Settlement of Low Embankments on Thick Compressible Soil

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SUMMARY: The problems involved in applying conventional consolidation theory to compression of a thick bed of estuarine silty clay beneath a low road embankment are described. Settlements and pore pressures beneath the embankments are presented together with typical laboratory consolidation data. It is shown that important field settlement characteristics are not duplicated in the laboratory. In particular, whilst in the laboratory the rate of settlement decreases continuously with time, in the field a point is reached where a constant rate of settlement obtains. Field data is presented showing that small but significant rates of settlement remained in an area where surcharge was removed after 27 months of loading.

1 INTRODUCTION

Practising geotechnical engineers generally believe that, whilst the rate of settlement cannot be reliably predicted using normal one-dimensional consolidation theory, the magnitude of settlement can be estimated reasonably well. However, in the case of highly compressible soils exhibiting long-term settlement characteristics, settlement observations are frequently discontinued after the particular facility is pressed into service. With road embankments, settlement markers are often obliterated during pavement construction and seldom replaced. In cases where long-term settlements continue, corrective action is usually necessary to maintain serviceability, especially adjacent to bridge structures where there would otherwise be a sudden change in level between the structure and the roadway founded on embankment.

There is therefore a need for geotechnical engineers to be informed of case studies involving large long-term settlements, so that improved design and construction techniques can be developed, resulting in better serviceability of road embankments on compressible soils.

The case study presented here is concerned with a major freeway project in Melbourne. Because of uncertainties associated with the prediction of settlement and rate of settlement, all embankments were surcharged 2 metres above the final pavement level. This paper focuses on the behaviour of the surcharged fills where settlements have been observed for almost five years. The problems involved in a "simple" one-dimensional settlement analysis, with modification to take into account secondary compression, are discussed in the light of the observed settlement behaviour.

2 SITE

2.1 Historical Background

The West Gate Freeway through South Melbourne traverses thick Quaternary estuarine sediments of the Yarra River delta. The geology of the area has been described in some detail by Neilson (1976).

The stratum of greatest significance with respect to embankment settlement is a slightly organic silty clay locally known as Coode Island Silt, which was deposited about 9000 years ago. Sandy partings and lenses occur frequently throughout this silty clay and numerous marine shell and shell fragments exist.

During the early stages of the project the sand lenses were believed to be sufficiently interconnected to give the silty clay a very much larger permeability in the horizontal direction, compared with the vertical permeability. Walker and Morgan (1977) indicated that the ratio of horizontal to vertical permeability could be as large as 200.

Figure 1 - Area and site location plan

The Coode Island Silt (CIS) has historically been associated with settlement problems in the area. Bridge approach embankments previously constructed over the CIS have continued to settle appreciably for decades. In several instances when demolishing old industrial buildings on the freeway alignment, as many as three complete floor slabs have been discovered at successively lower levels. Moore and Spencer (1969) produced data showing long term settlements of about 800 mm below a two story building on the CIS, imposing an average stress of 46 kPa. Regional settlements near the test area - see Figure 1 - have been observed to be about 10 mm per year, although there is considerable variation. Before freeway construction, this area was used for ship container and lumber storage and in one area,
substantial stacks of scrap metal existed. This information serves to illustrate the complicated nature of the settlement problem presented to the Road Construction Authority's geotechnical engineers.

Figure 2 shows a cross section of the embankment area discussed in this paper together with the location of various instruments and settlement markers used to monitor behaviour. The average height of the surcharged cohesive fills is 3.5 m. Construction commenced in November 1978 and was essentially completed over a three month period.

![Figure 2 - Embankment and instrumentation](image)

Coode Island Silt is a CH soil and typical physical properties in the area under consideration here are shown in Table I.

<table>
<thead>
<tr>
<th>TABLE I</th>
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<tbody>
<tr>
<td>PHYSICAL PROPERTIES OF COODE ISLAND SILT</td>
</tr>
<tr>
<td>Property</td>
</tr>
<tr>
<td>Liquid Limit</td>
</tr>
<tr>
<td>Plastic Limit</td>
</tr>
<tr>
<td>Moisture Content - %</td>
</tr>
<tr>
<td>Percent finer than 2 µm</td>
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<tr>
<td>Percent finer than 75 µm</td>
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<tr>
<td>Bulk Density - t/m³</td>
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The soil profile at the site is shown in Figure 3. An electrical friction cone penetrometer (CPT) having a projected tip area of 10 sq cm and a friction sleeve area of 150 sq cm was used to obtain the resistances. A Geonor vane was used to determine the undrained shear strength of the CIS. The average sensitivity was about 5 whilst the ratio of static cone resistance to vane shear strength was 12. Menard and Self Boring Pressuremeter tests were also performed near the site. Undrained strengths determined by the pressuremeter were generally lower than the vane shear strengths. An average static cone resistance to pressuremeter undrained strength ratio was 15. Beneath the CIS is a layer of firm to stiff silty clay about 6 m thick, which is in turn underlain by very stiff sandy clays and dense gravels.

