Settlement of fills on soft clay with vertical drains

Tassement des remblais sur des argiles avec drains verticaux

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SYNOPSIS

The settlement behaviour of wide road fills on deep soft silty clay is discussed. The fills have been in place for more than five years and the paper describes the results of settlement and pore pressure observations during this time. Because of the uniformity of the silty clay over large areas it has been possible to compare adjacent areas with and without prefabricated vertical drains. The author contends that there is still considerable uncertainty associated with superficially straightforward settlement problems of this kind and discusses the principal problems.

INTRODUCTION

The design of surcharging and vertical drain schemes in geotechnical engineering requires a basic understanding of the consolidation process. It is assumed that the essential soil parameters may be reliably determined for use in various theoretical settlement solutions that abound in the geotechnical literature. The author shows that the method by which the time to achieve a given degree of consolidation is determined can significantly affect the magnitude of computed settlements.

FILL CONSTRUCTION

The construction of wide clay fill road embankments over deep soft alluvium for a major freeway in Melbourne provided an opportunity to study the consolidation process in some detail. Fills were constructed along the freeway alignment over different periods because of delays in property acquisition. The two main areas discussed in this paper are designated A, centred on chainage 7120 m, and B, centred on chainage 6830 m. Field measurements at Area A have been previously published (McDonald and Cimino, 1984) giving results up to early 1983. At this location the surcharged fill is about 40 m wide with 2 (horizontal) to 1 side batters. Surcharge was placed 2 m above design level. An entry ramp abuts the main fill on the south side. The fills were slightly crowned for drainage, with an average height of 3.5 m and a unit weight of 20 kN/m^3. The mid-construction date for Area A was taken to be 10th December, 1978.

Fig 1 is a plan of Area B showing investigation and instrumentation positions and the areas where vertical drains (wicks) were installed in July 1983. Wicks were installed on a triangular array with transition zones on each side of the main area. The wick comprised an open polypropylene core wrapped in a light geotextile, with a 100 mm x 6 mm cross-section. Installation was achieved using a 160 mm diameter hollow steel mandrel containing the wick thrust into the soil to the design level of -17.5 m. Fill height at Area B was also 3.5 m and the mid-construction date was 1st November, 1981.

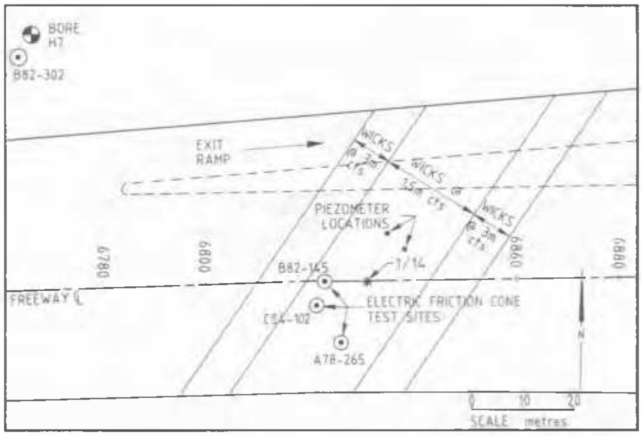

Fig. 1 Plan of Area B

Instrumentation consisted mainly of surface settlement markers and air pneumatic piezometers installed at various depths within the silty clay.

SOIL PROFILE

The soil profile at both of the above areas is nearly identical except that, at Area B, a naturally occurring fine loose sand layer between one and two metres thick exists above the compressible soft silty clay stratum. Fig 2 shows the soil profile at Area B. The silty clay is locally known as Coode Island Silt (CIS). Typical physical properties of the CIS obtained from Area B are shown in Table 1.

