the active state. The measurements carried out up to now on model walls seem to confirm this hypothesis (Matteotti, 1970; Fagnoul *et al.*, 1972; Rowe, 1969).

Once the angle of obliquity at the wall is fixed, it is not necessary to resort to the upper bound theorem so as to show that Coulomb's active thrust is a lower bound or that Coulomb's passive thrust is an upper bound.