Centrifuge tests on embankments on strengthened and unstrengthened clay foundations

M. S. S. ALMEIDA, M. C. R. DAVIES and R. H. G. PARRY (1985). *Géotechnique* 35, No. 4, 425–441

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The Writers are currently involved with the design and installation of sand piles for a substantial reclamation on soft coastal clays and in this context wish to raise several points, particularly relating to the interpretation and practical application of this work.

The role of the sand piles in limiting vertical and horizontal movements is particularly well demonstrated in fig. 13 of the paper. (It might incidentally be clearer to express both sets of height data in terms of $h/h_{\max}$ (MA3), making the point that the unstrengthened embankment failed at a little over 11 m (prototype) while the strengthened embankment MA6 had not failed at around 13 m.) A conventional 'improvement ratio' (settlement unreinforced/settlement reinforced) of around 2.3 is indicated at 11 m embankment height. This is somewhat greater than would be deduced from current theories and available case histories which suggest a range of 1.2–1.9 at 5% area ratio (area of columns/total area), see e.g. Greenwood & Kirsch (1984). It is also interesting that this work gives much greater improvement than Charles & Watts' (1983) laboratory tests on single gravel columns in a large-scale oedometer (see later). The improvement ratio in horizontal movement is also noteworthy—in excess of 3 at 11 m embankment height, although it was lower before stage 3. This lends further substance to the practice of monitoring horizontal movements to give early indications of approaching instability.

It is clear that the concept of improvement ratio is simplistic, in that it compares very different stress situations and will vary with the stage of loading and the consolidation time. The wide variety in reported results may be partly due to not comparing like with like. For instance, the improvement ratio must necessarily be large if the unstrengthened foundation approaches failure, but the reinforced case does not. Conversely a

low ratio could indicate that the unreinforced soil was still far from failure. Nevertheless, the improvement ratio gives a useful first indication and basis for comparison and will no doubt continue to be used in practice. To standardize, it is suggested that comparisons be restricted to virtual completion of primary consolidation, or at least similar stages of consolidation. In the present case the columns considerably accelerated the settlements, so that at comparable loadings and times the reinforced case would have undergone a greater proportion of primary settlement than would the unreinforced case. The test results suggest that there could have been an even greater improvement ratio if both cases had been taken to completion of consolidation.

The pore pressure data are less amenable to simplification. With the present state of knowledge, field pore pressure readings might be hard to interpret and thus difficult to use for field control. Do the Authors consider that the results obtained could have been predicted with reasonable accuracy from judicious application of theory and choice of parameters? It was reported that the sand-sized particles scaled up to stone size 30–50 mm; does the clay scale up similarly to silt size? How did the observed pore pressure behaviour compare with what would be predicted from the application of conventional consolidation theory, using say suitably scaled oedometer-derived c_v values?

Most of the improvement in stability was attributed to an acceleration of consolidation for the fairly low replacement ratio (5%) used in test MA6. How did this acceleration compare with what might have been expected from conventional theory for the use of 1.0 m dia. sand drains at (prototype) 4 m centres?

It is unfortunate that the strengthened foundation test did not reach a full failure condition. The unreinforced foundation MA3 failed with a calculated factor of safety of 0.91—the underprediction of F_s being attributed to side friction. In this case the calculation procedures are well established and supported by much other experimental data.

The reinforced foundation test, however, appears from the experimental data to have been a considerable way from failure, and the Writers suggest that the real factors of safety throughout this test series could have been considerably greater than shown in fig. 19. Without failure, the line plotted for MA6 is not validated by experiment but is a statement of the results of applying Priebe's theory to the experimental pore pressure readings etc. Priebe's theory may be conservative; the settlement data indicate greater improvement. This implies a more favourable stress redistribution, columns taking a greater proportion of load, which in turn implies a better factor of safety against slope instability. Fig. 19 might therefore be misleading.

