Correlations between index tests and the properties of remoulded clays

W. D. CARRIER III and J. F. BECKMAN (1984). *Géotechnique* **34**, No. 2, 211–228

Dr S. Leroueil, Dr F. Tavenas and Dr J. Locat,
Laval University, Quebec

The design of disposal facilities for fine-grained mine wastes and dredgings poses particular problems for the geotechnical engineer who must evaluate in advance the compressibility, strength and permeability characteristics of the material to be deposited, from knowledge of such physical properties as grain size distribution and plasticity. In their Paper, the Authors have produced simple correlations between physical and mechanical properties of remoulded fine-grained soils for use in design. Such correlations are needed in practice. However, to be useful, such correlations must be free of any ambiguity about the type of properties correlated, the methods of measuring the correlated data and the relevance of the correlated properties to the problem to be solved. The Writers observe that these conditions are not fully satisfied in the Paper and they would like to submit the following comments, based on their experience with intact, destructured, remoulded and resedimented clays.

Clayey soils can be encountered or produced in four different states of structure

- (a) the intact state, as it occurs in natural deposits, which is the result of complex geological processes involving deposition environment, consolidation, erosion, thixotropic hardening, leaching, weathering etc.
- (b) the destructured state, observed when an initially intact clay is submitted to volumetric or shear deformations of such magnitude that the original clay structure is broken (Leroueil, Tavenas, Brucy, La Rochelle & Roy, 1979), which occurs for example in an oedometer test or under an embankment for stresses in excess of the preconsolidation pressure
- (c) the remoulded state, obtained when sufficient mechanical energy is imparted to a clay mass to reduce its strength to a minimum
- (d) the resedimented state, obtained by deposition of clay particles originally remoulded and mixed to a slurry and by consolidation under the self-weight of the soil column of

increasing thickness, which depends on the mineralogy and grain size of the soil, the salinity of the slurry, the rate of sedimentation and other environmental factors.

When considering the possible behaviour of mine wastes and dredgings, the resedimented state is the state of interest. Its characteristics, particularly in terms of shear strength, should not be confused with those pertinent to the remoulded state. Experimental studies, in particular by Locat (1982) and Locat & Lefebvre (1982), clearly evidence the different behaviours of a clay in the various structural states identified here.

Figure 1 shows the main results of a testing programme carried out on specimens of Grande Baleine Clay. The main properties of this clay in its natural intact state are listed in Table 1. The c_u and σ_p' values corresponding to the intact state I are shown in Fig. 1; they are characteristic of the highly structured marine clays of north western Quebec. The remoulded strength of specimens at the initial water content is represented by c_{ur} I, corresponding to a sensitivity of 150.

Two samples A and B of this clay were completely remoulded and mixed with water to form slurries at two different salinities respectively of 0.2 g/l and 35 g/l; their activities were 0.22 and 0.4 respectively. The slurries were then sedimented in special oedometer cells, 10.2 cm in diameter, in which the sedimented samples could be consolidated under vertical effective stresses increasing progressively to 52 kPa. At this stage the samples were taken out; their undrained shear strength in the sedimented-consolidated state (c_{uA} and c_{uB}) and in the remoulded state (c_{urA} and c_{urB}) were measured.

The results presented in Fig. 1 first confirm the important effect of the activity pointed out in the Paper: both the position of the $\sigma_v'-I_L$ curve and the sensitivity are directly related to the activity.

The Authors have suggested that the effect of activity on the $\sigma_v'-I_L$ curve may be taken into account by normalizing the liquidity index according to their equation (7). This procedure has been applied to the data in Fig. 1; as shown in Fig. 2, the use of a normalized liquidity index I_L'

Fig. 1. Undrained shear strength–liquidity index and effective stress–liquidity index relations for Grande Baleine Clay (after Locat & Lefebvre, 1982)

does bring the experimental relations into coincidence, suggesting a general validity of equation (7). However, as also shown in Fig. 2, the I_L' – σ_v' relation developed by the Authors from their data on clays others than Attapulgitic-rich clay (equation (8)) should be used with great caution, particularly in the low stress range where significant scatter of the experimental data is observed. As we shall see, part of this scatter may be due to the method of measuring w_L and I_L ; the use of fall-cone tests to measure w_L could help to reduce this scatter. Another factor to explain the scatter could be the method of preparation of the resedimented soil and in particular the sedimentation rate.

