Fifth Australia-New Zealand
Conference on Geomechanics
Sydney, 22-23 August 1988

Embankment Settlements in Very Soft Alluvium Western Australia - Two Case Histories

D.M.A. SMITH
Director, Soil and Rock Engineering Pty Ltd

SYNOPSIS

Sand embankments are constructed over very soft alluvial deposits in Western Australia to consolidate the alluvium to enable civil works to be carried out. The settlement of an 8m high sand embankment, constructed in two stages over alluvium having shear strengths between 10 and 20 kPa and moduli of Elasticity less than 1.5 MPa, is presented as the first case history. The settlement of a 2m sand embankment over alluvial terraces of the Murray River is presented as the second case history. The screw plate test is used to provide load vs settlement vs time characteristics. Settlements predicted using screw-plate derived load vs deformation relationships are in good agreement with measured settlements. The rate of settlement, predicted from coefficients of consolidation derived from screw plate tests, over estimates measured rates.

1 INTRODUCTION

A rise in the sea level over the last 10,000 years has slowed river flows resulting in significant estuarine depositions of very soft, silty and clayey sediments. Churchill (1959). Sand surcharging is a common method of improving the strength of very soft alluvium to construct civil works.

The paper presents two case histories in Western Australia of settlements generated by sand embankment loading of very soft alluvial sediments. Screw plate testing was carried out to determine their strength and compressibility properties. The predicted settlement is compared with the settlement history measured at each site.

2 NICHOLSON ROAD BRIDGE - CASE HISTORY #1

2.1 Project Description

A 2 lane and 3 span, prestressed concrete bridge was built over the Canning River.

Sand embankments were constructed at the eastern and western approaches to the bridge. The eastern embankment was constructed over highly compressible alluvium varying in thickness to 15m and overlain by less compressible deposits to depths in excess of 30m. The western embankment was constructed over hard clays and medium dense to dense clayey sands. (Figure 2). The settlement of the alluvials supporting the eastern embankment is the subject of this case history.

The construction programme allowed 6 month preloading of the site for the eastern embankment prior to bridge construction. To assess the suitability of this preloading period, the following geotechnical information was determined:

(i) strength and compressibility properties of alluvial sediments; and

(ii) estimates of the magnitude and rate of embankment settlement.

2.2 Geology

The bridge spans variable ground conditions. The idealised stratigraphy, determined from the investigation boreholes and inferred from the results of electric friction-cone penetrometer probes (Figure 1), is shown in Figure 2.

Over the last 10,000 years (Quaternary period), the river cut a deep channel (>30m) through the strong cohesive materials (Guildford Geological Formation) present to the west of the existing sand island. (Figure 1). The channel was filled with alluvium, varying from medium dense silty sands to very loose mixes of silts, sands and clays. The non-symmetrical "bowl" shape of the loose to very loose alluvium suggests that the eastern limit of the former river channel (not defined by the investigation) may not be as steeply sloping as the western limit. It would appear that sediments were deposited in an alluvial fan in areas of slack water.

2.3 Engineering Properties

The engineering properties of the alluvium encountered by BH 3A (Figure 1) are tabulated in Figure 3.

Five screw plate tests were carried out in the upper 10m to measure the in situ strength and compressibility properties. All tests were carried out using a 300mm diameter plate. The plate aspect ratios and testing procedures have been documented by the author. Smith (1987) (a).

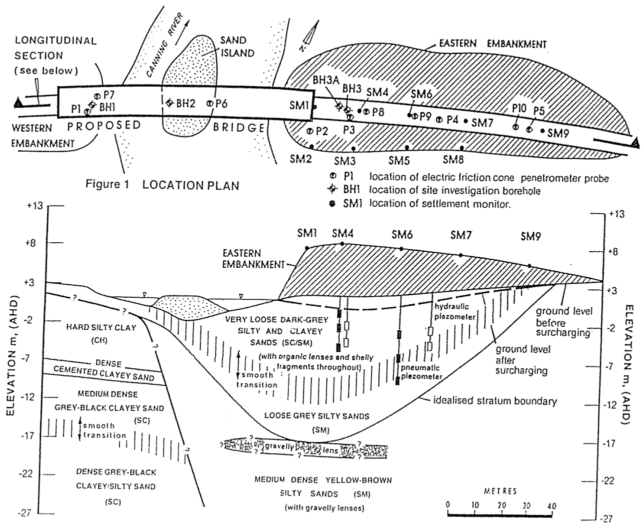


Figure 2. Idealised Ground Profile Along Proposed Bridge Centreline.

