Znidarčić, D., Schiffman, R. L., Pane, V., Croce, P., Ko, H.-Y. & Olsen, H. W. (1987). *Géotechnique* **37**, No. 4, 519–522

The theory of one-dimensional consolidation of saturated clays: part V, constant rate of deformation testing and analysis

D. ZNIDARČIĆ, R. L. SCHIFFMAN, V. PANE, P. CROCE, H.-Y. KO & H. W. OLSEN (1986). *Géotechnique* **36**, No. 2, 227–237

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The Authors presented an analysis for constant rate of deformation testing. The correspondence between the simulated and the actual test data on four different soils were reported to be good. On the basis of the findings, the Authors concluded that constant rate of deformation testing could be used to evaluate the consolidation behaviour of soft soils.

The Writers have studied the laboratory consolidation behaviour of two soils, kaolin NF and Dundas soil (Table 1), using conventional, constant rate of strain (CRS) and isotropic consolidation tests (Chakrabarti & Horvath, 1985a, b; Horvath & Chakrabarti, 1985, 1986). A fixed ring consolidometer was used for the one-dimensional consolidation testing on soil samples 50·70 mm in diameter and 18·29 mm in height. The CRS tests were carried out at four different strain rates ranging from $0{\cdot}4444 \times 10^{-3}$%/min to $13{\cdot}1219 \times 10^{-3}$%/min. The isotropic consolidation tests were performed in triaxial cells using cylindrical samples 38·00 mm in diameter and 76·00 mm in height.

The findings relevant to this discussion are summarized as follows.

Strain rate

For the one-dimensional tests, the value of the compression index C_c in the e–log σ_v' space was independent of the type of testing and the rate of strain. However, the location of the one-dimensional virgin compression line for the isotropically prepared kaolin samples was found to shift to the left-hand side as the rate of strain was reduced. This strain-rate-dependent behaviour has also been observed by Byrne (1972) and Crawford (1964).

Since the value of C_c was not affected by the strain rate, the locations of the virgin compression lines for various strain rates were compared using the value of e_n which was defined as the void ratio of the normally consolidated soil at the effective vertical stress of 1·0 kPa. From Fig. 1 a linear relation exists between e_n and the logarithm of strain rate. Byrne's (1972) data similarly treated gave an identical relationship for peat.

Information concerning the average strain rates observed in conventional consolidation tests and strain rates given by Crawford (1965), Gorman, Hopkins, Deen & Drnevich (1978) and Lee (1981) are included in Fig. 1 for comparison.

Consolidation lines

The study indicated that the one-dimensional virgin consolidation line and the isotropic normal consolidation line may not necessarily be parallel (Chakrabarti & Horvath, 1985b). A comparison of the results with those published suggests that the ratio of the slopes of these two lines depends primarily on the type and the amount of clay in the soil (Yudhbir, Mathur & Kuganathan, 1978).

The Authors reported liquid limit LL and plastic limit PL values for the three soils. Other physical data, particularly the clay content, were not given. Complete data concerning the physical properties of the soils tested should always be provided so that the soils can easily be characterized and classified. This characterization helps other researchers to compare their data.

The authors used four different deformation rates for the four soils. Several recommendations regarding the strain rate that is appropriate for a particular soil have been published (Crawford, 1964, 1965; Gorman *et al.*, 1978; Lee, 1981; Smith & Wahls, 1969; Wissa *et al.*, 1971). The deformation rates used by the Authors appear to have been chosen arbitrarily. The sample of Pacific illite ($e_0 = 2{\cdot}90$) with an LL value of 120 was tested at $2{\cdot}1 \times 10^{-7}$ m/s while the sample of Georgia kaolin ($e_0 = 2{\cdot}93$) with an LL value of 44 was tested at $1{\cdot}26 \times 10^{-7}$ m/s.

Table 1. Physical properties of soils*

Property	Soil		Percentage finer	
	Kaolin NF	Dundas soil	Kaolin NF	Sundas soil
Specific gravity	2·60	2.75		
Liquid limit LL	74	23		
Plastic limit PL	42	16		
Activity	0·40	0·44		
Grain size distribution†				
150 μm			100	100
75 μm			100	91
40 μm			100	86
10 μm			100	60
4 μm			96	36
3 μm			92	28
2 μm (clay size)			81	16
1 μm			55	3
0·5 μm			28	—

* After Chakrabarti & Horvath (1985a).
† Using the hydrometer method.

