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Development of structure in sedimenting soils

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THE ROYAL SOCIETY

Development of structure in sedimenting soils

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At very low densities, a cohesive soil slurry behaves as a dense fluid, with the particles supported by the pore water, which has a pressure equal to the total vertical stress. As the soil settles under gravity, the particles begin to interact and the pore water pressure drops. At this stage, as non-zero effective stresses now exist, the behaviour has changed from that of a fluid to that of a consolidating soil, with an associated structure. This paper explores this development of effective stress and structure, and presents evidence of the occurrence of creep in addition to the process of densification by consolidation.

Keywords: effective stress; pore-water pressure; consolidation; creep; suspension; void ratio

1. Introduction

Soil is a granular material with a wide range of behaviour, from cohesionless sands to cohesive clays, with volume change behaviour that can be contractive or dilatant, and with permeabilities that lie between limits which are many orders of magnitudes apart. For clays, the behaviour is strongly influenced by the net negative charges on the particles, giving rise to electrochemical forces that influence flocculation. For sands, the behaviour is typically frictional. As well as there being differences in sand and clay behaviour, there are similarities, with the stress–strain relationships affected by past history and by the present state of the soil, allowing parallels to be drawn between the behaviour of loose sands and soft clays, and between dense sands and stiff overconsolidated clays.

Soil can behave like a fluid or like a solid, depending on the density and the confining stresses. This paper examines the transition of clays and silty clays from the state of a dense fluid, in which the pore-fluid pressures are equal to the vertical total stress, to that of a very soft soil, defined by the existence of effective stress, corresponding to pore-fluid pressures that are less than the total vertical stress.

2. Experimental details

The experiments reported in this paper have been carried out at Oxford University in acrylic settling columns of internal diameter 100 mm. Sediment has been introduced as a well-mixed slurry of known density, in a single operation, with initial depths in the range 0.4–1.6 m. The key to the research has been the ability to produce accurate non-destructive density measurements throughout the subsequent settling and consolidation process, using an X-ray system developed by Been (1980), which is described below and in Been & Sills (1981) and Sills (1997). A highly collimated beam of X-rays is directed through the column to a sodium iodide crystal and photomultiplier assembly, producing a count rate N which can be related to the density γ through the equation

$$N = N_0 \mathrm{e}^{-k\gamma}.$$

The X-ray beam is moved up and down the settling column, generating a profile of count rate against height. The parameters N_0 and k can be calculated from two independent correlations relating count rate to known densities, so that the count-rate profile can be converted to a density profile. The calibrations can be obtained in two ways. The first is to use separate calibration samples, which should be made up with the same configuration of acrylic cell and soil as in the experiment, since k depends on atomic number. The second approach is to use the count-rate profile from the experiment itself, taking the water calibration from the water overlying the sediment bed, and the second calibration equation from the condition that the total stress at the base of the column is equal to the integrated area beneath the density profile. The accuracy of density calculation depends on the traverse speed of the X-ray, with a typical rate of 1 mm s^{-1} providing an accuracy of density measurement of the order of ± 0.002 Mg m^{-3}, with a spatial resolution of *ca.* 1 mm. The arrangement is shown schematically in figure 1. Further details of experimental conditions and of constraints such as wall friction are presented and discussed in Been & Sills (1981) and Sills (1997).

Pore-water pressures play a significant role in the process of soil settling and consolidation, so their measurement is as important as that of density. In a soil that flocculates, a distinction can be made between voids within flocs and those between them, giving rise to the concept of intra- and inter-floc pore pressures. Any measurement of pore-water pressures will be of inter-floc pressures, since there exists continuity of water through the large pores. Measurement of intra-floc pressure would require very precise placing of a very small pressure transducer within the floc itself, which would be difficult, if not impossible, to achieve. The inter-floc pressure is measured by inserting porous plastic Vyon filters in the column walls, and connecting the water in the soil through these filters via water-filled tubes (made of low compressibility plastic) to a pressure transducer. The single transducer is connected in turn to each of the ports and to a calibration reservoir, using a unit developed by Bowden (1988). Typical measurement accuracies are of the order of ± 0.01 kPa, or 1 mm head of water. The pore pressure ports are indicated schematically in figure 1.

3. Experimental results

Results will be reported from experiments on several different soils, characterized in table 1.

(*a*) *Settling behaviour*

The typical surface settlement rate of a suspension of cohesive particles was described by Kynch (1952). Subsequently, Been & Sills (1981) reported a series of experiments in which a sediment suspension was allowed to settle under its own weight. A similar experiment has been carried out more recently (Alves 1992) on a suspension of red mud from Brazil. This mud is produced by the processing of bauxite ores and contains a high percentage of iron oxide with some aluminium oxide. It

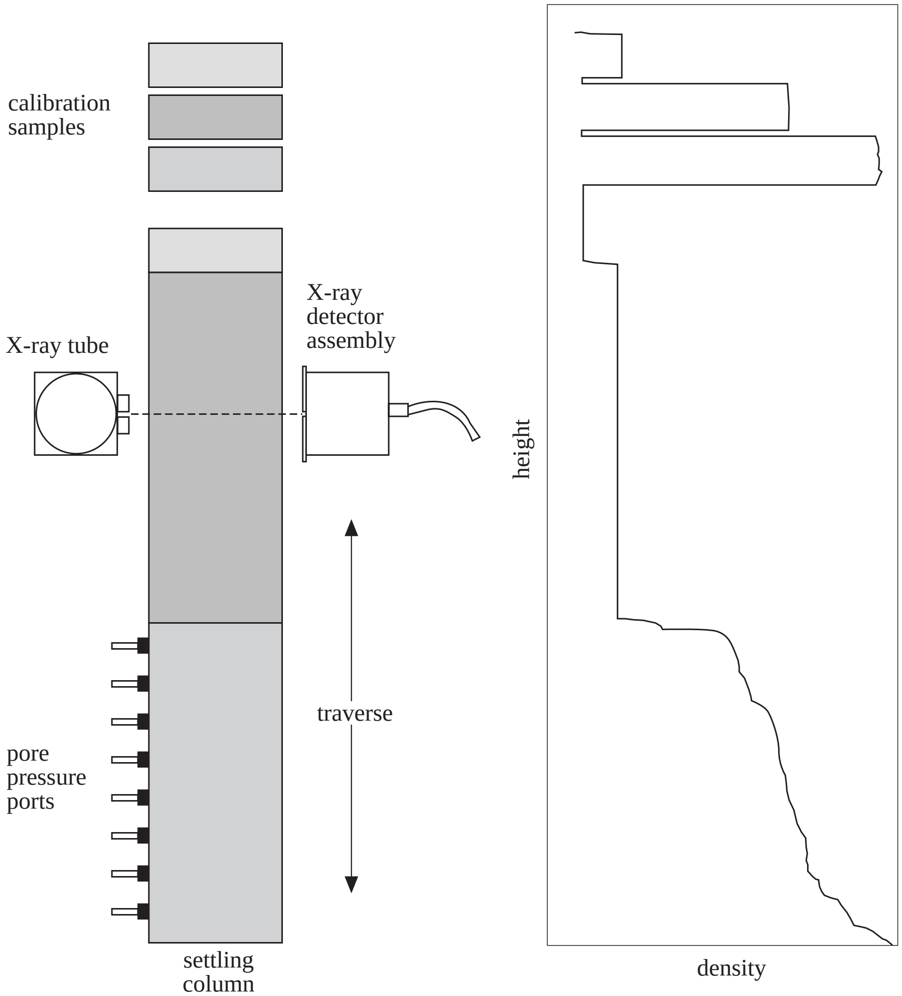

Figure 1. Schematic diagram of settling column including X-ray system for density measurement.