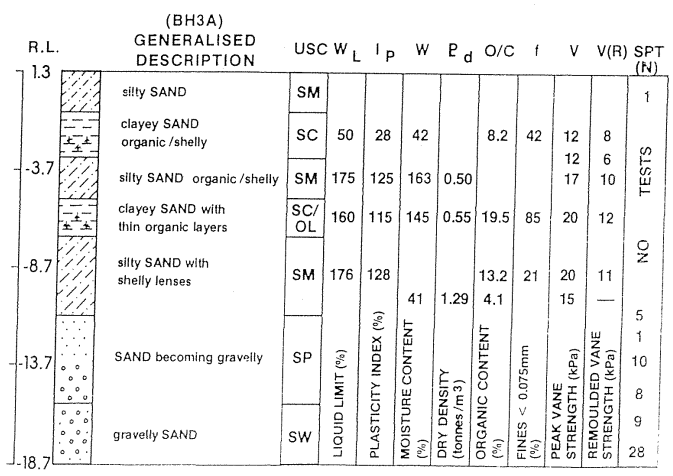


Figure 3. Engineering Properties of the Alluvium.

The results of two screw plate tests are shown in Figure 4(a) and (b).

A summary of screw plate test results is tabulated below.

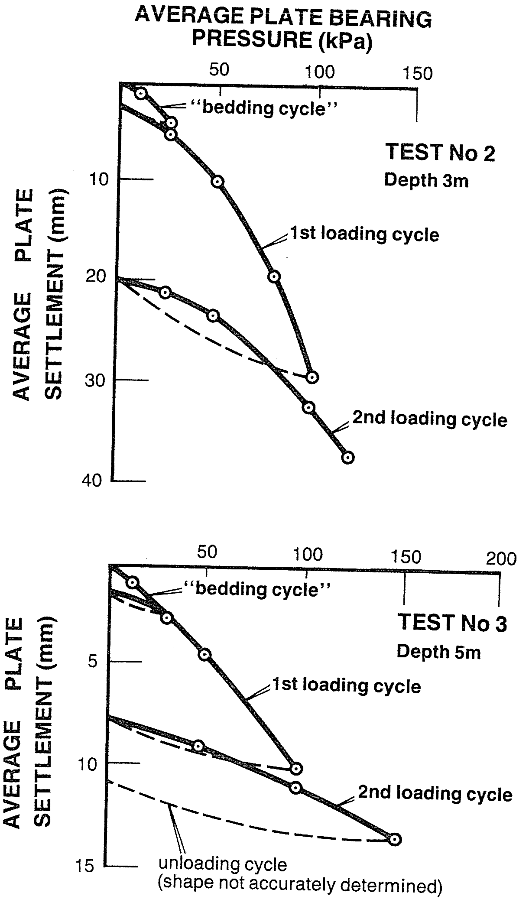


Figure 4 (a) (b). Loading and Unloading Relationships from Screw Plate Tests.

TABLE I SUMMARY OF SCREW PLATE TEST RESULTS

TEST NO.	DEPTH (m)	REDUCED LEVEL (m, AHD)	q_u (kPa)	τ_u (kPa)	$E_{1(100)}$ (MPa)	$E_{2(100)}$ (MPa)	$C_{v(100)}$ (m^2/yr)	$C_{r(100)}$ (m^2/yr)
			STRENGTH AND COMPRESSIBILITY PROPERTIES					
1	1.5	+0.2	100	9	0.6	0.4	900	200
2	3.0	-1.7	120	10	0.5	1.0	500	130
3	5.0	-3.7	150	12	1.3	3.2	500	80
4	8.0	-6.7	200	15	4.7	2.0	450	90
5	10.0	-8.7	230	17	1.7	1.3	600	100

Key: AHD denotes: Australia Height Datum;
q_u denotes: ultimate plate pressure estimated from hyperbolic model, (Kondner (1963));
τ_u denotes: undrained shear strength, (Smith 1987 (a));
$E_{1(100)}$ denotes: drained modulus of Elasticity (initial cycle after "bedding" cycle) for applied pressure increment of 100 kPa;
$E_{2(100)}$ denotes: drained modulus of Elasticity (2nd loading cycle) for applied pressure increment of 100 kPa;
$C_{v(100)}$ denotes: coefficient of vertical consolidation, (Kay and Avalle (1982)), for applied pressure increment of 100 kPa; and
$C_{r(100)}$ denotes: coefficient of radial consolidation, (Janbu and Senneset (1973)), for applied pressure increment of 100 kPa.