![Figure 3 - Soil profile](image)

3 SETTLEMENT ANALYSIS

In a conventional analysis incorporating an allowance for secondary compression, the following relationship is commonly used:

\[ \rho_T = \rho_I + \rho_P + \rho_S \]  

(1)

where \( \rho_T \), \( \rho_I \), \( \rho_P \), and \( \rho_S \) are the total, immediate (undrained), primary consolidation and secondary consolidation settlements respectively.

3.1 Immediate Settlement

The immediate settlement is usually estimated using elastic theory with an average undrained soil modulus. Foot and Ladd (1981) concluded that a value of \( E_s/\gamma_0 \) of 500 was sufficiently accurate to estimate immediate settlement, where \( E_s \) and \( \gamma_0 \) are the undrained soil modulus and cohesion respectively.

The authors found that widely differing results for immediate settlement could be obtained depending on the idealised geometry assumed, the theoretical solution adopted and the method of evaluating an equivalent homogeneous modulus. The centre settlement occurring during embankment construction (3 month period) was measured to be 200 mm. However, this is believed to be mainly due to relatively rapid initial consolidation rather than undrained movement. In similar nearby areas where fills were constructed more quickly, the centre settlements occurring during construction were between 25 mm and 60 mm.

3.2 Primary Consolidation Settlement

To obtain data for early analyses, standard consolidation tests on 75 mm diameter, 19 mm thick samples were performed, with load increment times dictated by the time to achieve 95% percent consolidation on a Taylor (1942) square root of time graph. Settlement was initially estimated using the relation

\[ \rho_p = \sum_m \Delta P \Delta H \]  

(2)

where \( m \) is the modulus of compressibility, \( \Delta P \) is the stress increase due to embankment loading and \( \Delta H \) is the thickness of each layer within the consolidating strata. Average \( m \) values were selected from plots of \( m \) against consolidation pressure but there was found to be a large
variation over the critical pressure range. Consequently, little confidence was felt in the primary consolidation settlement value of 1.0 m.

A further calculation was performed in which the primary consolidation settlement in each layer was computed directly from the laboratory compression vs. pressure plots over the relevant stress range. A value of 0.7 m resulted from this calculation. The authors believed this second estimate to be less subject to the errors inherent in averaging variable $m_Y$ values but were still concerned at the goodness of the estimate in view of the laboratory compression behaviour.

The consolidation behaviour of a typical sample of the CIS is shown in Figure 4. Note the dramatic change in compressibility and coefficient of consolidation ($c_y$) over a narrow range of pressures.

Figure 4(c) shows settlement rates (velocities) for pressures above and below $p_A$ plotted against time, on double logarithmic scales. Parkin (1981) has proposed this method as an alternative procedure for evaluating the coefficient of consolidation ($c_y$) and other consolidation characteristics. The theoretical one-dimensional velocity curves matching the experimental curves as nearly as possible are also shown on Figure 4(c). These curves illustrate a significant departure from theory as a result of secondary compression. The authors found that $c_y$ values determined using the velocity method were similar to those obtained using the Taylor square root of time method.

In some cases the compression vs. log time curves were not of the more common Type I ($L$, 1961) or S shape, especially for pressure increments straddling the apparent pre-consolidation pressure. Others have encountered soils displaying similar characteristics, with very large compressions attributed to collapsing structures in which the assumptions of the usual one-dimensional consolidation theory are invalidated e.g. Gorden (1968).

Secondary consolidation settlement was estimated using the following relation:

$$ \rho_S = \Sigma [C_o \Delta H \log_{10} (t/t_0)] $$

in which $t_0$ is some arbitrary reference time, usually taken at the end of primary consolidation.

Although secondary compression occurs simultaneously with primary consolidation, it is virtually impossible to separate the two processes. However, in calculating primary consolidation compressibility, secondary effects occurring before the end of primary consolidation are automatically accounted for and there is therefore no inconsistency in taking $t_0$ to be the time at which primary consolidation is practically complete. $C_o$ is the Coefficient of Secondary Compression, defined as the compression occurring over one logarithmic cycle of time divided by the sample thickness.

