Problems Associated with the Construction of the Ebute Metta Causeway over Soft Clays in Lagos, Nigeria

Problèmes liés à la construction de la jetée Ebute Metta sur des argiles molles à Lagos, Nigéria

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SUMMARY

The problems associated with the construction of a road embankment over thick deposits of soft highly compressible clays are described. Laboratory tests have been useful in predicting the magnitude of the settlement as a result of primary consolidation and also the rate of secondary settlement. In the field, the rate of settlement during primary consolidation was much faster than had been estimated from laboratory tests. Field data on the magnitude and the rate of settlement are given, and their value in predicting the future behaviour of the road, when additional fill is placed, is emphasized.

SOMMAIRE

L'auteur décrit les problèmes liés à la construction d'un remblai de route sur d'épaisses couches d'argile molle et très compressible. Les essais de laboratoire se sont avérés très utiles pour prédire l'ampleur des tassements dus à la consolidation primaire et la vitesse des tassements secondaires. On rapporte que la vitesse de tassement durant la consolidation primaire sur le chantier fut beaucoup plus rapide que prévu d'après les essais de laboratoire. Les mesures de tassement sont données, et on souligne leur importance dans la prédiction du comportement de la route si d'autres remblais y sont ajoutés.

LAGOS IS SITUATED on a tidal lagoon which forms part of the Niger delta complex. There are large areas of swampland around the edges of the lagoon and, whenever roads have been built across these areas, very large settlements have taken place. During 1957 and 1958 the Ebute Metta Causeway was constructed, providing an opportunity for carrying out a soil investigation and the taking of field settlement observations on the road embankment.

THE SITE

The general nature of the site is shown in Fig. 1. The road crosses several zones of relatively strong and incompressible ground, but, between these zones, there are areas in which the subsoil consists of thick layers of soft and compressible swamp deposits. Although recent sand filling operations obscure some of the original ground surface, the difference between the hard and soft ground may readily be seen. The harder areas (a ridge of which may be seen running diagonally across the central portion of the photograph) show up as the lighter zones on the photograph and they are probably the remnants of the topography which existed before the recent swamp deposits were laid down as the sea level rose at the end of the Pleistocene period. The swamp deposits consist of organic or peaty clays which have filled the pre-existing valleys and these have a characteristic texture on the photograph.

From the results of boreholes and soundings it has been possible to construct the geological cross-section shown in

FIG. 1. Aerial photograph of the Ebute Metta Causeway.

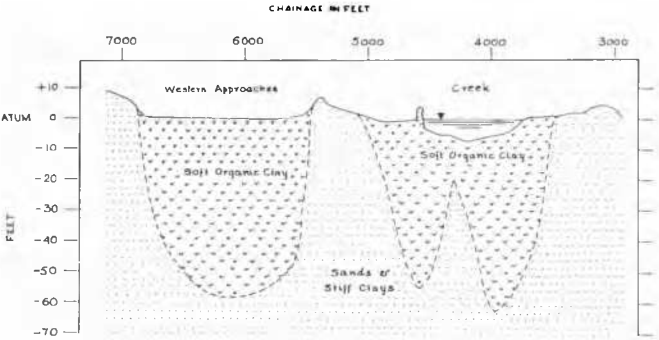

FIG. 2. Geological cross-section.

Fig. 2. The two buried valleys filled with soft organic clay are easily distinguished. The harder materials between and below the base of the buried valleys consist of sands and stiff clays but insufficient data is available to show all the details on the cross-section.

Two features of the valleys are worthy of note. The first is the buried ridge which can be seen below the creek. The other feature is that the clays near the edge of the valley, below the Western Approaches, are much more peaty (and therefore more compressible) than those in the central portion, because of the vegetation which flourished along the edges of the valley during the period of rising water levels.

The tidal range in the lagoon is fairly small and the general level of the swamp deposits is about that of high tide. The local survey datum used in plotting the elevations corresponds both with the general level of the swamps and the normal groundwater level.

THE SOIL PROPERTIES

A typical set of results obtained from the borings in the central portion of the Western Approaches and in the creek area is shown in Fig. 3.

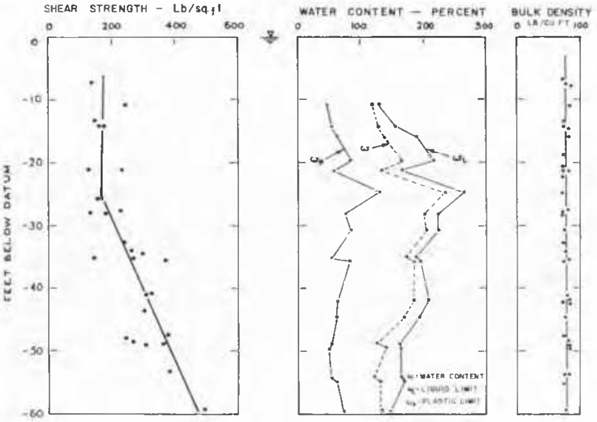

FIG 3. Typical soil test results.