Two oedometer tests were carried out on samples taken from 5.9 and 8.3m. Void ratio vs effective pressure relationships are presented in Figure 5. Rate of consolidation vs effective pressure relationships are presented in Figure 6.

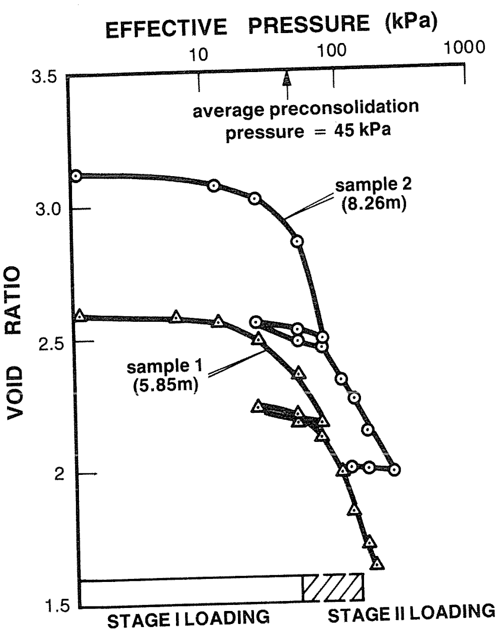


Figure 5. Void Ratio vs Effective Pressure.

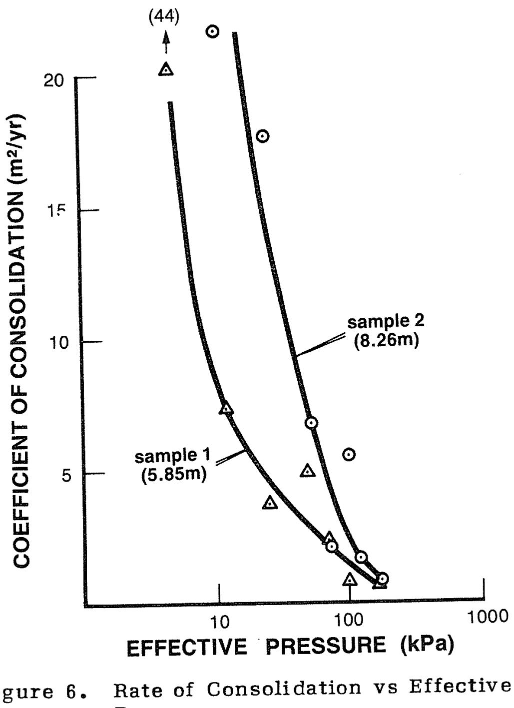


Figure 6. Rate of Consolidation vs Effective Pressure.

The constrained moduli of Elasticity, computed from the oedometer tests, are 1.9 MPa for sample 1 and 1.7 MPa for sample 2, for an effective pressure increment of 100 kPa. The coefficient of consolidation for this pressure increment is less than $2m^2/yr$.

From the results of field and laboratory testing, the salient engineering properties are summarised below:

(i) the high natural moisture contents and their proximity to liquid limits indicate normal consolidation behaviour;

(ii) the low dry densities indicate a highly stressed and open voided soil structure, that would be susceptible to disturbance produced by conventional sampling procedures and preparation for laboratory testing;

(iii) shear vane strengths (peak and remoulded) are low and generally increase with depth. The ratio of peak to remoulded strengths is typically 1.5 to 2, indicating soils of low sensitivity, (Smith (1974));

(iv) the moduli of Elasticity are low and consistent with the range reported by Bowles (1977) for silty soils. On reloading, the moduli are lower (with the exception of tests 2 and 3, where the soils are sandy), indicating that the clayey alluvials undergo strain softening; and

(v) the coefficients of consolidation determined from screw plate testing are significantly higher than the coefficients determined from oedometer testing. The screw plate derived coefficients are more consistent with expectation, given the very low dry densities and high natural moisture contents. (Figure 3). These properties indicate that the soil structure is open voided. It is possible that the voids are connected. Rates of consolidation are expected to be relatively fast in open voided soils.

2.4 Embankment Construction and Monitoring

An 8m high sand embankment was constructed in 2 stages over the site for the eastern embankment. Stability berms were incorporated into the stage 1 construction. These berms confine the alluvium at the toe of the stage 2 embankment and increase stability.