The data in Fig. 1 also raise serious questions about the Authors' analysis of their strength data. Fig. 1 demonstrates clearly the differences between the strength in the sedimented and

remoulded states. These differences occur most generally since it is seldom possible to observe a sensitivity of unity in sedimented clays. Consequently, when discussing the undrained shear strength of sedimented clays, a clear distinction should be made between sedimented and remoulded states. Unfortunately such a distinction is not made in the Paper. On the basis of the types of tests mentioned, it is clear that some of the strength data used in developing their Figs 5 and 6 and equations (11)–(15) refer to the sedimented state: triaxial tests cannot be carried out on remoulded material. However, other data such as those of Mitchell (1976) or from the

Table 1. Properties of Grande Baleine Clay

Natural water content	52.0%
Liquid limit	34.1%
Plastic limit	21.3%
Plasticity index	12.8%
Liquidity index	2.4
σ_p'	92 kPa
c_u	30 kPa
c_u remoulded (c_{ur})	0.2 kPa

Fig. 2. Relationships between σ_v' and the normalized liquidity index $I_L' = I_L(0.963 + 0.808A_c^{-1})$

Norwegian Geotechnical Institute definitely refer to clays in the remoulded state. Such confusion is unfortunate since it leads to the establishment of the highly questionable equation (15). Consistent with the stated objectives of the Paper, the undrained shear strength of interest is that of the sedimented clays. On the basis of the data in Fig. 1 as well as on other sedimented or normally consolidated clays, the Writers submit that this strength could be simply related to the vertical effective stress according to the classical $c_u/\sigma'_v = f(I_p)$ relations such as that proposed by Bjerrum (1973). The values of c_u/σ'_v of 0.17 for both specimens in Fig. 1 are consistent with such relations.

As for the remoulded shear strength, which is of little relevance in design, Casagrande (1932) stated that 'the shearing resistance of all soils at the liquid limit must have a constant value'. However, several investigations (Skempton & Northey, 1953; Youssef, El Ramli & El Demery, 1965; Sherwood & Riley, 1970; Leflaive, 1971; Karlsson, 1977; Le Bihan & Leroueil, 1981) have demonstrated that the results obtained using the Casagrande (1958) method do not meet this definition: moreover, the remoulded strength at the liquid limit measured in this manner decreases with increasing w_L . However, if the liquid limit is determined from fall-cone tests (w_{Lc}) following the standard methods developed in Sweden (Karlsson, 1977), England (British Standards Institution, 1975) or Quebec (Bureau de Normalisation du Québec, 1981), the liquid limit is systematically measured at a

constant remoulded shear strength of 1.6 kPa. Now, if the liquidity index I_{Lc} of any clay is obtained from such fall-cone measurements, then the scatter usually reported in $c_{ur}-I_{Lc}$ relationships (Houston & Mitchell, 1969) is almost eliminated. This has been established by Leroueil, Tavenas & Le Bihan (1983) on the basis of results from a very wide variety of natural clays: as shown in Fig. 3 a unique $I_{Lc}-c_{ur}$ relation with very little scatter can be defined, which is best represented by

$$c_{ur} \text{ (kPa)} = \frac{1}{(I_{Lc} - 0.21)^2}$$

The data in Fig. 1 fit this relation. Therefore, the introduction of a correction of either I_{Lc} or the $c_{ur}-I_{Lc}$ relation to account for the effect of activity is not justified.