CONSOLIDATION BEHAVIOUR AND FIELD SETTLEMENT PREDICTION

In calculating the coefficient of consolidation (c_v), the square root of time method (Taylor,1948) is often

TABLE I

PHYSICAL PROPERTIES OF COODE ISLAND SILT

Property	Range	Average
Liquid Limit (%)	68 - 107	81
Plasticity Index (%)	35 - 81	53
Moisture Content (%)	55 - 89	71
Unit Weight (kN/m^3)	14.9 - 16.7	15.7

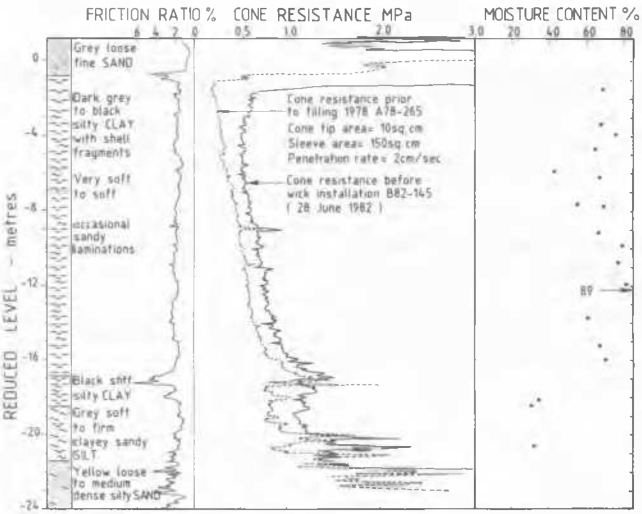

Fig. 2 Soil Profile

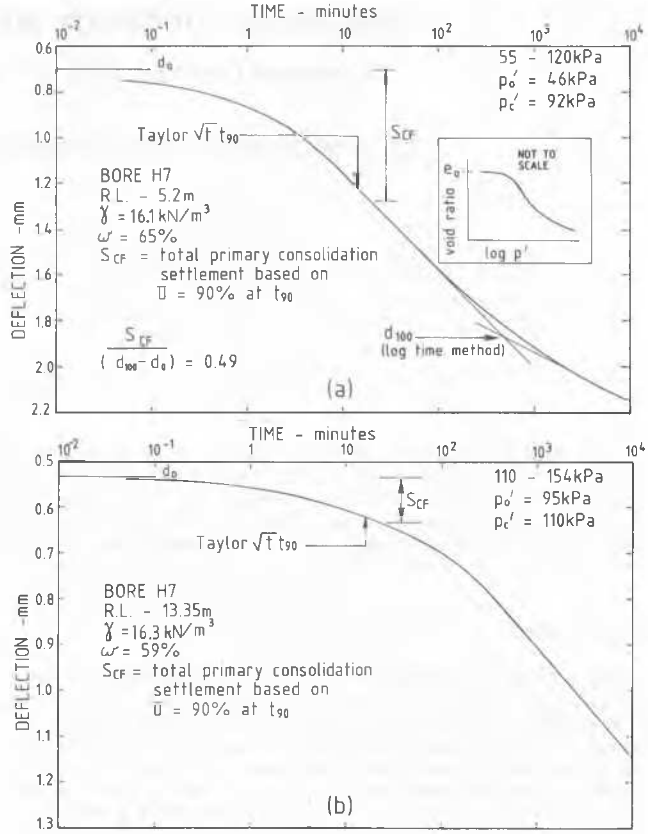

Fig. 3 Laboratory Consolidation Curves

preferred over the log-time method as the former method usually gives higher values. This leads to shorter predicted consolidation times, in closer agreement with field behaviour. In practice neither method may give reliable field time predictions, possibly because mass drainage characteristics are not properly represented in small samples.The square root method implies that ninety per cent consolidation settlement has occurred at the t_{90} time obtained by graphical construction. The total primary consolidation settlement for an increment may be taken to be 1.11 times the settlement between t = 0 and t_{90}. This procedure may lead to a significant under-estimate of primary consolidation settlement compared with that obtained using the log-time construction see Fig 3(a). Large changes in c_v were observed during the laboratory programme, especially for load increments straddling the apparent pre-consolidation pressure.

Re-construction of "field" consolidation curves using the Schmertmann (1955) procedure was not attempted because of the curvature of the laboratory void ratio vs pressure curves (see inset Fig.3(a)) and the fact that the curves usually did not extend sufficiently far to permit the appropriate construction. Fig 3(b) illustrates another problem sometimes encountered. The end of primary consolidation could not be determined using the log-time method even though the increment was maintained for seven days. If field behaviour was similar, predictions using the square root of time interpretation would seriously underestimate settlements. Measurement of pore pressure during consolidation tests may help to clarify this problem although many laboratories are not equipped for such testing.