Table 1. *Specification of soils used in experiments*

	location	initial height (m)	initial density (Mg m^{-3})	particle size (% clay, silt, sand)	liquid limit	plastic limit	specific gravity
CTP4	Ketelmeer	0.618	1.191	35, 65, 0	0.90	0.42	2.51
KC2	Ketelmeer	0.600	1.186	35, 65, 0	0.86	0.46	2.45
NK2	Slufter	0.600	1.178	48, 44, 8	0.86	0.46	2.45
REDM05	Brazil, bauxite mining waste	0.989	1.068	28, 47, 25	0.60	0.36	3.47
DME10	Combwich	1.553	1.147	40, 60, 0	0.62	0.30	2.66

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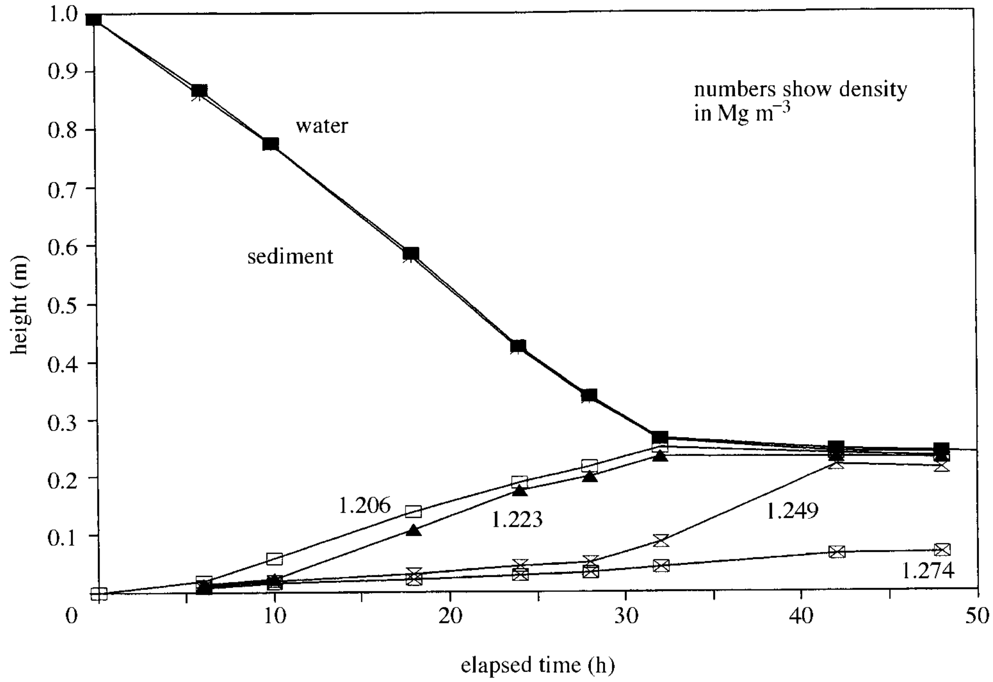

Figure 2. Surface settlement and upward development of density in experiment REDM05.

has a well-graded particle-size distribution of fine sand, silt and clay. In experiment REDM05, the initial density was 1.06 Mg m^{-3} and initial height 0.989 m. Figure 2 is a plot of the surface elevation against elapsed time, and also shows the build-up of densities in the consolidating bed. The surface settlement curve demonstrates the typical pattern described by Kynch of a steep initial slope associated with the presence of the initial suspension, followed by a shallower settlement of the consolidating soil bed. The numbers on the other curves demonstrate the development of density inside the bed, increasing upwards from the bottom of the settling column.

(*b*) *The consolidation process*

In figure 3, a series of density profiles is presented for the experiment REDM05 (Alves 1992). Figures 2 and 3 both show that the surface of the slurry settled quickly during the first 32 h, leaving clear water above. This stage was characterized by the presence of two density steps, one marking the transition between overlying water and the mud beneath, and the other marking the development of a denser layer upwards from the base of the settling column. Once this denser layer met the mud surface, the surface settlement was much slower, and the density gradually increased throughout the bed.

Figure 4 shows excess-pore-pressure profiles for the same experiment, calculated by subtracting the hydrostatic pressure from the measured pore-water pressure. The 1 h profile corresponds to the condition in which the pore pressures were equal to the total vertical stress throughout the slurry, and the weight of the sediment was totally carried by the fluid. By the time of the 18 h profile, the density had increased over the bottom 0.2 m of the column (as shown in figure 3), and the slope of the excess-pore-

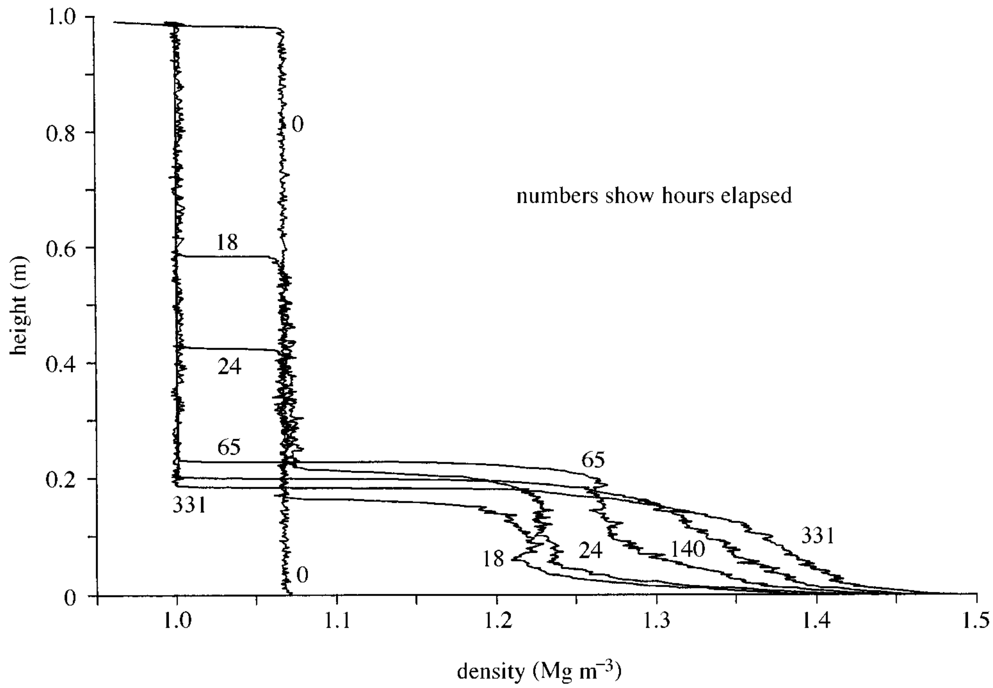

Figure 3. Density profiles in experiment REDM05.

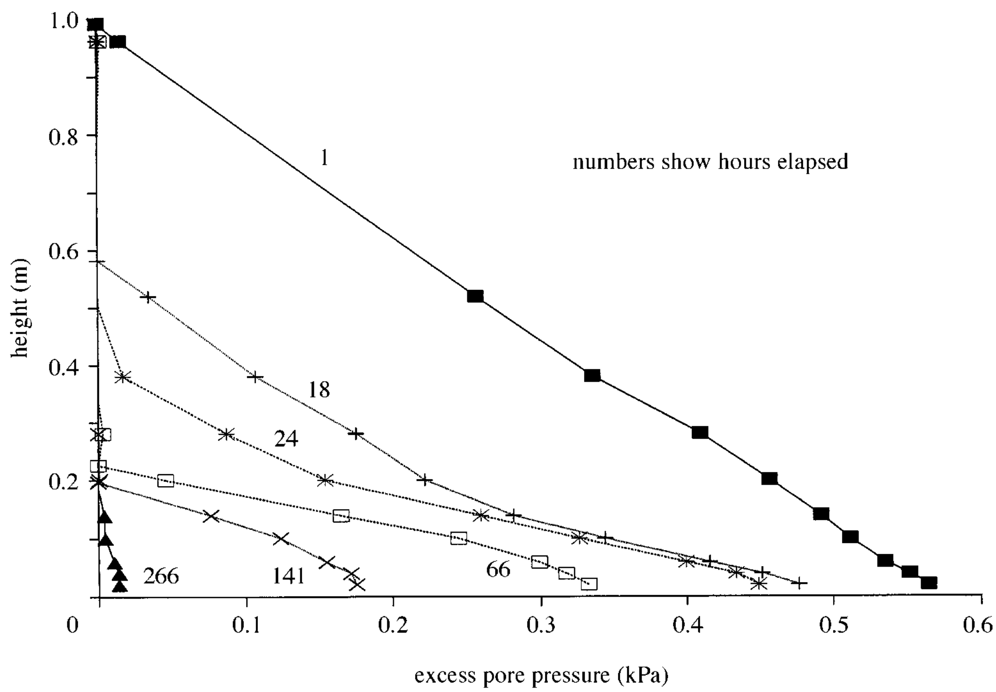

Figure 4. Excess-pore-pressure profiles in experiment REDM05.

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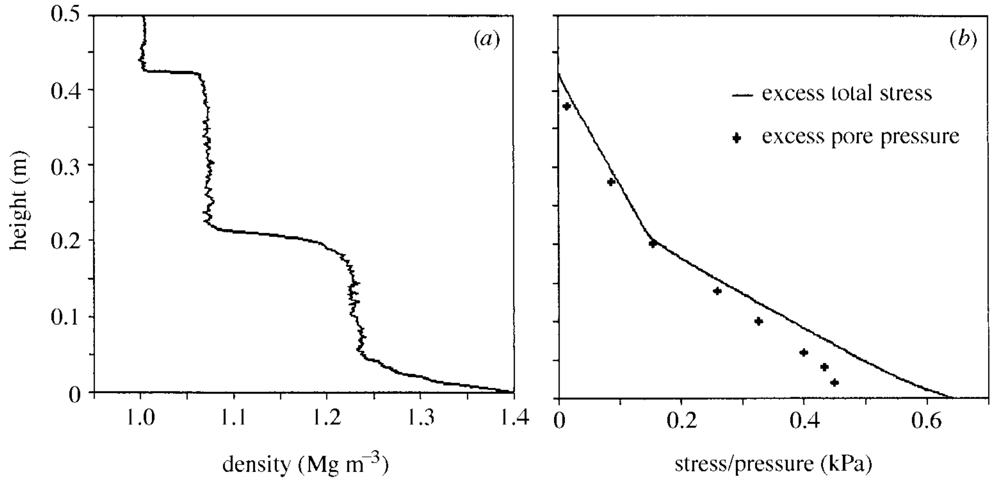

Figure 5. Calculation of effective stress as difference between excess total stress and excess pore pressure for experiment REDM05.

pressure profile reduced as a supporting framework developed. Over the upper part, where the density of the slurry was virtually unchanged from the input value, the 18 h profile has the same slope as the initial profile. Above the mud surface at 0.6 m there was clear water, so that the excess pore pressure was necessarily zero. Once the density was everywhere greater than the initial value, as observed in the density profiles of 66 h and later, the excess-pore-pressure profiles were broadly parabolic.