In applying Priebe's theory, the Authors have a printing error in equation (1). It should read

$$\tan \bar{\phi}' = m \tan \phi_s' + (1 - m) \tan \phi_c'$$

As stated, this equation implies a relatively small increase over non-composite strength parameters for the low area ratio of 5% adopted in these tests. The Authors have derived a value of $m = 0.142$ from Priebe's graphs, which give m as a function of ϕ_s' and area ratio. Do they agree that the test results indicate this would be a lower bound on m in this case, and it could be significantly higher?

It is noted that the Authors took a value of $\phi_s' = 45^\circ$ for the columns. Since ϕ_{crit} for the sand may be about 33° , the higher value was presumably adopted as a consequence of the achievable relative density of 89% when forming the columns. (45° was apparently also assumed for the embankment at 78% relative density.) It is important to control field compaction carefully and to choose ϕ' accordingly. Although the use of $\phi_s' = 45^\circ$ with Priebe's theory gave 'safe' results here, is this possibly because of conservatism in the theory? These sand piles performed significantly better than Charles and Watts' gravel columns, which had higher quoted ϕ_s' values of 53° , 51° and 47° for three tests.

It was of interest to note the observation that preliminary tests without surcharge resulted in the unloaded surface's swelling back to a slurry (presumably with the groundwater level at the soil surface). Later, it was reported that in the slope stability analyses, where forces at the base of slices were computed to be negative, they were set to zero. If this was purely a computational problem associated with Bishop's simplified method it would be interesting to compare this with results of a more rigorous method. If, however, pore pressures in the toe area actually exceeded the overburden pressures, the Writers would suggest that these should be modelled in

the analyses (appreciating that limit equilibrium does not attempt to consider the redistribution of stresses). This could significantly reduce the calculated factors of safety. The Writers believe that this situation can arise in practice. They were recently involved with a case where this was the most likely explanation for an earlier slip on a nearby embankment. This was therefore considered in the design of a new flood embankment; monitoring of the construction indicated that the new works might have failed from this cause, but for the presence of reinforcing geotextile.

The Paper has raised a number of complex issues, some of which are interlinked, so that it is difficult to isolate individual components.

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Authors' reply

Most of the work reported in the literature, and indeed the three papers referred to by the Writers, have involved loading the granular columns under pure confined compression. However, the Authors' centrifuge tests involved non-symmetrical loading of the columns under the embankment slope which resulted in significant shear distortion being applied to the columns. Additionally the embankment loading was applied in stages whereas theoretical and experimental studies reported by the Writers involved either instantaneous loading or loading increased monotonically with time. Finally the granular columns were not used with the sole purpose of limiting settlements but also to accelerate pore pressure dissipation and to improve the shearing resistance of the composite foundation.

For these reasons the Authors have avoided the use in the Paper of the expression 'improvement ratio', which has been linked with the use of granular columns with the main purpose of limiting settlements under the whole embankment. These reasons should also explain the differences in improvement ratios deduced from the centrifuge tests, in which columns were inserted in the area of the foundation beneath the embankment slope only, and those reported in the literature,

Fig. 1. Measured vertical settlements in tests MA3 and MA6 at the end of stage 3 loading and estimated final values had full pore pressure dissipation occurred

where columns were placed beneath the entire embankment.

The Writers have suggested restricting a comparison of settlements of reinforced and unreinforced foundations to virtual completion of primary consolidation. Because the loading history did not allow completion of primary consolidation, settlement profiles were extrapolated to that condition based on pore pressure and settlement data. These computations, presented previously by Almeida & Parry (1985), are shown in Fig. 1 for the stage 3 loading when both unstrengthened and strengthened clay foundations had virtually the same loading. The improvement ratio for the extrapolated situation is approximately the same in magnitude as the value measured following partial consolidation.