The stability of the embankment during stage 2 construction was evaluated daily by monitoring pore water pressures and settlements. The measured water pressures were put into a slope stability computer programme (STABL), and relationships between theoretical factor of safety, height of embankment and pore water pressures were determined. A theoretical factor of safety of 1.2 was adopted as an acceptable construction minimum. When the embankment had reached a height of 6.5m, construction was stopped for 7 days to allow time for some pore water pressure to dissipate.

Settlement monitors and piezometers were installed in the alluvium at the approximate locations shown in Figures 1 and 2.

Six remote reading pneumatic piezometers (low air entry) were installed throughout the highly compressible alluvium in the upper 10m. Prior to stage 2 construction, four hydraulic piezometers were installed as a check on the pneumatic piezometers.

Nine settlement monitors were installed in 0.5m deep trenches excavated in the natural ground and backfilled with sand. Each monitor comprised a steel standpipe, graduated in 0.3m intervals, and fixed into a circular concrete base.

Twenty survey pegs were placed around the embankment to monitor toe movements.

2.5 Results of Monitoring

The settlements recorded at SM4 (refer Figure 1 for location) are plotted against time and staged embankment construction, (Figure 7(a) and (b)). Predicted settlements at the end of stages 1 and 2 are shown in Figure 7(b). Maximum settlements measured at all monitoring stations are presented in Table II.

The excess pore water pressures determined from the pneumatic and hydraulic piezometers at location SM4 are shown in Figure 7(c).

The excess pore water pressures determined from the pneumatic piezometer measurements, were significantly lower than those determined from the hydraulic piezometers. Reliance on the pneumatic piezometers readings would not have led to a curtailment in the rate of stage 2 embankment construction and could have resulted in an embankment failure.

2.6 Predicted and Measured Settlements

Figure 7(b) shows reasonable agreement between the magnitudes of the predicted and measured settlements.

A back analysis was carried out on the measured settlements at SM4. A one dimensional consolidation model, using average coefficients of vertical consolidation of 100 and $150m^2/yr$ and doubly drained boundary conditions, was used. The results are summarised in Figure 8.

Using this simplistic model, it can be seen that the average effective coefficient of vertical consolidation is, for practical purposes, between 100 and $150m^2/yr$. This compares favourably with the range of coefficients derived from the screw plate test results using the Janbu and Senneset model (1973), (Table I).

The "knee" in the settlements measured from stage I construction represents a preconsolidation pressure of 35 kPa (approximately). (Figure 7(b)). An average preconsolidation pressure of 45 kPa was calculated from the two oedometer tests, (Figure 5).

It is possible that the preconsolidation pressure has been the result of "aging" of the deposits. Bjerrum (1972) pointed out that secondary compression or delayed consolidation under constant effective stress over a long period of time, can result in a more stable structural configuration resulting in greater strength and reduced compressibility. Another contributory mechanism may have been the effects of water table fluctuation and strength gain due to consolidation from capillary-induced stresses.

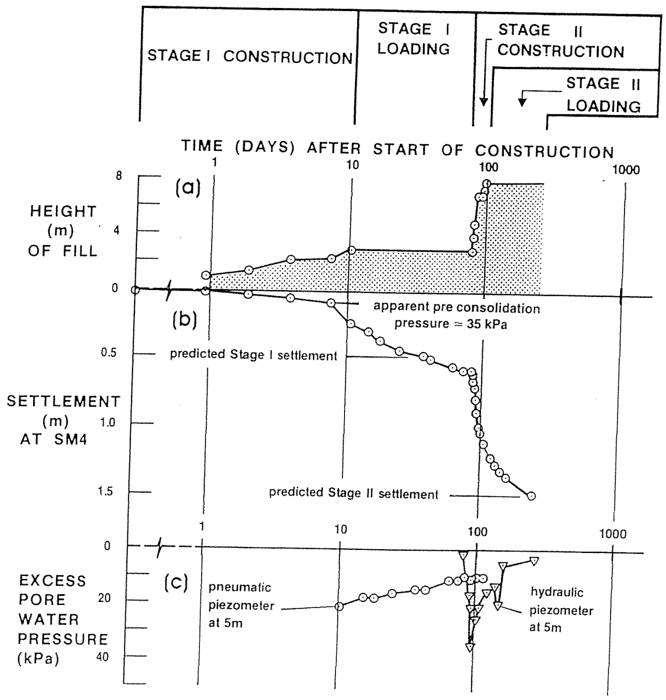


Figure 7 (a)(b)(c) Settlement and Pore Water Pressure Monitoring at SM4.