As is common with normally consolidated clays, there is a weathered crust, but in this case the thickness of the crust is greater than is usual. Within the weathered crust the undrained shear strength of the clay has a constant value of about 180 lb/sq.ft., but, below the crust, there is an increase of strength with depth. The ratio of the increase in strength to the increase in the effective overburden pressure, (c/p), is 0.55 which compares very well with the value of 0.52 predicted from the correlation between (c/p) and the plasticity index proposed by Skempton (1957).

The water content of the clay is close to the liquid limit, which has an average value of nearly 200 per cent. Higher liquid limits and water contents were found near the edges of the buried valley beneath the Western Approaches and in some cases the liquid limit was greater than 400 per cent.

Owing to the high water contents, the bulk densities of the clays are low and the results plotted in Fig. 3 show an average value of only 80 lb/cu.ft. The corresponding submerged density is about 17 lb/cu.ft. Most of the results lie close to the A-line of the plasticity chart (Fig. 4). Those points that fall considerably below the A-line are associated with the more organic materials found along the edges of the valley in the Western Approaches area.

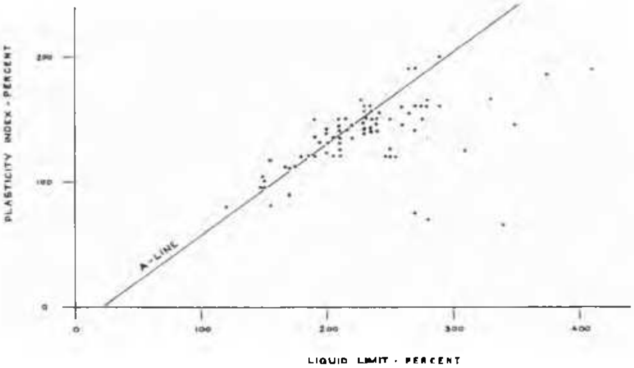

FIG. 4. Plasticity chart.

Consolidation tests were carried out on a number of specimens. Good agreement between the measured values of the compression index, C_c, and those predicted by the expression $C_c = w\%/100$ was obtained. The values of $C_c/1 + e_0$ ranged from 0.2 to 0.4.

Values of the coefficient of consolidation varied from 0.1 to 0.5 sq.ft. per month with an average value of 0.3 sq.ft. per month. Most of the laboratory tests did not show any significant differences between the horizontal and vertical permeabilities but a few special tests, carried out at the Delft Laboratories, did indicate substantially larger horizontal permeabilities.

The general nature of the clays and the field evidence of the behaviour of other causeways in the area suggested that secondary consolidation would be of importance. Attempts were made in the laboratory to evaluate the coefficient of secondary compression, C_α, defined as the amount of secondary compression per unit thickness per log cycle of time. These tests were very time-consuming and only four completely satisfactory results were obtained. The values of C_α were found to lie between 0.046 and 0.068. These are a little higher than would be suggested by the relationship $C_\alpha = 0.00018\ w$ (per cent) suggested by Moran, Proctor, Mueser, and Rutledge (1958).

The ratios of the secondary settlement per log cycle of time to the primary settlement under the particular load increment were found to lie between 0.19 and 0.34.

THE PROBLEMS OF CONSTRUCTION

Settlement

The magnitudes of the settlements under the strip loads of the width of the embankment are indicated in Fig. 5 where the primary settlement for various values of the soil compressibility are plotted. The depth of clay assumed in the calculations is 60 ft. The sand fill available had a bulk density of 125 lb/cu.ft. and typical loadings due to the fill are 1,200 lb/sq.ft. On the least compressible of the clays this loading would lead to a settlement of 6.5 ft.

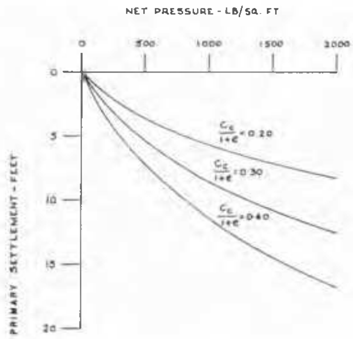

FIG. 5. Settlement of ground surface under fill loading.

In view of the large settlement, the change in the effective density of the fill as it settles below the water table is of importance and it is more convenient to convert the surface loadings into fill thickness, due allowance being made for the change in effective density. A diagram which permits the determination of the thickness of fill, necessary to achieve a desired final road elevation, is shown in Fig. 6. Three values for the compressiblility of the clay, corresponding to the values measured in the laboratory tests, have been used.