Pre-construction estimates of settlement at Area B gave a consolidation settlement of about 850 mm using the square root of time interpretation for laboratory consolidation tests on 75 mm diameter x 19 mm thick specimens. New samples were tested from bore H7 and a revised settlement estimate prepared, based on the log-time curve interpretation. For the few cases where the end point of primary consolidation could not be determined, this was arbitrarily taken as the seven day compression. Unfortunately the bore revealed stiffer strata below level -7.8 m than at other locations. Nevertheless, the calculated consolidation settlement of 840 mm suggests that the original estimate was far too low.

FIELD OBSERVATIONS

Settlements

Fig. 4 shows the fill centreline settlement with time at three adjacent areas. Total settlements were only recorded at Area A (CH 7120). At the adjacent locations CH 7173 and CH 6830 the settlements occurring before markers were placed were assumed equal to those recorded at CH 7120 over the same period since the mid-construction time.At CH 7173 an average depth of 1.7 m of surcharge was removed over a 40 m length of fill. After an initial rebound of about 25 mm the remaining 1.9 m high fill began to settle again at an average rate of about 20 mm per year. After seven months the surcharge was replaced.

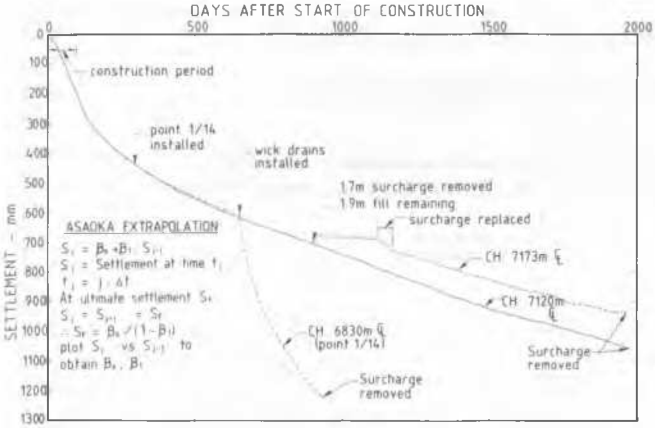

Fig. 4 Field Settlement Behaviour

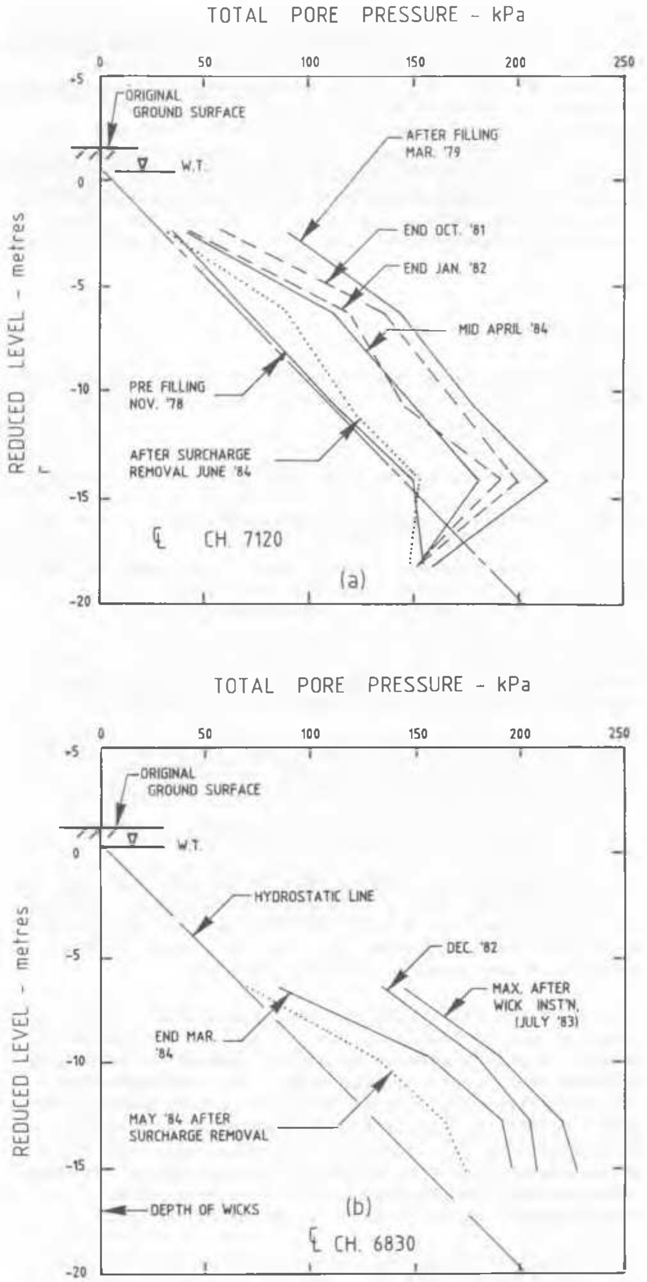

Fig. 5 Pore Water Pressures Beneath Fills

The dramatic increase in settlement rate after wick installation at Area B (CH 6830)demonstrates the effectiveness of the wicks in accelerating consolidation.

Settlement extrapolation at Area A was attempted using the Asaoka (1978) procedure. Asaoka indicates that large time increments are preferable and using a value of 200 days, the author obtained an ultimate settlement of about 1960 mm. A similar extrapolation was attempted for the Area B settlements after wick installation using time increments of 20 and 40 days. The ultimate settlements (including those prior to wick placement) predicted were 1700 mm and 1540 mm respectively. The sensitivity of the predicted ultimate settlement to time interval adopted indicates that, for certain practical cases, the Asaoka method may not be applicable.

Pore Pressures

