

Settlement Characteristics of Lateritic Soils near Worsley

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SUMMARY. The lateritic soils near Worsley, some 200 km south of Perth, Western Australia, are characterised by low Atterberg limits, low dry densities, high moisture contents, very high shear strengths and low compressibilities. The stress-strain curves measured by Camkometer tests are linear to peak shear stresses up to 800 kPa. These stresses are mobilised at relatively low strains. The engineering properties reflect the significant influences of the iron and aluminium cementitious oxides, that underpin the open-voided and brittle structure of these soils. A trial embankment was constructed to apply a foundation loading of 200 kPa. The natural soils were instrumented to measure inter alia settlement and pore water pressures. Settlement occurred concurrently with embankment construction. Linear elastic theory is suitable for predicting settlements because the shear stresses generated by the foundation loadings would be less than the average peak shear strengths of the supporting soils. The accuracy of settlement predictions depends on the techniques used to measure the compressibility parameters. Pressuremeter and large scale load testing provide less conservative estimates of compressibility than oedometer and triaxial testing. Semi-empirical correlations between the compressibility parameters and dry densities are presented and are discussed in a foundation design context.

1 INTRODUCTION

The unusual engineering properties of the lateritic soils at the Alumina Refinery Site near Worsley, located some 200 km south of Perth, and the significance of geological and environmental influences on their genesis have been presented in separate papers (Gordon & Smith, 1984, and Smith, 1984). This paper summarises the salient results of the above papers and discusses the settlement characteristics of lateritic soils by presenting a case history of the settlement monitoring of an earthen embankment. The paper concludes with recommendations on the most suitable investigation methodology for foundation design in the lateritic soils of the Worsley environment.

The findings presented in the paper were obtained by Soil and Rock Engineering Pty Ltd during the geotechnical investigations for the Alumina Refinery near Worsley.

2 CASE HISTORY: - EMBANKMENT LOADING

2.1 Background

The author has discussed in a separate paper the problems associated with "undisturbed" sampling of lateritic soils from the Worsley environment, and has concluded that the soil strengths and stiffnesses (expressed by Moduli of Elasticity) as measured by laboratory testing, may underestimate the insitu properties by a factor up to 3 (Smith, 1984). To better quantify the settlement characteristics for use in foundation design, an earthen embankment was constructed. The settlements of the natural soils supporting the embankment were measured during and post construction. These settlements are presented below and are compared with the settlement predictions made using the results of Camkometer and oedometer tests.

2.2 Ground Conditions

The distribution of soil types over the site selected for the embankment loading is presented in Figure 1.

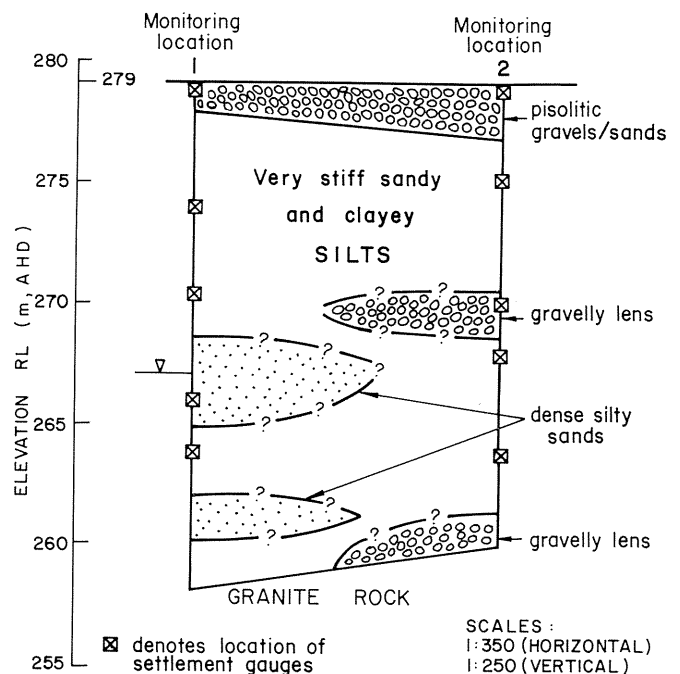


Figure 1. - Idealised subsurface section showing soil types and locations of settlement gauges

The above distribution of soil types is characteristic of the weathered profile derived from the Plateau weathering process of a quartz and feldspar-rich granite bedrock in a semi-tropical environment (Gordon, 1984 (b)).