The behaviour of the early stages of the settling process can be examined in more detail by considering the 24 h density profile shown in figure 5*a*. This can be integrated to produce a vertical total-stress profile (neglecting the deceleration of the soil particles since this has an insignificant effect). The vertical stress is compared with the corresponding pore pressure in figure 5*b*, in which each of these profiles is presented as excess values, or values above hydrostatic pressure. Where the density is close to its initial value, it can be seen that the excess pore pressures and excess total stress are very close in magnitude, while in the denser part of the bed the pore pressures have dropped significantly below the total stress. Thus, this figure demonstrates the transfer from a fluid-supported suspension, where total vertical stress, σ_{v}, and pore-water pressure, u_{w}, are equal, to a soil in which effective stresses, defined as the difference between total stress and pore pressure, $\sigma' = \sigma - u_{\mathrm{w}}$, exist. Such a pattern of behaviour was noted initially by Been (1980) and Been & Sills (1981), in similar experiments carried out on Combwich mud, a natural estuarine silty clay.

The concept of effective stress has been a cornerstone in the analysis of soil behaviour, based on the assertion by Terzaghi (1936) that the effective stress uniquely determines soil strains and strength. The experimental results from the settling column have made it possible to study the development of effective stress. It can be seen in figure 5*a* that the density at the top of the mud layer, in which the pore pressures are lower than the total stresses, is around 1.20 Mg m^{-3}, and that effective stresses and the corresponding soil structure are associated with densities higher than this. This density is therefore termed the structural density (Sills 1995, 1997). Figure 2 showed that the density line marked 1.206 Mg m^{-3} started from the base of the column at the beginning of the experiment, and then met the downward-moving

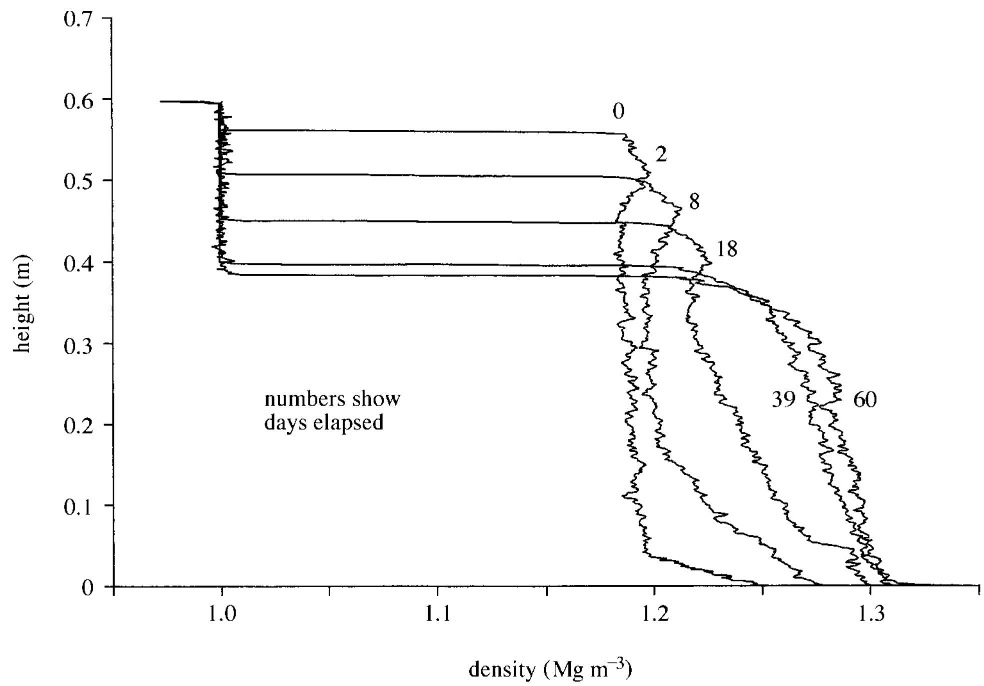

Figure 6. Density profiles in experiment NK2.

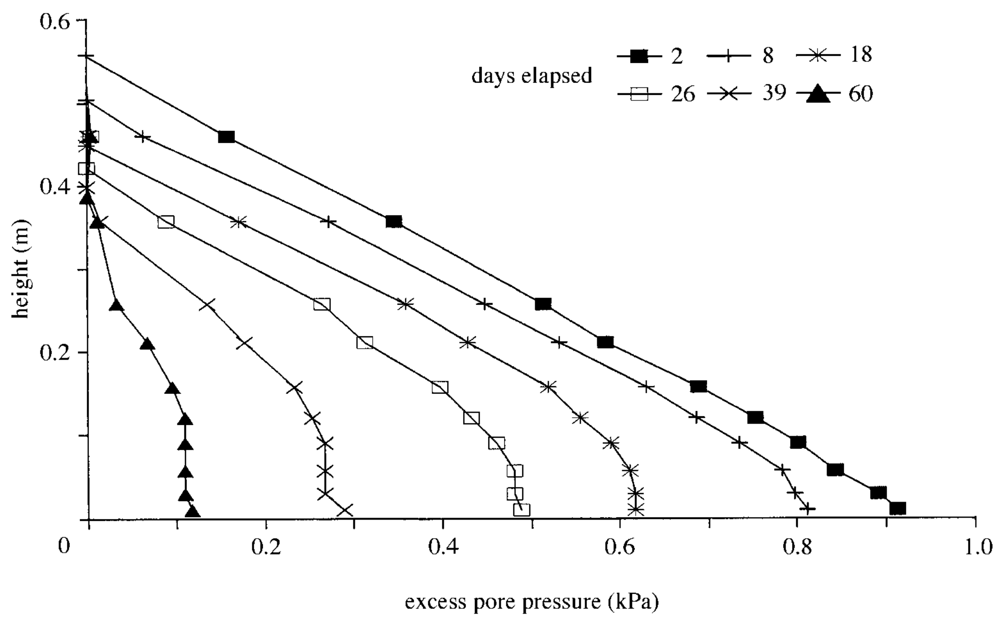

Figure 7. Excess-pore-pressure profiles in experiment NK2.

suspension surface at the time when the surface settlement rate slows down dramatically. This is further evidence that in this experiment, this density marks the transition from suspension to soil.

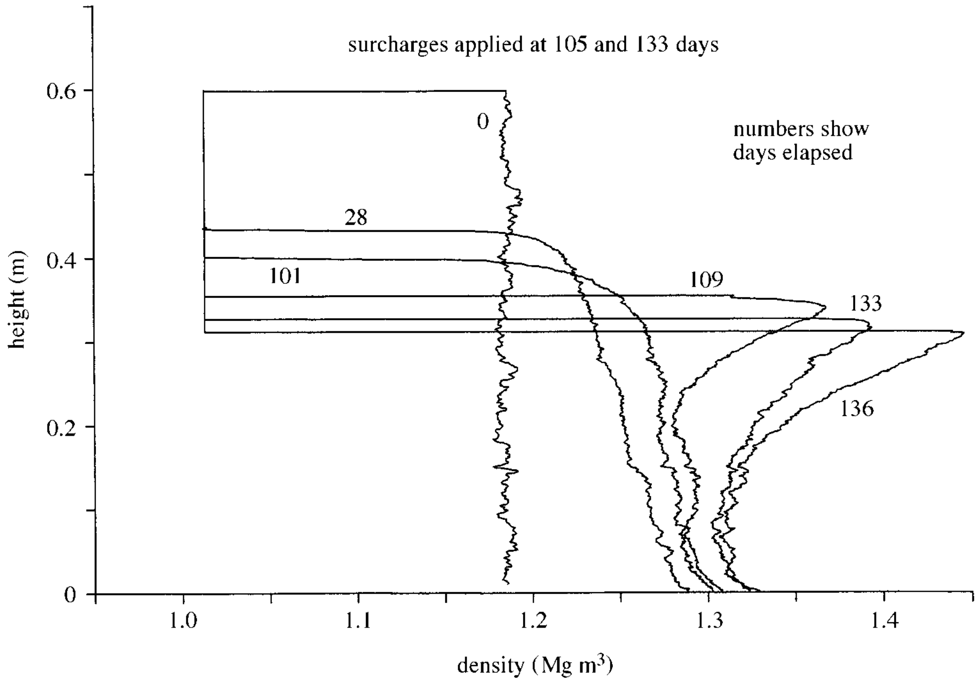

Figure 8. Density profiles in experiment KC2.