Pore pressures in the CIS below the embankments at Areas A and B are shown in Figs. 5(a) and 5(b) respectively. The lower than hydrostatic pore pressure at level -18 m indicates flow from the lower strata although no large scale water withdrawal by pumping is practised in this or nearby areas. In December 1981, six 100 mm diameter vertical sand drains were installed through the CIS near the piezometers at Area A using a drilling rig. The mid-layer piezometers exhibited an initial drop of almost 20 kPa although the early high rate of dissipation (relative to the pre-existing rates) did not continue. Pore pressures at Area B initially increased by up to 19 kPa, during wick installation. A relatively rapid rate of dissipation followed this rise, especially towards the top of the CIS. Upon surcharge removal (2 m) at Area A, mid-plane excess pore pressures reduced by almost 30 kPa, whilst at Area B, 1.35 m of surcharge removal reduced mid-plane pore pressures by 27 kPa.

Degree of Consolidation

Field pore pressure measurements at Area A before sand drain installation were used to estimate the average degree of consolidation ($\bar{U}$) according to simple consolidation theory. The corresponding values of $\bar{U}$ and settlement at various times produced c_v values of between 2 and 5 m²/year with consolidation settlements of about 2 m. More complicated analysis methods accommodating variable soil properties are available. However, determination of the appropriate parameters presents a formidable task involving numerous assumptions, which reduce confidence in the resulting time-settlement predictions.

$\bar{U}$ can also be estimated from the friction cone data shown in Fig. 2. If the ratio of undrained shear strength to effective overburden stress remains constant, as does the ratio of cone resistance to undrained strength, then equation (1) may be used to estimate the degree of consolidation (U) at any level, and by integration with depth, $\bar{U}$ for the entire layer.

$$U = \Delta p/q = (p_o'/q).(q_{ct}/q_{co}-1) \qquad (1)$$

where q is the total vertical stress increase, Δp is the effective stress increase due to consolidation, p_o is the initial effective overburden stress and q_{co} and q_{ct} are the cone resistances measured before filling and after some time, t, respectively. $\bar{U}$ was found to be about 40 per cent, giving a c_v of 12 m^2/year and a consolidation settlement of about 1 m, clearly an underestimate. Therefore, it would appear that friction cone data may only be used qualitatively in this respect. There was no further increase in cone resistance at position C84 - 102 in Area B following an additional 790 mm of settlement after cone test B82-145 had been performed, indicating considerable disturbance due to wick drain installation.