REFERENCES

- Bjerrum, L. (1973). Problems of soil mechanics and construction on soft clays and structurally unstable soils (collapsible, expansive and others). *Proc. 8th Int. Conf. Soil Mech. Fdn Engng, Moscow* 3, state of the art report, session 4, pp. 111-160.
- British Standards Institution (1975). *Methods of test for soils for civil engineering purposes*. BS 1377. London: BSI.
- Bureau de Normalisation du Québec (1981). *Sols—détermination de la limite de liquidité à l'aide du pénétromètre à cône suédois et de la limite de plasticité*. BNQ 2501-092. Quebec: BNQ.
- Casagrande, A. (1932). Research on the Atterberg limits of soils. *Publ. Rds, Wash.* 13, No. 8, 121-136.
- Casagrande, A. (1958). Notes on the design of the liquid limit device. *Géotechnique* 8, No. 2, 84-91.
- Houston, W. N. & Mitchell, J. K. (1969). Property interrelationships in sensitive clays. *J. Soil Mech. Fdns Div. Am. Soc. Civ. Engrs* 95, SM4, 1037-1062.
- Karlsson, R. (in cooperation with the Laboratory Committee of the Swedish Geotechnical Society) (1977). *Consistency limits*. Document D6:1977.
- Le Bihan, J. P. & Leroueil, S. (1981). *The fall cone and the behaviour of remoulded clay*. Research Report. Montreal: Terratech Ltd.
- Leflaive, E. (1971). *Les limites d'Atterberg et le pénétromètre à cône*. Bulletin de Liaison des Laboratoires des Ponts et Chaussées No. 50, pp. 123-131.
- Leroueil, S., Tavenas, F., Brucy, F., La Rochelle, P. & Roy, M. (1979). Behaviour of destructured natural clays. *J. Soil Mech. Fdns Div. Am. Soc. Civ. Engrs* 105, GT6, 759-778.
- Leroueil, S., Tavenas, F. & Le Bihan, J. P. (1983). Propriétés caractéristiques des argiles de l'est du Canada. *Can. Geotech. J.* 20, No. 4, 681-705.
- Locat, J. (1982). *Contribution à l'étude de l'origine de la structuration des argiles sensibles de l'est du Canada*. PhD thesis, University of Sherbrooke.

Fig. 3. Relation between undrained shear strength of remoulded clays and liquidity index determined with the fall-cone (after Leroueil *et al.* 1983)

- Locat, J. & Lefebvre, G. (1982). The compressibility and sensitivity of an artificial sedimented clay soil: the Grande-Baleine clay, Quebec. *Proc. 2nd Can. Conf. Marine Geotech. Engng, Bedford*.
- Mitchell, J. K. (1976). *Fundamentals of soil behavior*. New York: Wiley.
- Sherwood, P. T. & Ryley, M. D. (1970). An investigation of a cone-penetrometer method for the determination of the liquid limit. *Géotechnique* **20**, No. 2, 203–208.
- Skempton, A. W. & Northey, R. D. (1953). The sensitivity of clays. *Géotechnique* **3**, No. 1, 30–53.
- Wroth, C. P. & Wood, D. M. (1978). The correlation of index properties with some basic engineering properties of soils. *Can. Geotech. J.* **15**, No. 2, 137–145.
- Youssef, M. S., El Ramli, A. H. & El Demery, M. (1965). Relationships between shear strength, consolidation, liquid limit and plastic limit for remoulded clays. *Proc. 6th Int. Conf. Soil Mech. Fdn Engng, Montreal* **1**, 126–129.