Consolidation tests conducted to determine $C_o$ over the stress range $p_A$ to $p_r + \Delta p$ (where $\Delta p$ varied from 60 kPa to 70 kPa) gave values between 0.011 and 0.019, with an average value of 0.014. Assuming negligible secondary compression within the first year, a value of about 150 mm may be calculated between one and five years after construction.

In some cases the slope of the compression vs. log time curve was not linear beyond $t_{90}$ (the time for ninety per cent primary consolidation settlement to occur) in which case the adoption of a single $C_o$ value is not strictly correct. Using the velocity plotting method, non-linear secondary compression vs. log time curves are indicated by a departure from a 45 degree slope beyond $t_{90}$. The departures observed during the laboratory testing programme were generally not greater than 3 to 4 degrees (flattest slopes than 45 degrees) and it was therefore concluded that the linear $C_o$ approximation was acceptable.

4 FIELD BEHAVIOUR

4.1 Settlements

Post-construction settlements at three points across the embankment are shown in Figure 5. As previously discussed, it would appear that immediate undrained settlements are insignificant in this case, when compared with consolidation settlements.

In this context it should be noted that lateral soil movements away from the toes of the fills have been quite small at all stages during and after construction. A simple calculation of the areas bounded by inclinometer tube deflection curves and vertical settlement curves may be performed to assess the amount of settlement attributable to lateral soil movements. In the six month period until the end of 1982, lateral soil movements accounted for about 5 percent of the vertical settlement measured over the same period.
There is some question as to whether thick compressible soil deposits should be treated as single layers and consequently the applicability of graphical techniques such as the velocity method in assessing stages of the field settlement process.

However, if it is accepted that the CIS can be treated as a single thick consolidating stratum then the velocity method certainly indicates significant differences between field and laboratory behaviour - refer to Figure 6.

According to one-dimensional theory, the velocity plot should be a straight line having an angle with the horizontal of about 27 degrees up to an average degree of consolidation of 50 percent. Thereafter the curve steepens although it has already been shown that high secondary compressions exhibited by CIS result in elevated settlement rates above those predicted by Terzaghi theory during the latter stages of primary consolidation. The field velocity curve for the central fill settlement marker lies approximately on a 27° slope with the horizontal up until about two years after the mid construction time (the time origin for the plot). Beyond the two year period the settlement rate remained constant, unlike the laboratory behaviour which showed a steadily diminishing settlement rate with time. No additional loading occurred during this time. It is interesting to note that a similar phenomenon was reported by Chang et al. (1973) for low fills on soft organic glacial and post-glacial clays in Sweden. If their settlement data is replotted using the velocity procedure, it is seen that the rate of settlement was almost constant for a period of one to six years after construction, before reducing with a final velocity curve slope of about 45 degrees.

The two velocity curves for settlement markers towards the edges of the west gate fills do not bear any resemblance to the theoretical curve, both being flatter than the minimum theoretical value of 27 degrees. Possibly, shear deformations towards the edges of the fills are responsible for this characteristic.

Settlements calculated on the basis of one dimensional theory significantly underestimate the field values. If it was assumed that all primary settlement was complete within five years of construction, a total compression of about 910 mm would be predicted. The fills however had settled about 1000 mm by this time but the rate of settlement at the centre of the embankments was constant at about 120 mm per year. It is still not possible to confidently extrapolate a settlement time curve. Consequently, it is extremely difficult to determine an optimum time for removal of the surcharge. The procedures for designing an effective surcharging programme for precompression of the subsoil have been well documented by Johnson (1970). However, if the magnitude of settlement cannot be reliably estimated, the degree of consolidation cannot be assessed and reliance must be placed on pore pressure measurements to indicate an appropriate time for surcharge removal. A further complication is that the time - settlement characteristics of the surcharged fills are likely to be considerably different from those of unsurcharged fills where the subsoil stress increase is not sufficient to straddle the apparent pre-consolidation pressure.

4.2 Pore Pressures

Air pneumatic piezometers beneath the embankment indicated very slow excess pore pressure dissipation - see Figure 7. Pore pressures towards the centre of the CIS continued to rise by a small amount for about seven months following construction. Such high excess pore pressures would normally indicate that primary consolidation was far from complete. Significant excess pore pressures also exist beneath and beyond the edges of the fills. These observations suggested that the CIS permeability was particularly low and that it was unlikely that the horizontal permeability was appreciably greater than the value in a vertical flow direction at these stress levels.