VERTICAL DRAIN DESIGN

Vertical drain design was performed using the theoretical solutions of Hansbo (1981) given by equations (2) and (3).

$$t_h = (D^2/8\ c_h) \times \mu \times \ln(1/(1-\bar{U}_h)) \qquad (2)$$

$$\mu = \ln(D/d_s) + (k/k_s) \times \ln(d_s/d) - 3/4 \qquad (3)$$

where D is the diameter of the zone of influence of the drain, c_h is the coefficient of consolidation in a horizontal direction, $\bar{U}_h$ is the average degree of consolidation by radial drainage alone, t_h is the time required to achieve $\bar{U}_h$, d is the equivalent wick drain diameter, and d_s is the diameter of the disturbed zone around the wick due to the installation process.

Consolidation times should not be influenced greatly by resistance to flow within the wicks provided the discharge capacity of the wick satisfies equation 4.

$$Q_{drain}\ (\text{litres/hour}) > \dot{S} \times a^2/10115 \qquad (4)$$

$\dot{S}$ is the expected settlement rate in mm/year and a is the drain spacing (m) for a triangular wick array. Reduced soil permeability in a disturbed zone around the drain may significantly increase the time to achieve a given degree of consolidation, as shown in Fig.6.

Because of the difficulty in assessing correct field values of the variables s, k, k_s and c_h it is common to embody them in a reduced c_h value, clearly an extremely difficult task at the design stage. The numerous silty sand laminations and lenses observed in CIS during site investigations indicated that appreciable permeability anisotropy could be expected. The calculated time to achieve ninety per cent consolidation by radial drainage alone was about 5 months, with an equivalent wick drain diameter of 50 mm and a c_h value of 5m^2/year.

Field settlements and pore pressure at Area B indicate that consolidation was far from complete nine months after wick installation, suggesting that the effects of disturbance due to installation may be considerable.

CONCLUSIONS

For the case of large settlements of fills on deep soft clay, predicted settlements can vary appreciably, depending on the particular interpretation of laboratory consolidation data adopted. For this case, the use of unmodified laboratory consolidation curves together with the log-time graphical procedure would seem to give the best estimate of field settlements.

Changes in the coefficient of consolidation during consolidation invalidate the use of simple one-dimensional

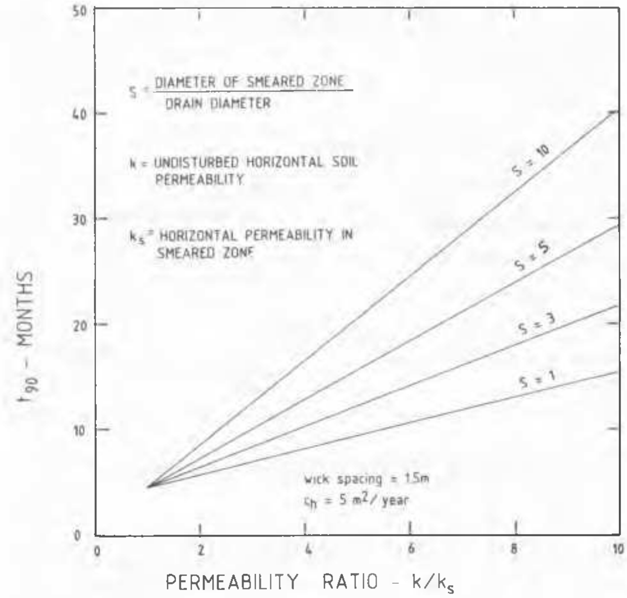

Fig. 6 Effect of Disturbance on Wick Drains

theory based on observed field pore pressures and settlements. These effects are particularly pronounced when loading beyond the apparent pre-consolidation pressure.

The design of a vertical drainage system is a simple theoretical but practically difficult task. Unless there exists sound field data regarding consolidation times in thick clay layers, it would be prudent not to assume significant permeability anisotropy. The detrimental effects of disturbance may be considerable and limit the theoretically desirable wick spacing. The author hopes that the above case study is useful in putting some of the practical design problems in perspective.

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