Figure 6 shows density profiles for an experiment, NK2, on a mud dredged from the Ketelmeer in The Netherlands. The particle-size distribution was 35% clay, 41% medium to fine silt and the rest coarse silt. In this experiment, the initial slurry density was 1.178 Mg m^{-3}, and the initial height was 0.600 m. It can be seen that the density increased steadily from the bottom upwards without the density steps that developed in the less dense slurry shown in figures 3 and 5*a*. This pattern is characteristic of a slurry with an initial density greater than the structural density of the soil. Figure 7 shows the corresponding excess pore pressures. The maximum vertical excess total stress was a little greater than 1 kPa, and small but measurable effective stresses existed very soon after the start of the experiment.

The maximum effective stress that can be achieved in the settling columns under self-weight consolidation is determined by the initial density and the height of the slurry. For example, the upper limit for a 2 m deep sediment bed of initial density 1.17 Mg m^{-3} is around 3 kPa. This corresponds to a very soft soil indeed, so that to produce stiffer soils under higher effective stresses, typical of field conditions, further loading is required. Figure 8 shows the density profiles for experiment KC2, which also used the silty sediment from Ketelmeer in The Netherlands, with initial density 1.186 Mg m^{-3}, in which a surcharge load was applied, with the immediate effect of increasing the excess pore pressure uniformly throughout the bed, except right at the surface, which acted as a drainage boundary with zero excess pore pressure. The effective stresses therefore increased first at this level, causing a high-density surface layer which moved downwards as consolidation proceeded.

An alternative method of increasing the effective-stress range above the self-weight stress level, is to use a hydraulic gradient, in which water is drained from the base

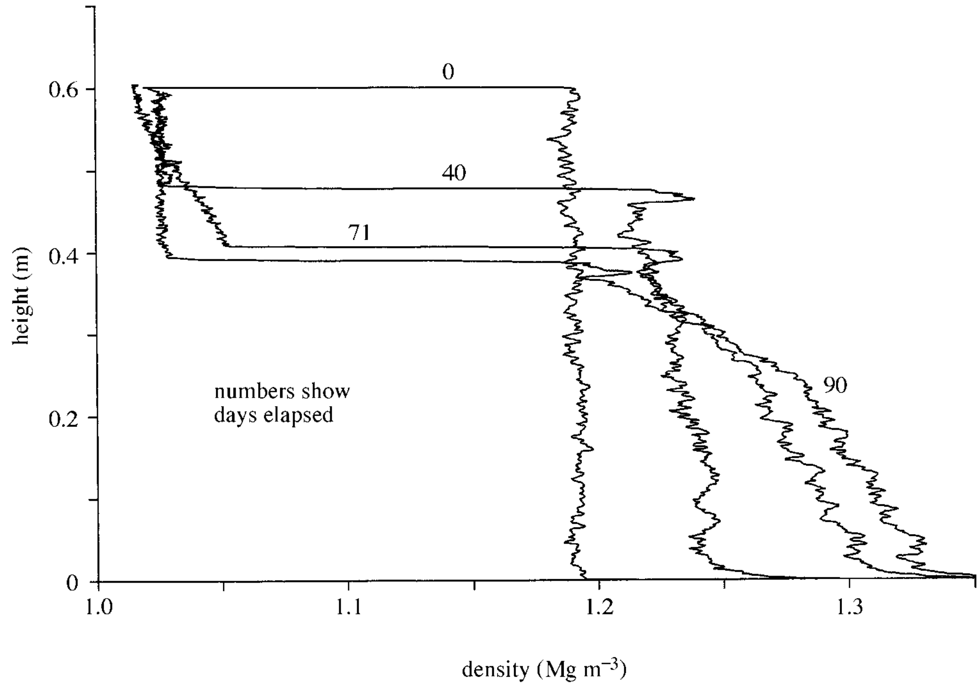

Figure 9. Density profiles in experiment CTP4.

of the settling column. Various methods are possible, including a Mariotte bottle to maintain a constant head of water above the consolidating sediment bed and the use of a peristaltic pump to return the drained water back into the column. Figure 9 shows a selection of density profiles from experiment CTP4 on a fine-grained sediment containing 48% clay sizes, dredged from the harbour at Rotterdam and deposited in the Slufter, a 30 m deep storage site in The Netherlands. The initial density of 1.191 Mg m^{-3}, was above the structural density of the mud, and the initial height of the slurry in the settling column was 0.600 m. The thin layer of high density that developed at the top of the bed is a feature that occurs quite commonly in settling-column experiments. There is often a corresponding change in colour, and the effect is probably a chemical one associated with oxidation of the surface. A comparison with the density profiles presented in figure 8 shows that higher densities have been reached in CTP4 earlier than in KC2, and this was achieved by changing the conditions of water flow. For the first few days of experiment CTP4, consolidation was by upward water flow only, as in all the other experiments reported in this paper. The base drain was then opened to hydrostatic pressure, so that the excess pore pressure in this region of the column dropped immediately to zero, with a corresponding increase in the effective stress and density. During this stage, upward water flow occurred in the upper part of the bed and downward flow in the lower part. Figure 10 shows the excess pore pressures registered during this sequence. The 0-, 2-, 5- and 9-day profiles correspond to the period in which the water flow was only upward, while the 13- and 22-day profiles show zero excess pore pressure at the column base, caused by the bottom drainage to hydrostatic pressure. In the next stage, the drain outlet was lowered to a level below the settling column base, thereby

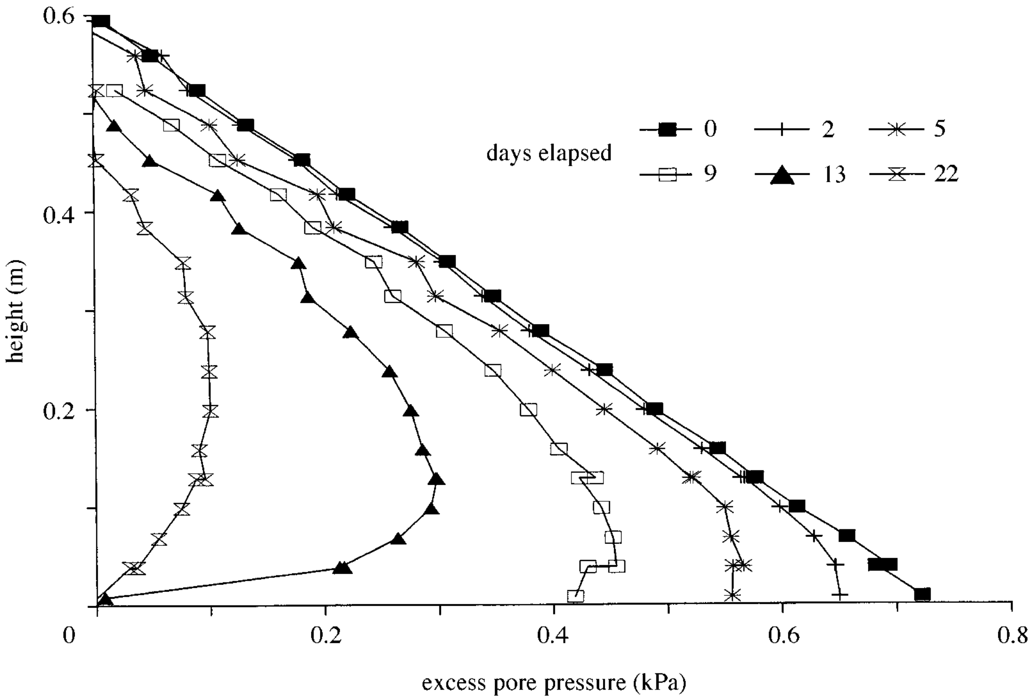

Figure 10. Excess-pore-pressure profiles in experiment CTP4.

causing downward water flow throughout the column. The excess pore pressures became negative, with a marked effect from day 40 onwards, as shown in figure 11. The overlying water level was kept constant using a Mariotte bottle, so that the total stress was not altered by this action. This caused a significant increase in effective stress in the lower part of the bed therefore, leading to further increases in density to values above 1.25 $\mathrm{Mg\,m^{-3}}$, as seen in figure 9. There is a limit to the magnitude of negative excess pore pressure (or pore-water suction) that can be applied in this way, as the consolidation caused by this type of hydraulic gradient is effectively isotropic rather than one dimensional (since the pore-water suction acts equally in all directions). There will inevitably come a stage where the consolidating soil moves inwards away from the walls of the cell or settling column, and the water flow occurs between soil and container rather than through the soil. When this happens, the measured pore pressures become hydrostatic, and no further consolidation can occur. A better approach is to produce a hydraulic gradient by increasing the pressure of the water above the sediment bed, and providing a drain at the level of the base. Figure 9 also shows a situation that can occur when the hydraulic gradient is applied by adding water from an external source. Even though the water had come from the same site as the sediment, it appears that there was a reaction with the water that had been expelled from the consolidating bed, leading to a density variation in the overlying water, as seen in the 71-day profile. This condition can be avoided by applying the hydraulic gradient by pumping the water drained from the bottom of the settling column back into the top of the column, an arrangement which has the added advantage of removing the need for the Mariotte bottle.