Very stiff sandy and clayey silts, classified as ML and MH in accordance with the Unified Soil Classification, dominate the weathered profile. Bands of silty sands and gravels of medium dense to dense consistency are present throughout the profile. A dense sandy and pisolitic gravelly surficial stratum caps the clayey silts. Ground water table is present at a depth of 12m. Perched water was encountered in the investigation boreholes at 6m.

The different soil types are lateritised. They contain variable proportions of cementitious iron and aluminium oxides, that have precipitated during the leaching processes. Capillary leaching, a term used by the author to describe the upward movement of groundwater, the resulting dissolution and cation exchange processes, has been the dominant weathering and lateritisation mechanism. Capillary leaching has occurred in response to soil suction stress gradients, produced by seasonal evapotranspiration exceeding the precipitation. Gravity leaching has been sub-dominant in the Worsley environment (Gordon & Smith, 1984).

The engineering properties reflect the geotechnical consequences of leaching and the variable mineral composition of the bedrock. Dry densities as low as $1.05 \text{ tonnes.m}^{-3}$ attest to the efficacy of the leaching mechanisms. There are trends of decreasing dry densities, shear strengths and Moduli of Elasticity with increasing depth within the weathered profile, (Figure 2). These trends and the significant interactions of the geological and environmental influences on the geotechnical properties have been discussed in a separate paper (Gordon & Smith, 1984).

2.3 Settlement Predictions

Four investigation boreholes were drilled at locations 1 to 4 (Figure 4) and samples were taken in 62mm diameter, thin-walled sampling tubes for consolidation and index testing. Four Camkometer tests were carried out in separately drilled boreholes.

The Camkometer is a self boring pressuremeter that is capable of being inserted into a soil mass with minimal disturbance. Technical details on the Camkometer have been presented by Wroth & Hughes (1974).

The results of the field and laboratory testing are presented in Figure 2.

The moduli determined from the Camkometer tests were initial tangent moduli computed from the shear stress vs strain curves for strains up to 0.5%. An average Poisson's ratio of 0.4 was computed from the results of the Camkometer testing (Smith, 1984), and was used to compute the elastic moduli.

The moduli from oedometer tests were calculated from the recompression indices, derived from the laboratory void ratio vs effective stress relationships that were corrected for disturbance and overconsolidation influences (Casagrande, 1936 and Schmertmann, 1955). The moduli were then modified to take into account the embankment stresses, that were estimated from elasticity theory. The resulting moduli are plotted in Figure 2.

The ratios of the Camkometer to oedometer derived moduli vary from approximately 1.7 to 3.4. The

factors that contribute to the significantly higher insitu moduli are considered to be sample disturbance, soil variability and anisotropic influences (Smith, 1984).