Mr M. D. Howat, Hong Kong Mass Transit Railway Corporation

The Cam Clay model which the Authors represent in their Fig. 1 by the equation

$$LI = 2.85 - 0.5 \log \sigma_v' \quad (\sigma_v' \text{ in pascals}) \quad (1a)$$

or

$$LI = 0.35 - 0.5 \log \sigma_v' \quad (\sigma_v' \text{ in bars}) \quad (1b)$$

is really an idealized critical state line, not a virgin consolidation line. Schofield (1980) warns that any data derived from it 'should be thought of as order-of-magnitude estimates' only. From the data given by Schofield & Wroth (1968), the critical state line of both London and Weald clays can be approximated to the equation

$$(LI)_{cs} = 0.21 - 0.28 \log \sigma_v' \quad (2)$$

The Authors postulate that the higher the activity of the clay, the lower the sensitivity after sedimentation from a slurry. Taking the argument to its limit, the Authors' equation for virgin consolidation of a clay with infinite activity

$$(LI)_R / 0.963 = 0.958 \sigma_v'^{-0.143} - 0.610 \quad (3)$$

might be considered as the equation for virgin consolidation of an insensitive clay. In the range $\sigma_v' = 0.5$ –2 bar, this equation approximates to within ± 0.04 bar to the simpler expression

$$(LI)_R = 0.335 - 0.28 \log \sigma_v' \quad (4)$$

which is parallel to equation (2) and may represent a generalized insensitive virgin consolidation line, at least for the English clays analysed by Schofield & Wroth (1968). Skempton &

Northey (1953) note a 'definite correlation between sensitivity and the liquidity index under any given pressure'. This correlation can now be expressed by

$$LI = 0.335 - 0.28 \log \sigma_v' + 0.6 \log S_t \quad (5)$$

which allows an estimate of the sensitivity of a clay to be made from only the Atterberg limits and water content (Fig. 4).

Combining the Authors' equations (8) for compressibility and (13) for shear strength, and applying them to the data for Weald and London clays given by Schofield & Wroth (1968), we obtain values of S_R/σ_v' of about 0.025 ($PI = 0.25$) and 0.04 ($PI = 0.52$) respectively, far lower than those computed using the relationship found by Skempton (1957)

$$\frac{c_u}{\sigma_v'} = 0.11 + 0.37 PI \quad (6)$$

It therefore seems likely that the clays analysed by Skempton (1957) are displaying a strength which cannot be explained by thixotropy or slurry-sedimented structure.

Lumb (1977) found that the c_u/σ_v' ratio in Hong Kong correlates, for a given plasticity, with sulphur content. Bearing in mind the presence of iron pyrites in many of the rocks from which the soils of Hong Kong are derived, it is possible that the correlation is due to cementation by ferrous oxides which accounts for the green tinge in some marine deposits. This cementation would be yet another factor explaining the occurrence of sensitivity in clays.

REFERENCES

- Lumb, P. (1977). The marine soils of Hong Kong and Macau. *Proc. Int. Symp. Soft Clay, Bangkok*, pp. 45–58.
- Schofield, A. N. (1980). Cambridge geotechnical centrifuge operations. *Géotechnique* **30**, No. 3, 227–268.
- Schofield, A. N. & Wroth, C. P. (1968). *Critical state soil mechanics*. London: McGraw-Hill.
- Skempton, A. W. (1957). Discussion on Planning and design of the new Hong Kong airport. *Proc. Instn Civ. Engrs* **7**, 305–307.
- Skempton, A. W. & Northey, R. D. (1953). The sensitivity of clays. *Géotechnique* **3**, No. 1, 30–53.

Authors' reply

With regard to compressibility, Leroueil *et al.* conclude that equation (8) should be used with caution, because of data scatter, and they have prepared Fig. 2 to support this point. The Authors agree that care must be exercised for clays with a plasticity index less than approximately

Fig. 4. Sensitivity chart

10–20, such as Grande Baleine Clay reported by the Writers. A small error in the plasticity values for such soils can result in a large error in the estimated compressibility. None the less, if the Writers' Fig. 2 is compared with Fig. 4 in the original Paper, it can be seen that the variation from equation (8) is no greater than for other clays. The apparent large variation shown in Fig. 2 is primarily due to the scale used by the Writers.