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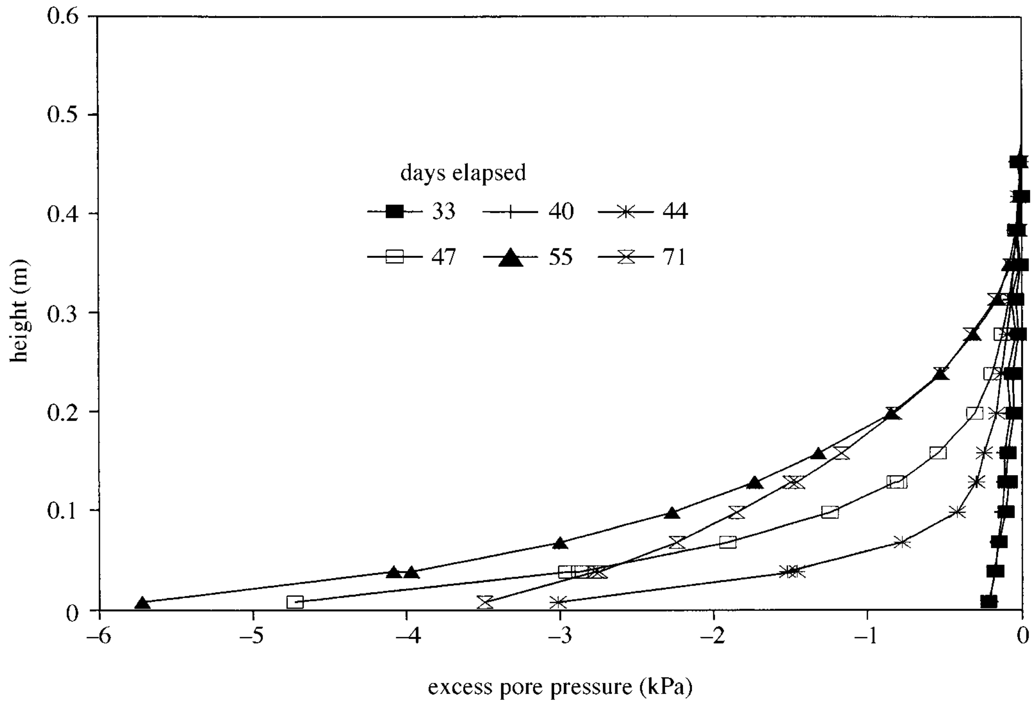

Figure 11. Excess-pore-pressure profiles in experiment CTP4, under application of hydraulic gradient.

4. Analysis of experimental behaviour

Terzaghi (1936) suggested that the structure of soil and its behaviour was uniquely associated with the existence of effective stress. This implies a correspondence with some volumetric parameter, and soil mechanics has traditionally used void ratio, e, specific volume, $v = 1 + e$, and porosity, n. In this paper, the interpretation of the experimental results will be presented in terms of correlations between effective stress and void ratio, to examine whether the void ratio can be said to be a unique function of effective stress, i.e. $e = f(\sigma')$.

Figure 5*b* has shown how the vertical effective stress at any height in the settling column can be calculated by subtracting the measured pore-water pressure from the total vertical stress obtained by integrating beneath the density profile. The corresponding void ratio can be calculated from the density at the same height, and a value for the specific gravity of the soil. A correlation between effective stress and void ratio can be obtained for each pair of density and pore-pressure profiles, and figure 12 shows four of these correlations for experiment NK2. It can be seen that there is considerable downward movement of the curves with time, particularly at the lower effective stresses. Some of the differences between the curves may be a consequence of the accuracy of calculation of effective stress of ± 0.02 kPa, but, even taking account of this, these results suggest that there may not be a unique correlation.

Further insight is provided by the results of a number of settling-column tests on Combwich mud, a natural estuarine silty clay from the River Parrett in Somerset,

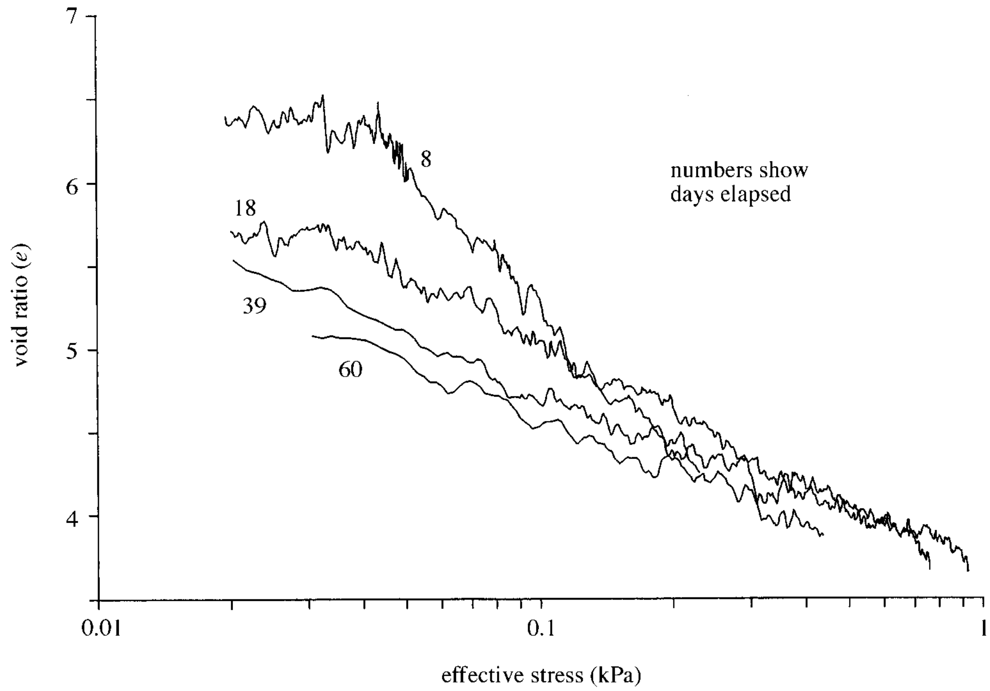

Figure 12. Correlation between void ratio and effective stress for experiment NK2.

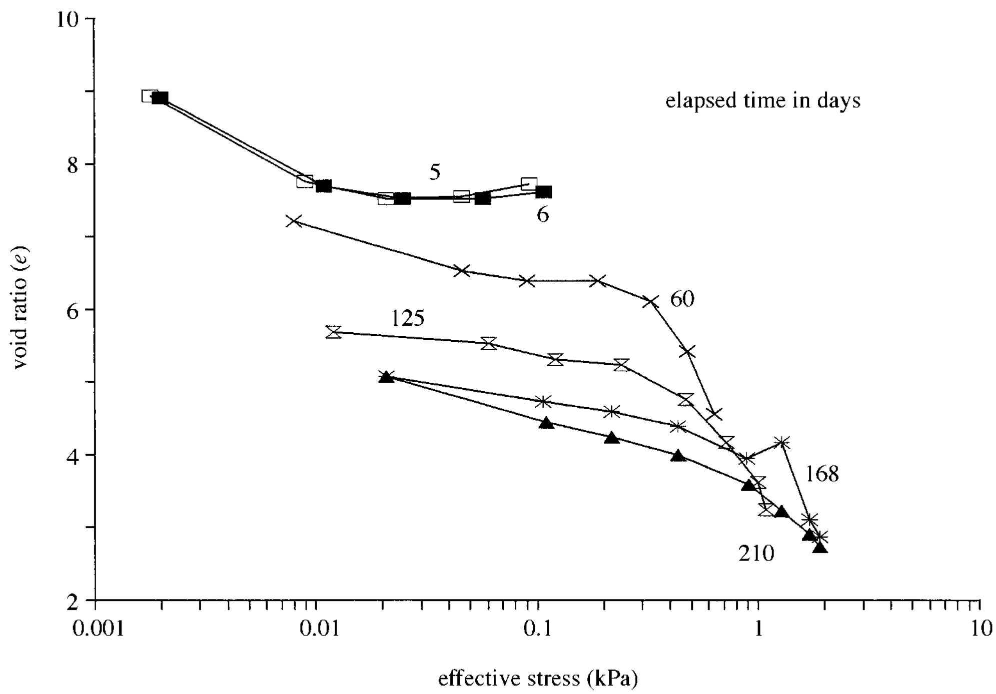

Figure 13. Correlation between void ratio and effective stress for experiment DME10.