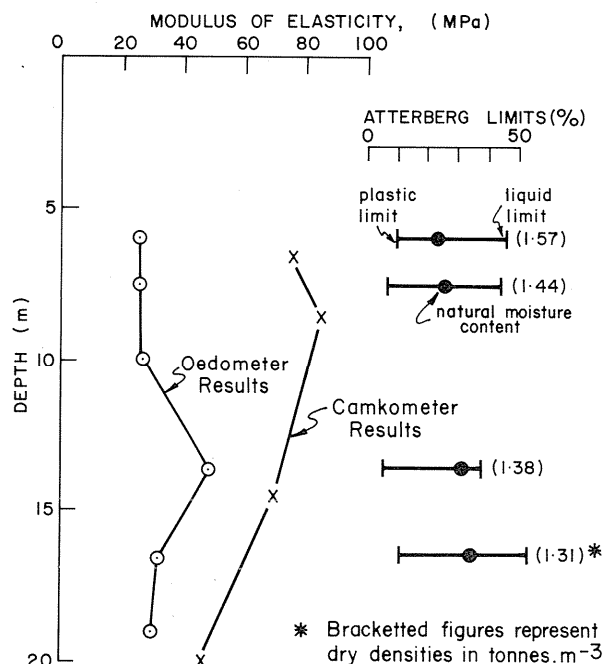


Figure 2. - Summary of test results, embankment soils

The shear stress vs strain relationships derived from three of the four Camkometer tests carried out at this site are presented in Figure 3.

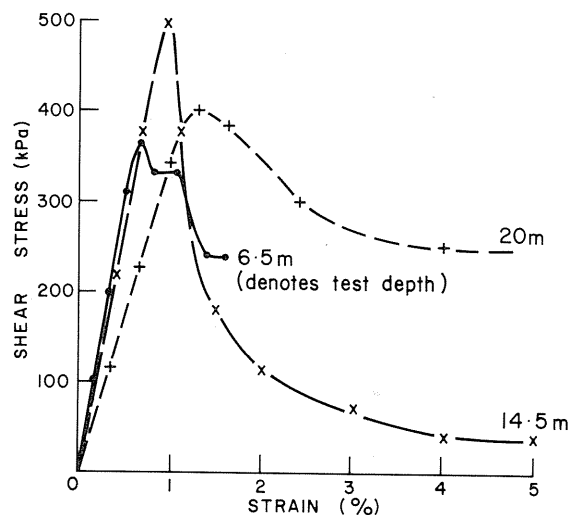


Figure 3. - Stress vs strain relationships derived from Camkometer testing

The salient features are the high and variable peak shear stresses mobilised at low strains, linear elasticity to the peak stresses and variable sensitivity to remoulding. In this context, sensitivity is defined as the ratio of the peak to residual shear stresses. For the above results, this ratio varies from 1.6 to 8.0. The upper limit is the maximum value determined from a programme comprising 14 Camkometer tests. The average sensitivity was 2.3 and the standard deviation was 1.9 (Smith, 1984).

The form of the shear stress vs strain relationships is not dissimilar to the constitutive properties of another brittle engineering material, namely cast iron. The analogy is significant in that the peak shear strengths are a function of the concentration of the cementitious iron and aluminium oxides, that have underpinned the leached fabric of the soil. A detailed discussion of the constitutive properties of these soil types has been presented in a separate paper (Smith, 1984).

The settlement of the embankment was estimated from elasticity theory to be 50mm using the Camkometer results and 110 mm from the oedometer results.

2.4 Embankment and Instrumentation

An earthen embankment, 100m long by 80m wide and 10m high, was constructed from lateritised silts and clays.

The ground supporting the embankment was instrumented with settlement gauges (magnetic probe extensometers), remote reading pneumatic piezometers and Gloetzl pressure cells, (Figure 4). The depths that the settlement gauges were installed are shown on the idealised subsurface section in Figure 1. The settlement gauges and pneumatic piezometers were installed in the ground investigation boreholes.

It was originally intended to measure the distribution of settlement throughout the weathered profile. This objective was not realised due to post installation problems. The settlements were measured by the settlement gauges installed immediately below the natural ground surface.

All instruments were read at regular intervals through out construction and thereafter until the readings had equilibrated.