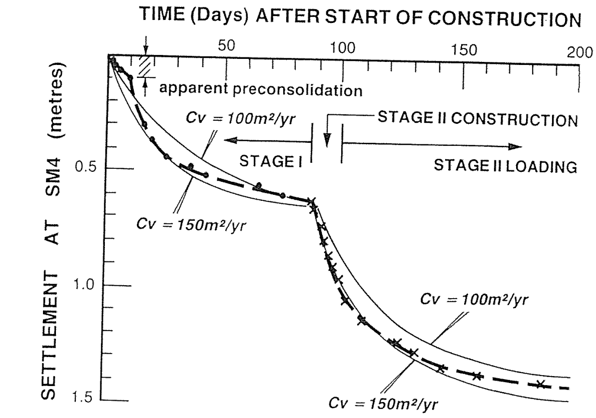


Figure 8. Back Analysis of Rate of Settlement at SM4.

TABLE II MAXIMUM SETTLEMENTS, EASTERN EMBANKMENT

Monitoring Location (SM)	Embankment Height (m)	Thickness of Highly Compressible Alluvium (m)	Maximum Settlement (m)
1	7.2	15	0.737
2	3.5	15	0.424
3	1.1	14	0.397
4*	8.3	14	1.472
5	2.5	12	0.370
6	7.6	12	1.329
7	5.1	8	0.835
8	3.2	8	0.209
9	2.7	0.3	0.150

Notes: * settlement vs time embankment loading relationships are shown in Figure 7 (a), (b) and (c).

** from settlement monitors buried 0.5m below natural ground level.

3 MURRAY RIVER RESORT DEVELOPMENT CASE HISTORY #2

3.1 Project Description

Canal developments allowing home owners access to their own water frontage are popular in Western Australia. The ground conditions at the canal sites are invariably highly compressible, alluvial sediments.

The Murray River Resort Development in South Yunderup, located 110 kilometres south-east of Perth, is located over an alluvial fan of the Murray River. (Figure 9). The development of this site required the placement of 2m of imported clean sand filling. Key geotechnical considerations in the planning and development included inter alia, the strength and rate of consolidation of the very soft alluvial deposits to determine the most appropriate time to commence construction.

3.2 Ground Conditions

Two boreholes and 25 electric friction-cone penetrometer probes were carried out to define the ground conditions across the site. (Figure 10)

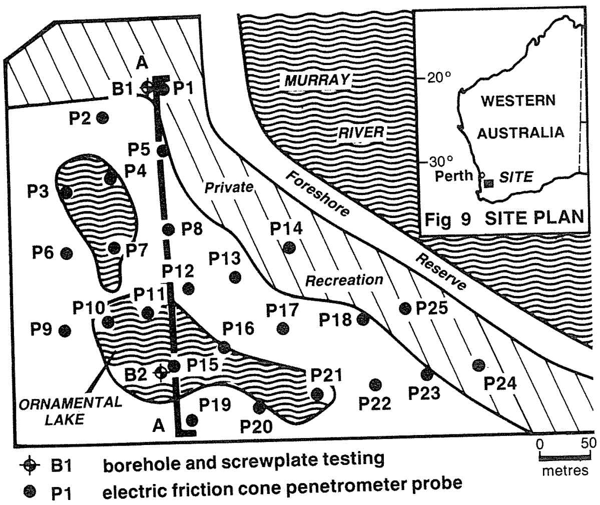


Figure 10. Borehole and Penetrometer Locations.

The ground conditions investigated were very loose to loose, silty and clayey, medium to fine grained sands approximately 2m thick, overlying very soft, clayey silts. The thickness of the clayey silts varied from 0.5m to 4.3m. They are underlain by loose sands with thin clayey and silty lenses to 15m. Groundwater was encountered between 0.6m and 0.8m below ground level.

Subsurface conditions are shown in idealised form in Figure 11.

3.3 Engineering Properties

The results of particle size distribution and Atterberg limit tests carried out on the clayey silts are presented in Figure 12.

Five screw plate tests were carried out in the highly compressible deposits within the upper 4m. A 125mm diameter plate was used for each test. Plate aspect ratios and testing procedures have been documented by the author. Smith (1987) (b).