Unlike the Writers, the Authors do not see any difficulty in using measured pore pressures for control during construction, provided that stability charts or access to suitable computer programs are available on site to enable the computation of factors of safety given the pore pressure data. However, as has been suggested by the Writers and demonstrated in centrifuge models by Davies & Parry (1985), horizontal measurements should also be monitored to give early indications of approaching instability.

Because of centrifuge scaling laws, soil in a centrifuge model represents soil of a larger grain size in the prototype. However, the engineering behaviour of the material in the model is not affected by being subjected to an elevated gravity field, and material parameters measured at 1g, at the appropriate level of effective stress, are applicable to the analysis of the model. For consoli-

dation, because in the model the drainage path is shorter than in the prototype, consolidation times are accelerated by g^2 , e.g. consolidation for 10 h in a model at an acceleration of 100g is equivalent to 11.4 years in a full-scale prototype.

Owing to the complexities involved, no attempt was made using simple theories to predict values of pore pressure during embankment construction and the subsequent consolidation. However, a finite element analysis, which uses a critical state model with the Biot coupled consolidation theory, has been performed for the unstrengthened clay foundation (Almeida, Britto & Parry, 1986). Values of porewater pressure predicted by this analysis for both generation and dissipation stages agreed well with measurements in the centrifuge experiment. The three-dimensional nature of the consolidation in the strengthened foundation was outside the capabilities of the finite element program. Consequently, no attempt was made to predict the pore pressure response in this case.

Using the conventional theory for consolidation using sand drains (Barron, 1948), the average degree of consolidation in the kaolin layer of the foundation in the area of the foundation bounded by sand drains was calculated to be 95% following 40 min consolidation. This value agrees well with the measurements of pore pressure transducer P7 in test MA6, shown in fig. 15(b) of the Paper. About 40 min after each construction lift the transducer, which was located between two rows of sand drains, showed almost complete dissipation of excess porewater pressure generated during the lift. By comparison, at the location of P7 in test MA3 (the unim-

proved foundation), using one-dimensional consolidation theory the local degree of consolidation was calculated to be 40% following a similar period of dissipation. This value compares well with the observed response of the transducer, fig. 15(a) of the Paper. The rate of dissipation of pore pressures and hence the rate of improvement in stability in the area of the foundation adjacent to the improved zone was also enhanced owing to the reduced drainage path lengths, as can be seen from the response of transducers P9 and P10 in fig. 16 of the Paper. However, the analysis of this situation is outside the scope of the conventional theory for sand drains.

Regarding the application of Priebe's theory, the Authors agree that this theory may be conservative and that the real values of m used in equation (1) (there is indeed a printing error in this equation as pointed out by the Writers) are expected to be higher than those given by Priebe's theory. This implies that columns really take a greater proportion of the embankment loading.

The value of the angle of shearing resistance, ϕ_s' , assigned for both the sand columns and the sand embankment with relative densities RD varying between 78% and 89% was 45° based on experimental data available at Cambridge. While there was careful control of relative densities of the sand columns, RD = 89%, prepared after stopping the centrifuge, the same was not possible for the sand embankment, RD = 78%, constructed during centrifuge flight.

When conducting stability analyses using Bishop's method, the situation rarely occurred where negative forces were calculated at the base of slices, necessitating the value to be set to zero.

Therefore, this particular computational problem did not significantly affect the results of the analyses. In any slip circle method porewater pressures in excess of the overburden pressure will reduce the frictional resistance along the slip surface to a minimum value of zero, and since excess porewater pressures act normally to the slip surface and therefore through the centre of rotation these pressures can have no overturning or restoring influence. However, redistribution of stresses may result in a reduction in effective stress and hence the frictional resistance at adjacent locations along the slip surface. Quantifying the extent of redistributed pore pressure is very complex, particularly since any zones of pore pressure greater than the overburden pressure which may occur will be of a limited extent. The Authors do not agree with the Writers that this condition should be modelled in this type of analysis but rather by more sophisticated analytical techniques.

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