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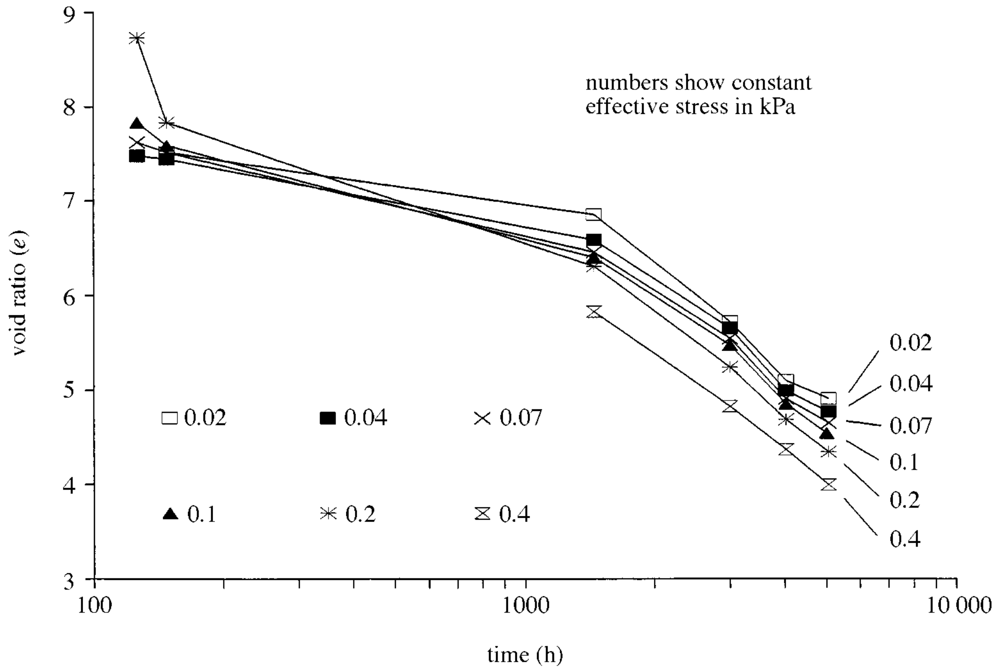

Figure 14. Void ratio changes with time under constant effective stress for experiment DME10.

England, described by Elder (1985) and Elder & Sills (1984). One of these, experiment DME10, started with an initial density of 1.147 Mg m^{-3}, and an initial depth of 1.553 m, and produced the correlations shown in figure 13 between void ratio and effective stress. The effective stress range between 0.001 and 0.01 kPa is included in the graph so that the early void ratios may be shown, but the actual values of effective stress are not accurate in this range. The individual relationships are sufficiently far apart that it is clear that there is not a unique correlation in this case. In fact, it is possible to identify changes in void ratio that occur at constant vertical effective stress by choosing specific effective-stress values, and plotting the corresponding void ratio against time. This analysis is presented in figure 14, in which a logarithmic scale is chosen for the time axis, and a range of effective-stress values is taken from 0.02 to 0.4 kPa. It can be seen that there is very similar behaviour throughout this range of effective-stress values. This process of compression at constant effective stress, or creep, has been discussed in Sills (1995), where it was concluded that creep could occur throughout the consolidation process. The void ratio should therefore be recognized as depending on time or strain rate as well as on the effective stress. Such a conclusion is consistent with Leroueil *et al.* (1985), who studied natural soils which, although soft by traditional geotechnical standards, were stiffer than those described in the present paper.

The other experiments already described provide further examples of correlations between void ratio and vertical effective stress. In the data of CTP4 shown in figure 15, it is not easy to distinguish the three correlations but, in fact, the 47-day profile is virtually a continuation of the 13-day profile to higher vertical effective stresses, while the 71-day profile lies somewhat lower than the other two, suggesting the existence of creep throughout the 5 kPa stress range of the experiment. The cor-

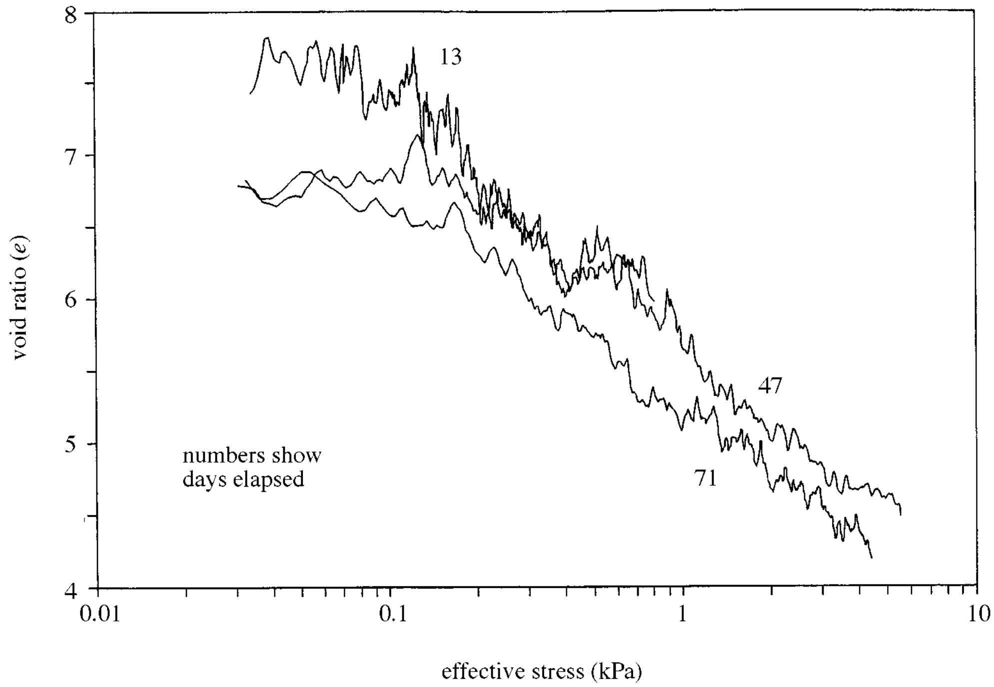

Figure 15. Correlation between void ratio and effective stress for experiment CTP4.

relations for REDM05 are shown in figure 16, where there is a larger range of void ratios at low effective stresses, but the curves appear to converge at effective-stress levels above about 0.2 kPa.

5. Discussion

Before discussing the results of the experiments, it is worth considering the constraints imposed by the experimental setup. Perhaps the most significant of these is the effect of the boundary conditions imposed by the column walls. Friction will reduce the settling rate of the sediment, as well as causing a rotation of the principal stress directions near the walls. However, a comparison by Elder (1985) of settling rates of similar soil slurries in different diameter columns suggested that, for the silty clays used in most experiments, behaviour in a column with a diameter of 100 mm was very similar to the behaviour in larger diameter cells. There is also a possibility of preferential drainage paths developing between the soil and column wall where there may be a lower tortuosity in the flow channels. This would lead to higher effective stresses at the walls than in the middle of a given horizontal plane in the column, and consequently a locally higher density. There are, in fact, indications that preferential drainage paths develop across the whole cross-section near the surface, where small volcanoes can often be seen, particularly in the early stages of an experiment. Where these drainage channels are visible on the side of the column, they are not very deep, but there is still the possibility that macro features such as these influence the time-scale of the consolidation in the settling columns in a different way from that which occurs in the field. Overall, it must be recognized that the sediment

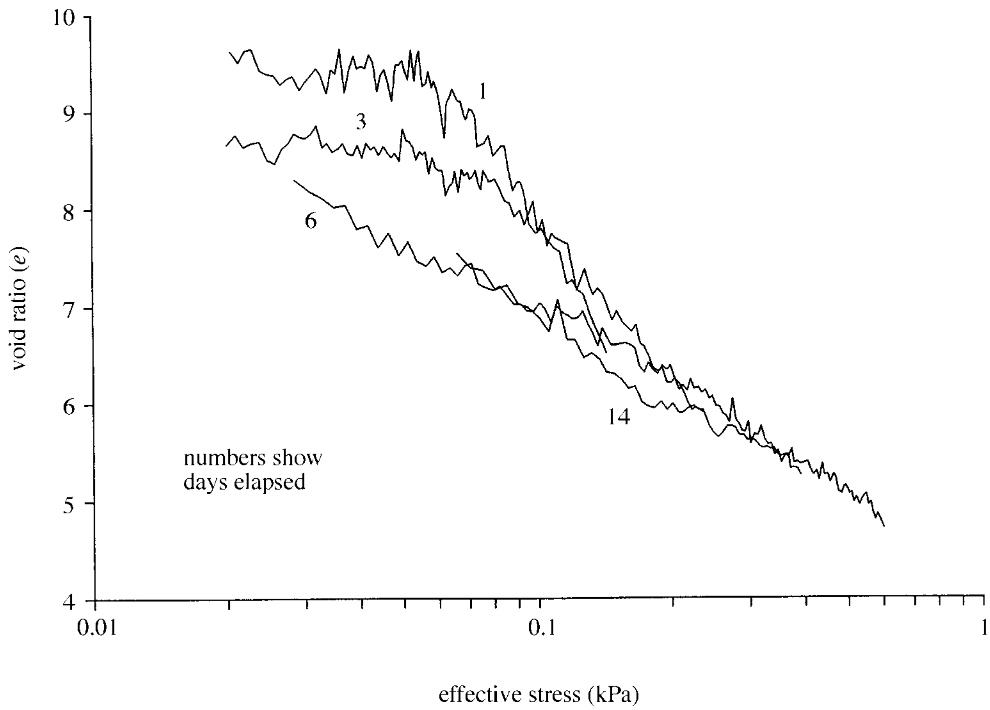

Figure 16. Correlation between void ratio and effective stress for experiment REDM05.

bed will not develop totally uniformly across the cross-section, and that the measurements represent averaged values, as indicated in Elder (1985) and Sills (1995). Similar constraints exist in all laboratory tests, but they nevertheless provide an insight into fundamental soil behaviour, and allow a study of the effects of different initial and boundary conditions, such as varying deposition rates and different bed thicknesses. However, care should be taken when applying soil parameters calculated in the laboratory directly to field conditions.