The Authors tested soil samples which had very high void contents. The sample of phosphatic clay had a void ratio of 22·83. The initial moisture contents of the Authors' soil samples ranged between 100% and 1000% (approximately). The Writers would like to know how the Authors had taken care of the following.

(*a*) Testing such soils in the consolidometer used by the Authors would be quite a difficult task. When loaded, the soil slurry would always try to escape through the annular space, however small, in between the top porous disc and the consolidation ring.

(*b*) The vertical load and the corresponding pore pressure readings for such soil slurries would be extremely small during the initial stages of testing and might even be smaller than the sensitivity of the conventional load cells and pressure transducers used in practice.

The Authors' data for Florida phosphatic clay (fig. 3) support these observations. The Authors measured and reported vertical stresses slightly

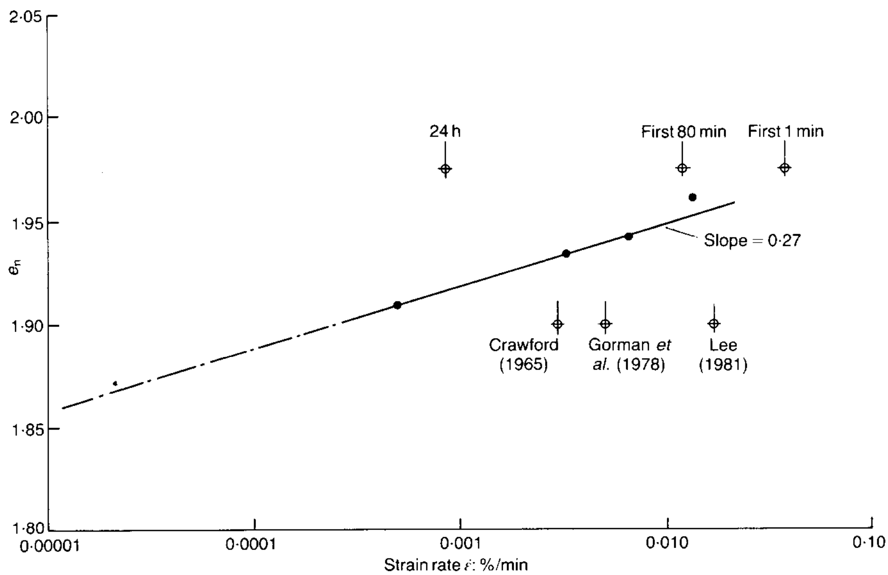

Fig. 1. e_n versus strain rate for isotropically prepared kaolin (the range of strain rates is for conventional consolidation tests)

greater than 0·1 kPa and excess porewater pressures less than 0·1 kPa during the initial stages of testing. The maximum reported values of the vertical stress and the porewater pressures were around 100 kPa.

As pointed out earlier, the Authors did not give the clay contents of their soils. An examination of the reported LL values indicates that the soils had moderate to high clay contents. Previous studies by the Writers (Chakrabarti & Horvath, 1985a; Horvath & Chakrabarti, 1986) as well as studies by other researchers (Byrne, 1972; Crawford, 1964, 1965; Smith & Wahls, 1969) indicate that the soils used by the Authors might exhibit strain-rate-dependent behaviour as discussed earlier. The Authors, however, have neglected the strain rate effects on the laboratory consolidation behaviour of their soils.

Regarding the Authors' evaluation of consolidation behaviour of soft soils in the laboratory

(*a*) the strain rates used by the Authors were rather high compared with average field strain rates (Crawford, 1965): how did the Authors intend to use their data to predict the field consolidation behaviour?

(*b*) it is generally recognized that for any particular soil the method of test and the boundary conditions would greatly affect the laboratory consolidation behaviour (Crawford, 1964, 1965; Hanrahan, 1977; Yudhbir *et al.*, 1978): how would the Authors predict the field behaviour using the constant rate of deformation test data when the constant rate of loading prevails for most conventional constructions (Aboshi, Yoshikumi & Maruyama, 1970)?