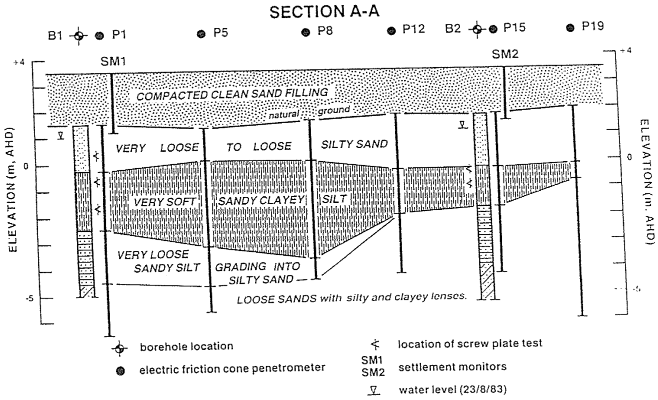
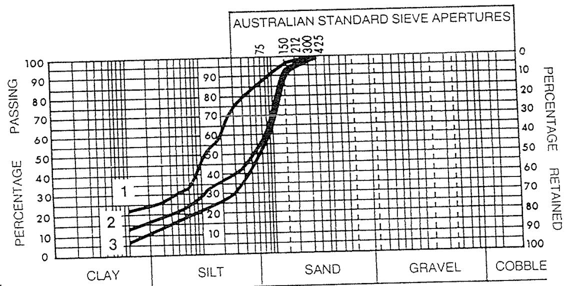


Figure 11. Idealised Subsurface Profile at Murray River Resort.



SAMPLE	BOREHOLE No	DEPTH	LL(%)	PI (%)
1	2	3.0	74	49
2	1	2.1	47	18
3	1	2.7	37	15

Figure 12. Engineering Properties of Clayey Silts.

The relationship between average plate bearing pressure and average settlement for one test is shown in Figure 13. Plate settlement due to self weight of the testing apparatus is included.

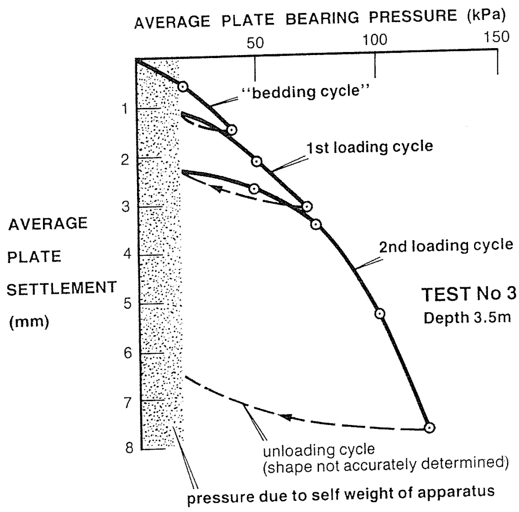
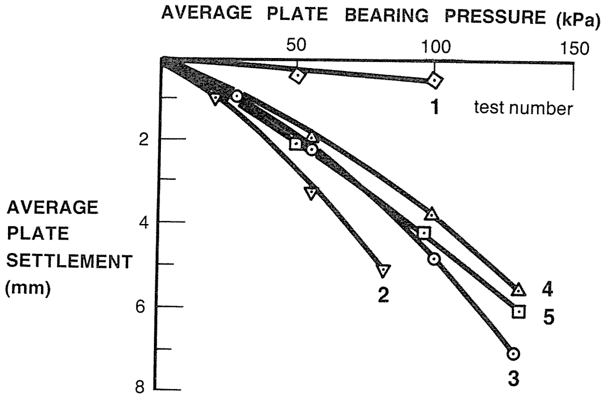


Figure 13. Results of a Cycled Screw Plate Test.

Figure 14 summarises the results of all tests. Unload and reload loops are omitted for clarity. The significantly lower strengths of the clayey silts are evident by comparing test number 1 (carried out in surficial sand) with the other results. The settlement at the end of primary compression corresponding to each pressure increment was estimated using the graphical construction, proposed by Kay and Avalle (1982).



TEST	GROUND CONDITIONS
1	*loose medium to fine grained SAND*
2 to 5	*soft clayey SILT*

Figure 14. Summary of Screw Plate Test Results.

Estimates of shear strengths and moduli of Elasticity, determined from the results of the screw plate tests, are tabulated below.