Not surprisingly, the soil behaviour observed in these experiments varies with the soil itself, and with the stress history, such as initial density or sedimentation rate and depth of bed. Some of the void-ratio effective-stress correlations show more variability with time at low effective stresses than they do at higher values, and there can be a significant change in slope of the correlation of void ratio against the logarithm of vertical effective stress. Examples include the one- and three-day curves for REDM05, where the compressibility is higher for vertical effective stresses less than 0.1 kPa than for those above 0.1 kPa, as shown in figure 16. On the other hand, in NK2 in figure 12 and CTP4 in figure 15, there appears to be some creep throughout the stress range of the experiments, although it is smaller at the higher stress levels than at the low ones, and there does not appear to be a change in slope.

The structural density has been defined as the density which marks the change from a fluid-supported suspension to a soil in which effective stresses exist. There is a corresponding structural void ratio, associated with zero vertical effective stress. Given the creep effects noted in the correlations between void ratio and effective stress, it is not surprising that the structural void ratio for a given soil does not appear to be unique. The vertical effective stress at the surface of the bed is always zero, but in many experiments, examination of the consecutive density profiles suggests that the surface void ratio decreases (density increases) as consolidation proceeds. Thus, there is not a unique void ratio associated with zero effective stress. This observation provides further support for the existence of creep, as discussed in Elder & Sills (1984). However, the shape of the density profile often changes at the same time, becoming more rounded, as in NK2 and REDM05, for example. When this happens, it is difficult to determine the value of the surface void ratio precisely, thus precluding quantitative correlations in general.

Further evidence for the non-uniqueness of the structural void ratio is presented by Sills & Thomas (1984), who showed that a slowly sedimented mass of Combwich mud (whose behaviour is illustrated in this paper by the experiment DME10) can develop effective stresses at a much lower density, i.e. higher void ratio, than the same mass of soil deposited quickly. In one experiment, XRYCI1, 906 g of sediment input over 0.1 days produced a bed of height *ca.* 0.2 m after 46 days. In contrast, in experiment XRYCI5, a similar mass of sediment, 916 g, was input over 9 days, and the bed height was around 0.5 m after 50 days. In neither case had the surface settlement completely stopped by the times quoted, but the settlement rates had become very slow, and it seems unlikely that the bed produced by the slower rate of input would have consolidated sufficiently to reach a thickness as small as that produced by the faster input rate. Edge & Sills (1989) reported the results of experiments on a sandy silty clay in which small quantities of the sediment were introduced at frequent intervals during the day, with no addition during the night. They found that during the day, after the first two or three inputs, the sand penetrated the sediment that had settled earlier in the day, but that for the first input, the sand formed a high-density layer above the last input of the previous day. They interpreted this behaviour as being due to a time-dependent increase in stiffness and strength of the silty clay that had settled at the surface of the bed overnight, that enabled it to support the first sand. Later in the day, the silty clay at the surface had little time to increase in strength, so that it could not support the sand. A similar reasoning could apply to the slow sedimentation case reported by Sills & Thomas (1984), whereby the flocs arriving at the surface of the bed would strengthen if they had sufficient time to do so before the arrival, and loading, of the later flocs. This led to an open structure, albeit one that was liable to sudden collapse if further sedimentation or a surcharge load exceeded the resistance developed, behaviour which was in fact noted in other experiments (Sills 1995).

The laboratory conditions are an idealized representation of a sedimentary process in the field, so that it is of interest to consider the results in terms of the void index, I_v, defined by Burland (1990) as $I_\mathrm{v} = ((e - e^*_{100})/C^*_\mathrm{c})$, where $C^*_\mathrm{c} = e^*_{100} - e^*_{1000}$ and the intrinsic void ratios, e^*_{100} and e^*_{1000}, are the void ratios of a reconstituted sample at a vertical effective stress of 100 kPa and 1000 kPa, respectively. The intrinsic void ratios, e^*_{100} and e^*_{1000}, should ideally be measured from an oedometer test, but they may also be obtained by calculation from the 'liquid limit'. Burland (1990) considered data spanning a vertical effective-stress range from just below 1 kPa to 10 000 kPa, and suggested that the *in situ* state of many normally consolidated soils lay on a sedimentation compression line (SCL). Figure 17 is a plot of void index against vertical effective stress in the range 0.01–10 kPa, showing the position of the SCL on the assumption that it can be extrapolated back to the lower stress range. It also shows a selection of the experimental results already presented, but now plotted in

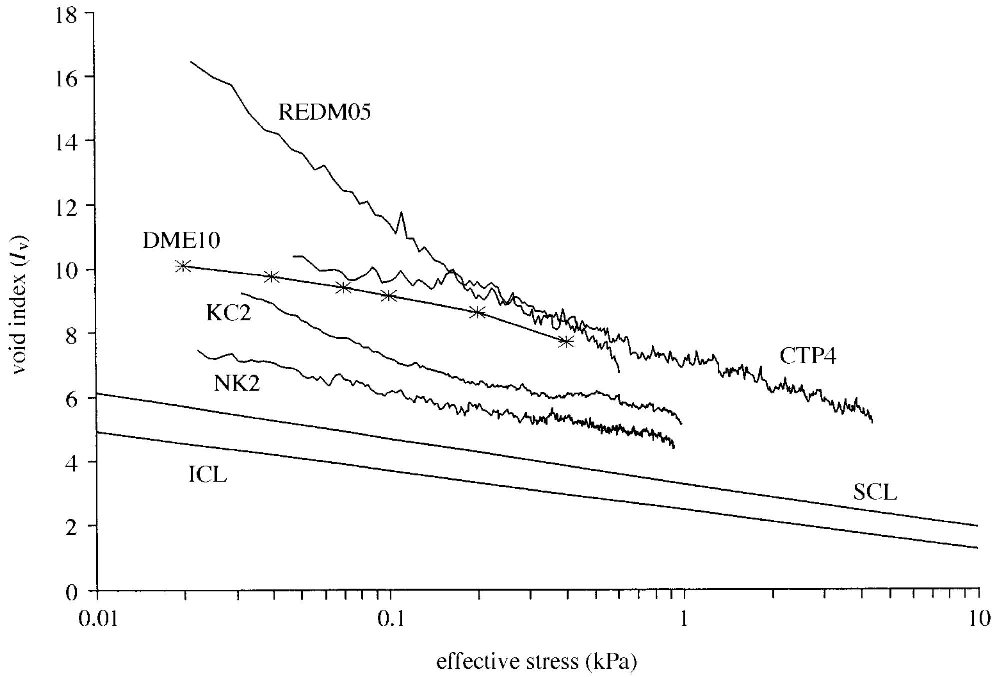

Figure 17. Correlation between Burland's void index and effective stress for all experiments.

terms of void index against vertical effective stress. The selection has been made by taking the latest of the correlations between void ratio and vertical effective stress for each experiment, corresponding to the lowest void ratio and hence the maximum creep compression. The conversion to void index has been made assuming the liquid limits recorded in table 1, and the empirical correlations quoted in Burland (1990)

$$e^*_{100} = 0.109 + 0.679e_{\mathrm{L}} - 0.089e^2_{\mathrm{L}} + 0.016e^3_{\mathrm{L}} \qquad \text{and} \qquad C^*_{\mathrm{c}} = 0.256e_{\mathrm{L}} - 0.04,$$

where e_{L} is the void ratio at the liquid limit. It should be noted that there is some variability in the liquid-limit values obtained for different batches of the mud from Ketelmeer and the Slufter, and the values quoted in table 1 for NK2 and CTP4 were not measured on the actual batch of mud used for the experiments. It can be seen that all the data lie well above the SCL, and that at the higher effective stress levels, the correlations are converging gradually on it. It is not unreasonable to suggest that a continuation of the creep process over a geological time-scale could bring the curves close to the SCL in a continuing development of soil structure. It is also possible that, rather than extrapolating back linearly, the SCL bends upwards at the lower effective stress levels, and that the laboratory data may then be closer to the SCL than appears.