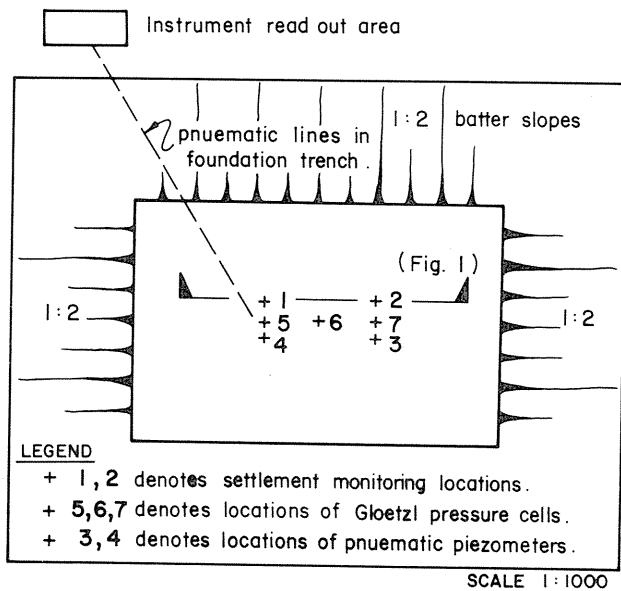


Figure 4. - Plan of embankment and monitoring stations

2.5 Results of Monitoring

2.5.1 Settlement:- Magnitude and Rate

The measured settlements of the natural ground are plotted against embankment construction and time in Figure 5. Maximum settlements of 60mm and 72mm were recorded at the two monitoring stations. The

reading accuracy for the magnetic probe extensometers, as given by the manufacturer, is ± 1 mm.

Settlement was essentially linear with applied loading and occurred concurrently with construction. At the completion of the embankment, the settlement was virtually completed.

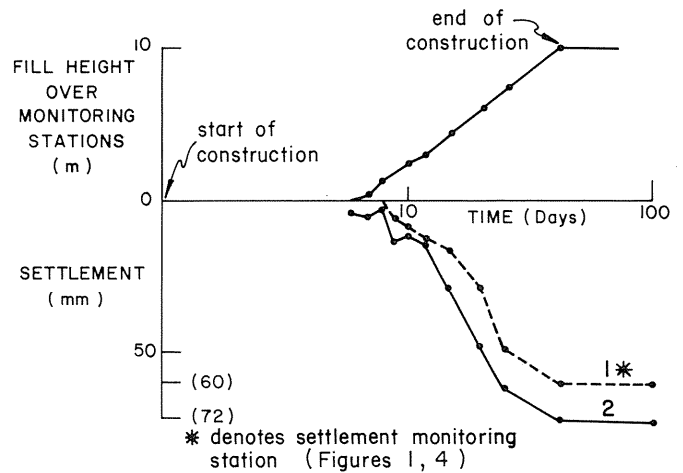


Figure 5. Results of settlement monitoring

2.5.2 Embankment pressure

The Gloetz pressure cells were installed in 0.5m deep trenches below the natural ground-embankment interface and gave maximum embankment pressures of 213, 214 and 271 kPa. Calibration problems contributed to the highest cell pressure. The average bulk density determined from the first two cell pressures was 1.9 tonnes/m³. This was confirmed from the results of routine compaction control testing carried out during construction of the embankment.

2.5.3 Pore water pressures

The excess pore water pressures measured by the pneumatic piezometers installed below the natural ground water table were less than 10 kPa. The piezometers placed at higher elevations to intercept perched water sources measured excess pore water pressures less than 5 kPa.

The excess pore water pressure dissipation response is more characteristic of cohesionless than cohesive soils.

3.0 BACK ANALYSIS OF EMBANKMENT SETTLEMENTS

The effective drained Modulus of Elasticity of the soils supporting the embankment was back-calculated from the measured settlements using the form of the relationship first given by Schleicher (1926). The shape factors used were those modified by Egorov (1958) and cited by Harr (1966).

The analysis yielded an effective drained Modulus of Elasticity for the entire weathered profile of 55 MPa for the average embankment settlement of 66 mm. The back-calculated modulus compares with an average horizontal modulus of 70 MPa determined from the results of the four Camkometer tests, and with an average modulus of 35 MPa determined from the six oedometer tests.