TABLE III STRENGTH AND DEFORMATION PROPERTIES

Test No. /Depth (m)	Estimated Shear Strength (kPa)	Static Drained Modulus of Elasticity (MPa)	Coefficient of Consolidation (m^2/yr)	
			C_r	C_v
1/1.5	N/A*	18	500	500
1/2.5	12	0.8	50	145
3/3.5	16	1.3	38	88
4/3.0	18	1.4	53	167
5/2.5	20	1.3	36	77
6/2.8	not determined	1.3 (64 kPa)	4	14
7/3.2	not determined	1.5 (34 kPa)	16	60

Notes

(1) Tests 1 to 5 are standard tests. The drained moduli of Elasticity and coefficients of consolidation are appropriate to an effective stress level of 60 kPa.

(2) Tests 6 and 7 were carried out over 6 days. Stress levels, at which the moduli and consolidation coefficients have been determined, are bracketted.

(3) C_r denotes the coefficient of radial consolidation, determined from Janbu and Senneset (1973) model.

(4) C_v denotes the coefficient of vertical consolidation, determined from Kay and Avalle (1982) model.

(5) Shear strengths are determined from the ultimate plate pressure minus effective overburden pressure, divided by 10. Smith (1987) (b).

(6) *N/A denotes not applicable.

Two sustained load tests were carried out over 6 days to evaluate long term deformation characteristics and the appropriateness of the graphical procedure, recommended by Kay and Avalle (1982), to estimate the end of the primary consolidation phase. A simple load frame was set up at each test location and screw plates of 125mm and 300mm diameter were subjected to constant loads over the test.

The result of one test is presented in Figure 15. Superimposed is the graphical construction proposed by Kay and Avalle (1982). The result of the other test is very similar.

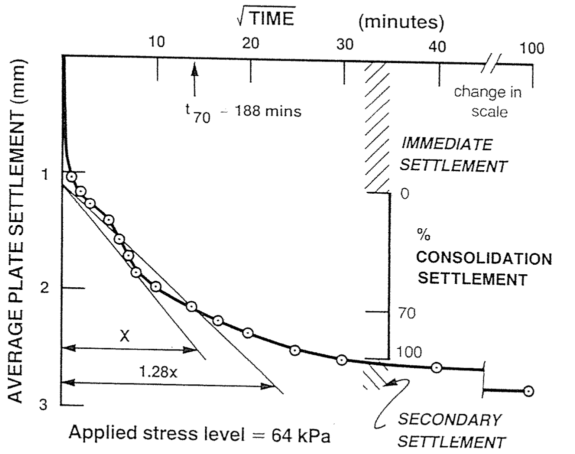


Figure 15. Sustained Load Test Result with Kay & Avalle (1982) Construction Superimposed.

The salient results are:

(i) the average plate settlement vs time relationship exhibits three distinct and recognisable phases, namely immediate compression, primary and secondary compression;

(ii) Kay and Avalle (1982) graphical construction provides a reasonably accurate estimate of the end of primary consolidation; and

(iii) secondary compression is not very significant at the applied stress level.

3.4 Site Filling and Settlement Monitoring

As part of site development, all root matter within the upper 300mm was removed and the site was filled with approximately 2m of compacted clean sand.

Two settlement monitors were installed prior to placement and compaction of the sand fill. (Figure 11). Each monitor comprised a square steel plate (0.6m x 0.6m) to which was attached a graduated standpipe. The monitors were buried in 0.3m deep trenches excavated into the natural subgrade, that had been stripped of surficial root matter. The settlement of steel plates was determined by taking levels of the graduated standpipes at regular time intervals.

Settlements are plotted against time in Figure 16. Monitoring was discontinued after 8 months.

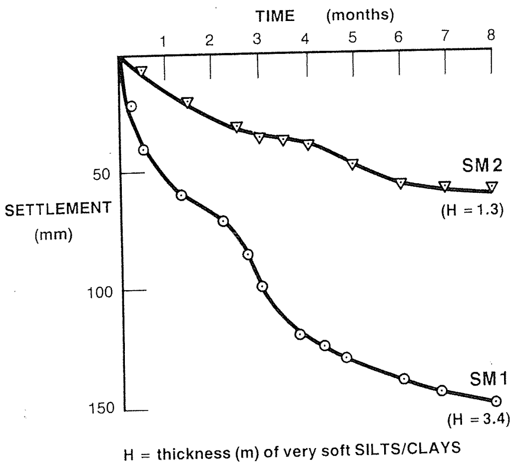


Figure 16. Measured Settlements of Monitors SM1 and SM2.