(*a*) *Effective stress*

The traditional model of soil compressibility uses a linear correlation between void ratio and the logarithm of effective stress, and figures 12, 15 and 16 suggest that this may be an appropriate assumption for effective stresses greater than 0.1 kPa. (It is not, of course, possible to say whether the same linear correlation would extend to stress levels greater than those reached in the settling column experiments.) There may also be a linear correlation within the effective-stress range 0.01–0.1 kPa, but with a different slope. Similar bilinear behaviour was reported by Been & Sills (1981) in experiments on Combwich mud in which they observed a higher compressibility associated with vertical effective stresses lower than *ca.* 0.8 kPa than with higher values. They described this low-effective-stress region as being in a state intermediate between suspension and soil. This approach was formalized by Pane & Schiffman (1985), who redefined the effective stress at these low values by the introduction of a β factor. They assumed that $\sigma' = \beta(\sigma - u_{\mathrm{w}})$, with $\beta = 0$ corresponding to a suspension, $\beta = 1$ a soil, and $0 < \beta < 1$ the intermediate state. This practice is not adopted here, as it is more satisfactory to maintain the historical definition of effective stress as the difference between total stress and pore-fluid pressure. Once this is accepted, then it is logical to define the existence of a soil to be coincident with the existence of an effective stress. Thus, if the pore-water pressure is equal to the total stress, there is no effective stress, and the sediment is in suspension, behaving as a dense fluid. If the pore-water pressure is less than the total stress, then an effective stress exists, as does a soil structure. It is, of course, possible for the soil characteristics, such as its tendency to creep, its compressibility and its permeability, to change with changing effective stress level. The condition of zero effective stress occurs only as a limiting value at the surface of the sediment bed. It may be noted that this definition of the existence of soil, in terms of the existence of effective stress, is not necessarily reversible, although a condition of liquefaction, in which the pore-water pressure increases to a value equal to the total vertical stress does correspond to zero soil strength and behaviour as a dense fluid.

It should be noted that there is no physical meaning to the parameter of effective stress. It cannot be measured directly, but can only be calculated from measurements of total stress and pore-fluid pressure. It can be used only in a continuum approach to modelling soil behaviour, and cannot therefore be identified with the intergranular stress or related by equations to other longer-distance forces that act within soils, such as the electrochemical forces between clay particles, as has been suggested by some authors. It is certainly true that the balance of local forces will change as the soil compresses, and that changing the local conditions—for example, by ion exchange—will alter the soil behaviour, but the definition of effective stress, like any semantic definition, should not change.

6. Field applications

The effective-stress levels encountered in traditional soil mechanics are often at least two or three orders of magnitude greater than those reached in the experiments described in this paper. However, there are some situations where an understanding of the behaviour at very low stress levels is crucial.

One example is the prediction of the storage capacity of a disposal site in still water. Mining and dredging operations can produce waste that cannot be dispersed into the community, and storage methods frequently involve high deposition rates into deep pits, as in the oil sands tailings ponds of Alberta, Canada, and the 30 m deep Slufter disposal site in The Netherlands. At the end of consolidation of the waste material, vertical effective stresses at the bottom of this latter site could reach 150 kPa, but long drainage paths would mean that the time-scale to reach this condition could be

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many tens of years. For a significant time, and over a significant portion of the pit, the effective-stress levels would be very low, and the type of creep processes observed in the settling columns could be active and, indeed, dominant.

A second example is a situation such as the storage of bauxite tailings, or red muds, from the mining operation in Minas Gerais, Brazil, which are discharged into a river dammed to produce a storage depth of the order of 20 m, described by Consoli (1997) and Consoli & Sills (1999). The mining waste drops out of the river flow at a rate determined by its concentration and transport speed, and then consolidates. In such a case, both the sediment transport and the subsequent compression are significant in determining where and how the sediment is retained, and, again, the effective stresses would be expected to be low over a long period of time. It is encouraging to note that the surface void ratios measured in samples from the top 0.1 m of the bed were found to be very similar to those reached in the settling column experiments on red muds, illustrated by REDM05.

A third example is the application to dredging programmes, where geophysical survey equipment may identify the presence of sediment in the water without being able to determine whether it poses a navigation hazard. An understanding of the transition from suspension to soft soil, coupled with a suitable model, could identify present and even future problems.

7. Conclusions

The development of soil structure has been observed in soils consolidating from a suspension. It has been argued that the transition from a fluid-supported suspension to a soft soil occurs unambiguously when pore-fluid pressures drop from being equal to the total vertical stress to being lower than the total vertical stress. At this stage, by definition, effective stresses begin to exist. The consolidation process has been studied for a number of soils in which the initial effective stresses have been very small, under conditions of self-weight stresses alone, followed in some experiments by surcharge or hydraulic gradient loading. It has been shown that large strains, and large void ratio changes, occur while the soil is extremely soft, and that these changes cannot be attributed only to changes in vertical effective stress, but occur concurrently as creep. Although the creep compression appears to reduce at higher effective stress levels, there is considerable evidence in the literature (see, for example, Leroueil *et al*. 1985), that it remains a significant phenomenon. The consequences for modelling soil behaviour are that a constitutive relationship is required in which the volumetric state e is assumed to vary in some way with time. Although one possibility would be a direct dependence on time, there are difficulties with identifying the $t = 0$ condition in geological time, so that a functional relationship with strain rate, of the form $e = f(\sigma', \dot{e})$ would be more practical.

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References

Alves, M. C. M. 1992 Comportamento de sedimentação e adensamento de uma lama vermelha. PhD thesis, PUC-Rio. (In Portuguese.)

Been, K. 1980 Stress–strain behaviour of a cohesive soil deposited under water. DPhil thesis, University of Oxford, UK.

Been, K. & Sills, G. C. 1981 Self-weight consolidation of soft soils: an experimental and theoretical study. *Géotechnique* **31**, 519–535.

Bowden, R. K. 1988 Compression behaviour and shear strength characteristics of a natural silty clay sedimented in the laboratory. DPhil thesis, University of Oxford, UK.

Burland, J. B. 1990 The Rankine lecture: on the compressibility and shear strength of natural clays. *Géotechnique* **40**, 329–378.

Consoli, N. C. 1997 Comparison of the measured and predicted performance of tailings sedimentation. *Proc. Inst. Civ. Engineers, Geotech. Engng* **125**, 179–187.

Consoli, N. C. & Sills, G. C. 1999 Soil formation from tailings: comparison of predictions and field measurements. *Géotechnique*. (In the press.)

Edge, M. J. & Sills, G. C. 1989 The development of layered sediment beds in the laboratory as an illustration of possible field processes. *Q. Jl Engng Geol.* **22**, 271–279.

Elder, D. McG. 1985 Stress–strain and strength behaviour of very soft soil sediment. DPhil thesis, University of Oxford, UK.

Elder, D. McG. & Sills, G. C. 1984 Time and stress dependent compression in soft sediments. In *Sedimentation/consolidation models* (*Proc. ASCE Symp. on Prediction and Validation of Consolidation, San Francisco*) (ed. R. N. Yong *et al.*), pp. 425–444.

Kynch, G. J. 1952 A theory of sedimentation. *Trans. Faraday Soc.* **48**, 166–176.

Leroueil, S., Kabbaj, M., Tavenas, F. & Bouchard, R. 1985 Stress–strain rate relation for the compressibility of sensitive natural clays. *Géotechnique* **35**, 159–180.

Pane, V. & Schiffman, R. L. 1985 A note on sedimentation and consolidation. *Géotechnique* **35**, 69–72.

Sills, G. C. 1995 Time dependent processes in soil consolidation. In *Proc. Int. Symp. Compression and Consolidation of Clayey Soils, Hiroshima, Japan* (ed. H. Yoshikuni & O. Kusakabe), pp. 875–890. Rotterdam: Balkema.

Sills, G. C. 1997 Hindered settling and consolidation in cohesive sediments. In *INTERCOH 1994, 4th Nearshore and Estuarine Cohesive Sediment Transport Conf.* (ed. T. N. Burt, W. B. Parker & J. Watts), pp. 107–120. Wiley.

Sills, G. C. & Thomas, R. C. 1984 Settlement and consolidation in the laboratory of steadily deposited sediment. In *Proc. IUTAM/IUGG Symp., University of Newcastle-upon-Tyne, September 1983, Seabed Mechanics* (ed. B. Denness), pp. 41–49. London: Graham & Trotman.

Terzaghi, K. 1936 The shearing resistance of saturated soils and the angles between the planes of shear. *Proc. 1st Int. Conf. Soil Mechanics*, vol. 1, pp. 54–56.