To assess the practical significance of this variation, the Authors have numerically analyzed the equilibrium consolidation of the Writers' sample A. The clay was assumed to be fully submerged and deposited at an initial liquidity index of 3, in a pond with an impervious bottom. The equilibrium consolidation is then obtained by integrating the compressibility curve shown in the Writers' Fig. 1. The results of this

analysis are presented in Fig. 5, in terms of dry unit weight versus depth. Also shown for comparison are the results using equation (8) and the Cam Clay model (extrapolated). It can be seen that the Authors' model agrees reasonably well, whereas the Cam Clay model is much poorer. For the same mass of clay, initially deposited approximately 3 m deep, equation (8) underestimates the average final density by slightly less than 7%. The Authors consider this to be reasonably accurate for a preliminary analysis. Furthermore, the error will decrease as the depth of the pond is increased and the compressibility curves converge. Note that for the case considered the Cam Clay model overestimates the average final density by 28%. This is because the Cam Clay model plots well below the compressibility curves shown in the Writers' Fig. 1.

The Writers' compressibility data deserve a more thorough analysis. This can be done when the following items are resolved.

- Equation (8) is based on the Casagrande method for liquid limit determination, but, presumably, a fall-cone penetrometer was used by the Writers for the Grande Baleine Clay. If the liquid limit were measured with an 'old-style' Casagrande cup, how would the compressibility curves shift? As discussed later, the plasticity index of sample A would probably decrease, and the comparison with equation (8) would improve.
- Were the slurries of samples A and B initially prepared at a liquidity index greater than 3? The Writers' Fig. 1 is not clear on this point. Equation (8) is only applicable when this condition is met.
- Presumably, the Atterberg limits of samples A and B were different, owing to the different porewater salinities. How do they compare with the values reported by the Writers for the natural clay? (The Authors assumed that the low salinity clay, sample A, would have the same values as the natural clay in the analysis presented in Fig. 5.)

With regard to undrained shear strength, Leroueil *et al.* point out that there is a difference between sedimented and remoulded conditions. The Authors concur. It was our intention to refer to only remoulded shear strength, which we attempted to make clear in the footnote on p. 217 of the original Paper. All the data that were employed to develop equation (15) are based on remoulded samples.

We chose to limit the original Paper to remoulded strength and to defer sedimented strength to a future paper, because the latter is not a unique value, contrary to the suggestion made by Leroueil *et al.* Rather, it is related to the age of the deposit, among other factors (see Bjerrum, 1973). Furthermore, immediately after deposition, the sedimented strength is essentially equal to the remoulded strength (see Elder, 1984), and thus the sensitivity of a fresh deposit is equal to unity. Hence, in the Authors' practical applications, the remoulded strength is a critical design parameter.

The Authors recognize that by redefining the liquid limit to be a single remoulded shear strength value the variation regarding liquidity index versus shear strength can be reduced. However, we have been reluctant to make that leap for two reasons. Firstly, at the liquid limit, according to the Casagrande method, the shear strength varies by an order of magnitude (see Fig. 5 in the original Paper): it is difficult to

Fig. 5. Comparison of equilibrium consolidation

justify the choice of an average value. Other investigators (see Kenney, 1963) have concluded that the Casagrande liquid limit is not a measure of strength, and that the controlling soil properties are not fully understood. If the suggestion of Leroueil *et al.* to use a constant shear strength value of 1600 Pa is adopted, then, by combining equations (12) and (13), it can be seen that this would correspond to a clay with an activity of approximately 0.44. For clays with an activity less than 0.44, the new definition would result in a higher plasticity index and activity, and for clays with an activity greater than 0.44 the new definition would result in a lower plasticity index and activity. The appropriate expressions are given by

$$(PI)_{\text{new}} = (PI)_{\text{old}} [0.648 + 0.156(ACT)_{\text{old}}^{-1}] \quad (7a)$$

$$(ACT)_{\text{new}} = (ACT)_{\text{old}} \frac{(PI)_{\text{new}}}{(PI)_{\text{old}}} = 0.648(ACT)_{\text{old}} + 0.156 \quad (7b)$$