As the final road levels in the Western Approaches are required to be between 7.3 and 8.5 ft. above datum in order to achieve this, fill thicknesses of up to 25 feet are necessary.

Stability

The soft and highly compressible nature of the ground and the great thickness of fill required to achieve even a modest final embankment elevation meant that there were very severe stability problems. Experience in placing fill material on the surface of soft clay deposits has shown that the strength of the fill may be neglected and that the stability problem may be treated in terms of the bearing capacity of a strip foundation. In these circumstances the bearing capacity factor, N_c, is 5 and failures might be expected under a loading of 900 lb/sq.ft., a loading corresponding to a height of fill of 7.2 ft., using full bulk density. In practice, however, the fill was placed in a number of lifts by hydraulic methods and some of the settlement occurred under the earlier lifts and before the final layers were placed. This settlement not only led to some strengthening of the clay but the effective density of the sand that had settled below the groundwater table was reduced. If both these factors are taken into account the calculations show that, with the construction schedule adopted, failures should occur when the fill thickness exceeded 10 ft. In fact, a number of failures did occur when this thickness of fill was reached and it was necessary to construct berms, about 6 ft. thick, on each side of the road embankment to allow the required thickness of fill to be placed.

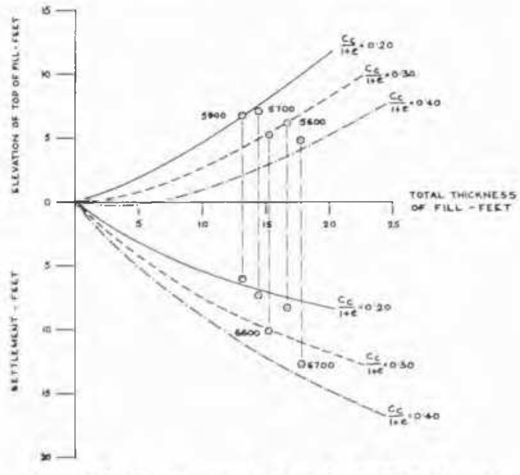

FIG. 6. Design curves for required fill thickness.

CONSTRUCTION PROCEDURE

At an early stage in the construction programme, the possibility of using sand drains to accelerate the rate of settlement and gain in shear strength was investigated. In the circumstances at the site these would have been very expensive and the cheapest procedure was to construct the road to the highest possible elevation and to place a temporary wearing surface on the top of the embankment. After the main part of the primary settlement was completed it was proposed that the road would be raised again so that the final profile would conform to the design specifications. An important part of this procedure was the collection of field data on the settlement of the road so that more accurate predictions of the final settlements under the additional loads could be made.

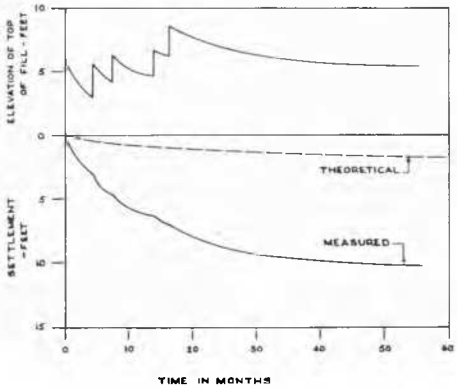

FIG. 7. Time settlement curve for chainage 6600.

THE FIELD OBSERVATIONS

Primary Consolidation

The results of the settlement observations at chainage 6600, where the settlements have been in excess of 10 ft., is shown in Fig. 7. An initial layer of fill, 6 ft. in thickness, was placed and this was followed at intervals of some months by additional layers until a total thickness of 15.5 ft. had been built up and the temporary wearing surface was laid. In the meantime the berms had been formed on either side of the road to permit the placing of the full thickness of fill. It will be noted that out of a total settlement of 10.2 ft. in 56 months, 7 ft. occurred in the 17-month construction period.

When the value of the coefficient of the clay consolidation, as determined in the standard oedometer test, is used together with the assumptions of one-dimensional consolidation and double drainage, the calculated rate of settlement is found to be very much slower then that shown by the field results. The time settlement curve calculated on the above basis is shown in Fig. 7 as the "theoretical" curve and the large difference between it and the field curve may be seen.

An examination of a large number of field settlement curves from the Western Approaches indicates that after about 40 months the primary settlements were complete.

If one-dimensional consolidation is assumed, the field settlement rates correspond to a coefficient of consolidation of about 15 sq.ft. per month or 50 times greater than the laboratory values.