The back-calculated modulus appears high by world standards. De Graft - Johnson and Bhatia (1969)

reported the compressibilities of lateritised granitic soils in other parts of the world. No details on sampling methods, soil classifications and test methods were reported. The elastic moduli, computed from their reported one dimensional coefficients of volume decrease, without allowance for foundation stresses, varied from 1 to 10 MPa, with an average of 6 MPa. From the information reported, it is not possible to state whether the relatively low average modulus reflects low natural strengths or the effects of disturbances to the soil structure produced by sampling and testing or a combination of both factors.

The back-calculated effective modulus is more consistent with moduli reported for medium dense to dense sands. Bowles (1977) has quoted a moduli range of 50 to 100 MPa for the latter and 7 to 10 MPa for clays and silts, presumably not lateritised.

The vertical strain produced by the embankment loading was approximately 0.3%. This is on the average 25% of the strain range over which the stress vs strain relationships of these soils are linear (Smith, 1984).

The average measured settlement was approximately 32% greater than predicted from elasticity theory using Camkometer derived, elastic moduli and 60% less than the prediction based on the oedometer derived moduli.

4 SIGNIFICANCE TO FOUNDATION DESIGN

4.1 General

The rafts and conventional shallow foundations for the Refinery facilities were designed to satisfy allowable settlement criteria. The factor of safety against either localised or generalised shear failure was generally in excess of 3. Accordingly, designing economical foundation systems in these soil types depends upon accurately estimating the settlement profiles across the structures.

4.2 Settlement Theory

The results of the Camkometer testing showed that the stress vs strain relationships of the lateritised soils are essentially linear until the peak shear stresses are mobilised (Figure 3). These stresses are very high and indicate the beneficial influences of the cementitious sesquioxides. The average peak shear stress from the 14 Camkometer tests was 429 kPa (Smith, 1984). The foundation loadings for the Refinery facilities are such that the applied shear stresses will be less than 40% of the average peak shear strength as measured by the Camkometer. The maximum soil strains generated by the foundation loadings are estimated to be less than 0.5%.

Linear elastic theory, combined with layer modelling (as appropriate) with each soil layer of sufficient thickness to satisfy the assumptions of isotropy and homogeneity, provides a reasonable estimate of settlements.

Analytical and/or numerical methods are available for the determination of settlements in multi-layered elastic soils. Computer programmes such as Focals and Cranlay (Harrison et al, 1972) and solutions to boundary value problems of stresses and displacements in earth masses and layered systems are readily available (Hampton et al, 1969).

4.3 Measurement of Compressibility Parameters

In sensitive and brittle soil types, the techniques used to measure the compressibility parameters will have a very significant influence on the accuracy of settlement predictions. The use of laboratory determined compressibilities, without recourse to the results of field testing, may result in settlement overestimates by a factor up to 3. The cumulative disturbance effects of sampling, extrusion and pretest preparations could generate strains of at least 1% (i.e. 0.2 mm vertical compression over the thickness of the standard oedometer sample). This would mean that the ensuing consolidation test would be carried out on a sample of remoulded soil, that is physically very different to the soil in its undisturbed state. The parameters measured in the laboratory would approximate the soil's residual properties. The Camkometer tests indicated that the average residual strength of these soils is approximately 50% of the average peak shear strength (Smith, 1984).

The most suitable tests to measure the compressibility parameters are large scale loading and Camkometer testing. Large scale loading of the form discussed above, tests the lateritised profile in its undisturbed state and thereby provides the most accurate estimate of the compressibility properties. The Camkometer tests the soil in a relatively undisturbed state but in a horizontal direction. In assessing the results, account should be taken of the soils' anisotropic properties (Smith, 1984).

Preloading reduces the compressibilities of the lateritised soils and can be beneficial in minimising foundation settlements. The ratio of the reloading to initial tangent moduli of Elasticity, determined from the Camkometer programme, varied between 1.2 and 4.1, with an average of 2.5 (Smith, 1984). The average result means that on reloading over the initial stress range, the resulting settlements will be approximately 40% of those produced by the initial loading. The author has provided experimental evidence to show that further load cycling will not significantly increase the elastic modulus (Smith, 1984). The stiffening on reloading behaviour can be taken into account in the design of foundations for settlement-sensitive structures.