For example, a clay with a Casagrande plasticity index of 20 and an activity of 0.2 would have a new plasticity index of about 29 and a new activity of 0.29, or, with an old plasticity index

of 100 and $ACT = 1.0$, the new plasticity index would be 80 and the new activity would be 0.80. Hence, the redefinition reduces the ability to discriminate between properties. Conversely, the above relationships can also be expressed as

$$(PI)_{old} = (PI)_{new} [1.53 - 0.241(ACT)_{new}^{-1}] \quad (8a)$$

$$(ACT)_{old} = 1.543(ACT)_{new} - 0.241 \quad (8b)$$

whence, sample A of Leroueil *et al.* would have a Casagrande plasticity index of about 6 and an activity of 0.10. If nothing else, a number of correlations that are used routinely by the profession would have to be reformulated in terms of the new plasticity index.

Secondly, even if a single shear strength value were used for the liquid limit, there would still be a large variation regarding liquidity index versus effective stress (see Fig. 1 in the original Paper). Compressibility and shear strength are intrinsically related and thus it is presumed that, if activity influences one property, then it must also affect the other. Compare the similarity of the empirical coefficients in equations (9) and (10) for compressibility and shear strength respectively

$$LI' = 0.953 \left(\frac{\bar{\sigma}}{p_{atm}} \right)^{-0.143} - 0.610 \quad (9)$$

$$LI'' = 0.916 \left(\frac{S_R}{p_{atm}} \right)^{-0.158} - 0.899 \quad (10)$$

It is important to continue to gather data on the fundamental properties of clays. In that regard, the contribution by Leroueil *et al.* (1983), and reproduced in Fig. 3 of their Discussion, is very valuable. It would be further enhanced if a comparison could be made based on the Casagrande method.

The interpretations made by Howat are very

interesting. The Authors are in philosophical agreement that sensitivity can be estimated on the basis of the Atterberg limits and water content: note that Howat's Fig. 4 is similar to Fig. 12.9 in Mitchell (1976). However, we feel that the age of the deposit, plus the effect of the porewater chemistry, must also be considered. Recently, it has been shown that even changes in the porewater chemistry that occur after sampling must be taken into account (see Lessard & Mitchell, 1984, and Mitchell, 1984).

Understanding the geologic context of natural deposits will aid in unravelling these relationships. For example, the paper by Lumb (1977) shows the effect of sulphur on the peak undrained shear strength of tropical marine soils.

REFERENCES

- Bjerrum, L. (1973). Problems of soil mechanics and construction on soft clays and structurally unstable soils (collapsible, expansive and others). *Proc. 8th Int. Conf. Soil Mech. Fdn Engng, Moscow* **3**, state of the art report, session 4, pp. 111–160.
- Elder, D. McG. (1984). Private communication; thesis, Oxford University, to be published.
- Kenney, T. C. (1963). Correspondence on Atterberg limits. *Géotechnique* **13**, No. 2, 159–162.
- Leroueil, S., Tavenas F. & Le Bihan, J. P. (1983). Propriétés caractéristiques des argiles de l'est du Canada. *Can. Geotech. J.* **20**, No. 4, 681–705.
- Lessard, G. & Mitchell, J. K. (1984). The causes and effects of aging in quick clays. Submitted to *Can. Geotech. J.*
- Lumb, P. (1977). The marine soils of Hong Kong and Macau. *Proc. Int. Symp. Soft Clay, Bangkok*, pp. 45–58.
- Mitchell, J. K. (1976). *Fundamentals of soil behavior*. New York: Wiley.
- Mitchell, J. K. (1984). Practical problems from surprising soil behavior. 20th Terzaghi Lecture, American Society of Civil Engineers, San Francisco. Submitted to *J. Geotech. Engng Div. Am. Soc. Civ. Engrs.*