If the width of the berms is taken into account, the ratio of the loaded width to the thickness of clay is about 4:1, while, for the road embankment alone, the ratio is 2:1. In these circumstances the three-dimensional effect would not be expected to be very large unless the horizontal permeability of the clay was very much greater than the vertical permeability. In spite of the inconclusive laboratory tests it would appear that this was in fact the case. It is well known that the field settlement rates are often more rapid than predicted from laboratory data but, in this case, the magnitude of the difference was larger than had been expected. On the other hand, however, the calculated settlements during primary consolidation appear to agree with those predicted on the basis of the laboratory tests. Some of the field settlement data have been superimposed on the design curves shown in Fig. 6. The results from chainages 5700 and 5900, from the central portion of the Western Approaches where the value of $C_c/1 + e_0$ is 0.2, show close agreement with the predicted values. The results from chainages 5600, 6600, and 6700 near the edges of the valley where the clays are more organic, also fit in well with the predictions.

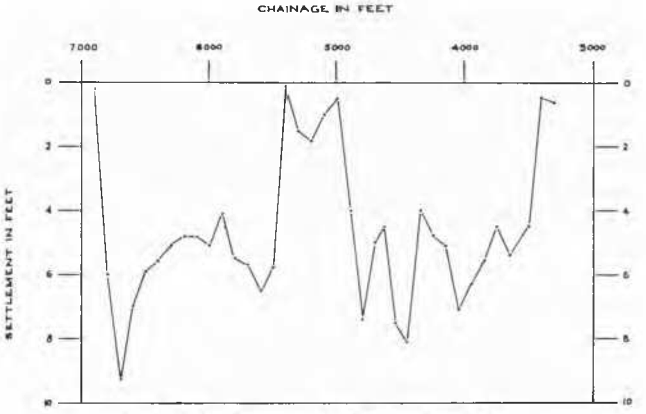

FIG. 8. Settlements during construction.

In Fig. 8 the settlements that took place during the construction period are shown and these conform to the soil conditions illustrated in Fig. 2. The relatively higher compressibility of the clays towards the edges of the buried valley beneath the Western Approaches is clearly shown and the influence of the ridge of hard ground beneath the creek is apparent from the relatively small construction settlement at chainage 4300.

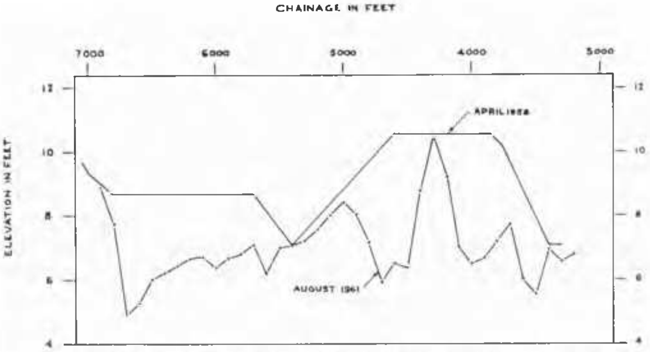

FIG. 9. Road profiles in April, 1958, and August, 1961.

Fig. 9 shows the change in road profile between April, 1958, when the road was surfaced, and August, 1961. The general pattern of the settlements during construction is repeated but, rather surprisingly, no further settlements over the ridge at chainage 4300 have occurred. The relatively thin layer of the clay in this area was fully consolidated during the construction period.

Secondary Consolidation

Although the time that has elapsed since the end of the period of primary consolidation is too small to allow very accurate deductions about the field rates of secondary consolidation to be made, it is nevertheless useful to make some preliminary comparisons.

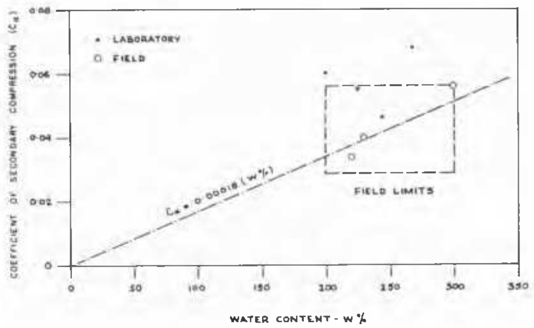

FIG. 10. Relationship between C_α and water content.

In Fig. 10, the field values of C_α have been plotted against the average water contents of the clay found in the borings for the three complete water content profiles available. All the other field data fall within the range indicated by the rectangle enclosed by the dashed lines. The laboratory values are also shown on the diagram. It will be seen that the field data agree quite well with the empirical relationship, $C_\alpha = 0.00018\ w$ per cent.

The ratios of the secondary settlement per log cycle of time to the primary consolidation computed for the field data are found to lie between the limits 0.11 and 0.32, compared with the laboratory range of 0.19 to 0.34. Although these limits are fairly wide, the laboratory measurements can give a useful guide to the field behaviour of these highly compressible clays.

The field settlement curves and the data on the magnitude of the secondary consolidation have been used to predict the further settlement of the road following the next filling operation. Without this field data the task of prediction would have been extremely difficult.

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