The cost of large scale loading is the reason that this method of investigation is not often employed. In certain situations, the savings in foundation costs resulting from less conservative settlement predictions and the utilisation of the beneficial proof loading effects may far exceed embankment construction and monitoring costs. For some of the very heavy Refinery structures, the total and differential settlements estimated using laboratory derived compressibility parameters could not have been accommodated. The results of the field testing produced a more realistic settlement estimate that was significantly less than the prediction based solely on laboratory results. The practical significance of the less conservative estimate was that all of the Refinery facilities, with the exception of the Powerhouse Facility, were founded on shallow footings.

4.4 Compressibility vs Index Properties

De-facto relationships between the field and laboratory test results were used in settlement-based designs of the foundations for the majority

of the heavy Refinery facilities. This approach has been discussed by Smith (1984) in a separate paper, and the salient features are presented below.

Semi-empirical, linear relationships were determined between the recompression indices, void ratios, dry densities and moisture contents from the results of the settlement monitoring of the trial embankment. Two of the relationships are presented below:

$$C_r = - 0.04 \times \rho_d + 0.08 \quad (1)$$

$$e = - 1.27 \times \rho_d + 2.50 \quad (2)$$

where C_r = recompression index

ρ_d = dry density (tonnes. m^{-3})

e = void ratio.

Higher order relationships are not considered warranted having regard to their empirical nature. It has been previously shown that these relationships approximate the lower limits of the laboratory results (Smith, 1984).

This approach enabled the results of the field testing to be extrapolated to areas where similar geological conditions prevailed. The elastic moduli, estimated from the recompression indices and void ratios determined from equations 1 and 2 above, were used in conjunction with elasticity theory to estimate the settlement profiles for foundation design.

5 CONCLUSIONS

From the information presented in this paper, the following conclusions are reached.

1. The lateritised soils at the site of the Worsley Alumina Refinery have significantly different engineering properties than temperate-zone soils of a similar Unified Soil Classification. The soils are products of insitu weathering in a semi-tropical environment of a porphyritic granite, and to a lesser extent, intrusive doleritic dykes. The soils have an open voided, skeletal-type structure, that attests to the efficacy of the capillary and gravity leaching mechanisms. Their structure is underpinned by relatively high and variable concentrations of cementitious iron and aluminium oxides (sesquioxides).
2. The sesquioxides dominate the engineering properties of these soils, particularly their shear strengths and compressibility properties. Shear strengths up to 800 kPa and Moduli of Elasticity up to 80 MPa (initial tangent moduli) were measured by Camkometer testing at the Refinery site. The constitutive properties of the clayey and silty soils exhibit brittle behaviour with the peak shear stresses generated at strains as low as 1%.
3. The settlements produced by a 10m high earthen embankment, imposing a maximum bearing pressure of 200 kPa, were 60 and 72mm at the two monitoring locations. The settlement occurred concurrently with construction.
4. Allowable settlement criteria govern foundation design in these soil types as the factor of safety against shear failure is generally well in excess of 3. Linear elastic theory is appropriate for settlement analyses as the shear

stresses that would be generated by the foundation loadings are significantly less than the average peak shear strengths of the supporting soils.

5. The accuracy of the settlement predictions depends on the techniques used to measure compressibility parameters. Camkometer and large scale testing (embankment loading) provide the most reliable estimates of the compressibility parameters. Laboratory testing in these soils will underestimate the elastic moduli and thereby overestimate settlements by a factor up to 3. This is due to unavoidable disturbance during sampling and pre-test preparations.

6. Semi-empirical relationships between the compressibility parameters and index properties were back-calculated from the results of the embankment loading and laboratory testing. These relationships were used in the evaluation of settlements for the Refinery structures at sites where similar geological conditions prevailed.

6. ACKNOWLEDGEMENTS

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