Attempts were made to relate settlements and the average degree of excess pore pressure dissipation over successive time periods using laboratory determined soil compressibilities but this proved to be unsuccessful. The lack of correspondence between pore pressure dissipation and settlement has been reported by others, even when advanced finite difference analyses have been performed with variable consolidation parameters depending on the effective stress state e.g. Taylor et al. (1975).
A further point of significance was the pore pressure observed at a depth of 20 m. The initial total pressure registered by the piezometer at this depth was about 30 kPa lower than a projected value based on the higher level piezometer readings prior to filling. This behaviour indicated flow from the lower strata and was confirmed by piezometric pressures measured at other locations and at similar depths.

4.3 Effect of Surcharge Removal

As it was not possible to reliably estimate a degree of consolidation on the basis of field measurements of settlement and pore pressure, the surcharge was removed from an area of embankment near the main instrumented area 27 months after construction. However, a study of the subsequent settlement behaviour of the embankment with surcharge removed could provide the best information on likely in-service performance.

The surcharge was removed over a distance of almost 50 m. The depth of surcharge removed varied between 1.6 m and 2.1 m with an average of 1.8 m. The average remaining fill height was about 2.0 m at the centre of the embankment. During surcharge removal, rebound was measured to be 20 mm. Settlement measurements conducted over an ensuing period of 7 months showed that the average centre of fill settlement rate had decreased from about 120 mm per year (prior to removal) to 20 mm per year. Surcharge was replaced after this time and the rate of settlement measured over the following two and a half months was about 115 mm per year. It should be noted that these rates are total values without adjustments for pre-existing regional settlement rates (measured at about 10 mm per year in this area).

Unfortunately there were no nearby areas where unsurcharged fills had been constructed to enable a more precise evaluation of the effects of precompression in limiting in-service settlements.

The authors are of the opinion that for soft, compressible soils of low permeability the maximum amount of surcharge possible should be used, consistent with stability requirements. This usually involves exceeding the apparent pre-consolidation pressure of the soil and large settlements may be expected. The low permeability of the soil may retard the settlement process to such a degree as to limit the practical utility of the surcharging techniques, to improve foundation soils within a reasonable period of time. In this case the presence of numerous more permeable silt and sand lenses does not appear to have increased the mass permeability to the extent that surcharging alone would be able to produce the desired over-consolidation effect within a satisfactory period. Deep vertical drains provide a viable method of accelerating settlements within realistic project times and should therefore be used in conjunction with surcharging where strict in-service settlement tolerances must be satisfied.

5 CONCLUSIONS

Coode Island Silt in the area investigated is a highly compressible silty clay exhibiting large secondary compressions when loaded not far above the existing overburden pressure. These compressions may be large enough to obscure the end of primary consolidation of laboratory samples using some common graphical procedures.

Beneath wide embankments on the CIS, settlements appear to be occurring in a manner incapable of analysis using conventional one-dimensional settlement theory, even with modification for secondary compression. Field rates of settlement may remain constant for many years although a similar phenomenon was not observed with laboratory samples loaded over an identical stress range.

The velocity method of Parkin is a simple and convenient technique for determining whether one-dimensional theory may be used to analyze settlements.

Significant excess pore pressures persisting long after embankment construction may not be used in a conventional analysis to estimate the degree of primary consolidation or the time for consolidation.

One dimensional theory underpredicts the magnitude of field settlements considerably. At present the rate of settlement is essentially constant and it is not possible to reliably estimate long term settlements.

Surcharging may be used to improve the compressibility characteristics of soils. However, if the soil permeability is low, surcharging may not produce the desired over-consolidation effect within a reasonable period of time. In such cases measures to accelerate pore water drainage, such as vertical drain installation, should be used in conjunction with surcharge. The authors found that significant rates of settlement still occurred after removal of a 20 per cent surcharge from a 2 m high fill after almost 2½ years.

6 ACKNOWLEDGEMENTS

The authors gratefully acknowledge the efforts of technicians and West Gate Project personnel who have contributed in providing the information necessary for this paper. Out thanks are also extended to Dr A. Parkin of Monash University who has assisted in the interpretation of settlement information. The paper is presented with the permission of Mr T.H. Russell; Chairman and Managing Director, Road Construction Authority of Victoria. The views expressed are those of the authors and do not necessarily reflect the views of the Road Construction Authority.

7 REFERENCES