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Authors' reply

The intention of the Paper was to present a new analysis method for the constant rate of deformation test. The justification for using this test to evaluate the consolidation characteristics of a soil is given in the Paper as well as in an earlier paper in the series (Znidarčić, Croce, Pane, Ko, Olsen & Schiffman, 1984). It was not the intention of the Paper to present data on consolidation characteristics of various soils. It is for this reason that only the essential soil characteristics are stated. The Authors wish to discourage anybody from using the presented data as typical for the given material. All the specimens tested were completely remoulded and prepared in the laboratory. Had the samples been prepared differently the results would be quite different. The test data were presented only to demonstrate the range of materials and conditions for which the new analysis is applicable and to compare the results of the new analysis with the independent determination of consolidation characteristics on identical samples. Any other use and interpretation of the data presented are inappropriate.

The Writers raised the question of the appropriate deformation (or strain) rate in a constant rate of deformation test and its influence on the material characteristics determined. There are two sides to the issue. One side is the influence of the deformation rate on the test results when the soil tested exhibits significant creep behaviour. This issue has not been resolved yet and comprehensive research on the subject is needed. The analysis presented was based on the finite consolidation theory developed by Gibson, England & Hussey (1967) which does not account for secondary compression. Thus, the tests performed on a material exhibiting significant creep behaviour should not be analysed using the procedure described in the Paper. The other side of the issue regarding the deformation rate is related to the permeability of the soil tested and to the development of excess pore pressure within the sample. This aspect is extensively addressed in the Paper when the simulated tests are analysed. It is shown to what extent the increase in the test velocity affects the results obtained. On the basis of the analysis it is recommended that the tests be performed at a velocity for which the induced pore pressure within the specimen will not exceed 30–50% of the applied load. An identical criterion is suggested by Gorman, Hopkins, Deen & Drnevich (1978). The excess pore pressure depends on the material permeability, sample height and imposed test velocity. Furthermore, the criterion regarding the maximum allowable excess pore pressure within the sample is related to the assumptions made in the test analysis. If an unrestrictive analysis is developed which properly takes into account the variation in the effective stresses within the sample any test velocity would be appropriate and the results obtained would not be affected by the magnitude of the excess pore pressure generated within the sample. The restrictive assumption in the analysis presented is that the g(*e*) is a piecewise constant function. As stated in the Paper, additional work is being done to eliminate this restriction.

The test velocities reported were chosen on the basis of the excess pore pressure criterion. Realizing that a proper test velocity depends on many different factors it is not appropriate to relate it only to the plasticity characteristics of the soil.

The Authors agree that testing soil samples at very high void ratios is a difficult and challenging task. The constant rate of deformation consolidometer described is designed to overcome the problem of squeezing the soil slurry between the top porous disc and the consolidation ring. In the description of the device a careful reader will find that the porous disc is mounted with a Teflon scraper ring which effectively seals the gap.

The nominal sensitivity of the pore pressure and total stress transducers used in the consolidometer is 0·01 kPa. After using the device to perform several tests it was realized that it was difficult to obtain reliable results for stresses below 0·1 kPa. It is possible to improve the sensitivity by using differential transducers and by performing tests in a temperature- and pressure-controlled environment. However, on the basis of the experience gathered a different technology should be developed for testing soft materials when consolidation characteristics for stress levels below 1·0 kPa are sought. Any new approach should avoid the need for compressing a soft soil with a rigid piston. The seepage-induced consolidation test provides an appealing alternative. Research is in progress to develop such a test and the corresponding analysis procedure.

Regarding the use of the laboratory-generated data for solving field problems two different approaches can be used. In the first approach, the laboratory tests are viewed as physical models of field conditions. The test results are then extrapolated to the field through empirical or semi-empirical relations. Certainly in this case it is imperative to duplicate the field condition in the laboratory to the maximum extent possible. In the second approach, however, the link between the laboratory and the field is made by a rational theory which takes into account any differences in the conditions between these two entities. The role of the laboratory test in this case is to provide information regarding the constitutive relations of the material. The test presented in the Paper is an integral part of the rational theory of consolidation developed by Gibson *et al.* (1967) and the field analysis is performed using the theory and material characteristics from the test.

It is clear from the discussion that the Writers consider the laboratory tests as a physical model of field conditions and they propose a semi-empirical approach for extrapolating the laboratory results to the field. Although the empiricism is essential and still necessary in solving geotechnical problems any new development should replace the empiricism with rational solutions whenever possible. For solving consolidation problems in which the one-dimensional condition prevails and when the material does not exhibit significant creep effects the non-linear finite strain consolidation theory provides a rational solution in which empiricism is not necessary.

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