3.5 Discussion

A back analysis was carried out on the settlements of Monitor No. 1. The results are summarised in Figure 17(a). The idealised ground model used in the back analysis is presented in Figure 17(b). The objective is to match theoretical with observed settlement behaviour to compare the soil parameters used in the back analysis with those derived from screw plate testing.

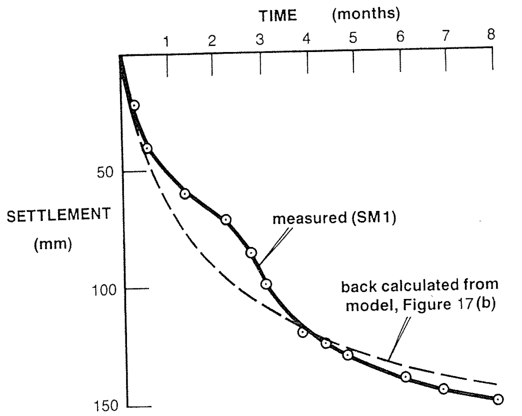


Figure 17 (a). Back Analysis of SM1 Settlements.

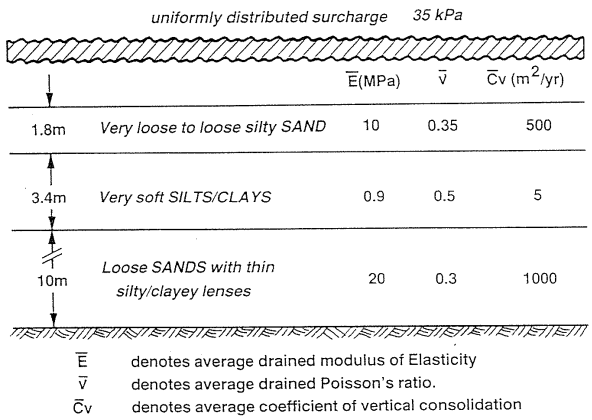


Figure 17 (b). Idealised Ground Model for SM1.

The salient results of the analysis are:

(i) the settlement within the clayey silt layer is approximately 120mm and represents 80% of the total settlement generated by the 2m sand surcharge;

(ii) the rate of settlement in the clayey silt is two orders of magnitude slower than the settlement in the near surface, silty sands. One dimensional coefficients of consolidation of $5m^2$/yr (silt) and $500m^2$/yr (silty sand) were assigned to the idealised ground model, (Figure 17 (b)); and

(iii) the match between the back calculated relationship (shown as the broken curve in Figure 17 (a)) and the measured settlement is reasonable over both the initial part and latter half of the monitoring period. Between 0.5 and 3.5 months, the curve joining the measured points deviates from the theoretical curve and exhibits a reverse S shaped curvature, similar to that exhibited by screw plate test results. (Figure 15). This settlement characteristic may be due to a change in soil structure. The rate of settlement increased appreciably after vertical strains of 2% had been reached, indicating the possibility of a strain softening mechanism.

A comparison of the soil parameters used in the idealised ground model with those derived from the results of screw plate testing (Table III) indicates the following:

(i) the assigned average modulus of Elasticity of 0.9 MPa for the clayey silt is towards the lower end of the range determined from screw plate tests (i.e. 0.8 to 1.5 MPa); and similarly

(ii) the assigned average coefficient of consolidation of $5m^2$/yr is towards the lower end of the range of coefficients derived from the screw plate tests (i.e. 4 to $53m^2$/yr) using the Janbu and Senneset (1973) radial consolidation model. The Kay and Avalle (1982) model gives coefficients up to 3 times greater than those derived from the Janbu and Senneset model.

The coefficients of consolidation, derived from screw plate testing, are generally greater than those back-calculated from measured settlement vs time relationships. Reasons for this difference are considered to be a combination of:

(i) anisotropy of flood plain deposits, that tend to be more permeable in a horizontal than in a vertical direction. Accordingly, the coefficients of consolidation determined from screw plate testing (a three dimensional test) will be greater than the coefficients determined from large scale field loading, that is essentially one dimensional;

(ii) method of installation of the screw plate. The radial slot produced by the cutting edge of the screw plate together with disturbed soils above the plate can provide additional drainage paths to accelerate the dissipation of excess pore water pressures; and

(iii) the natural variability of the flood plain alluvials, and in particular, the presence of thin and discontinuous permeable sandy lenses present within the ground stressed by the screw plate.

4 MAJOR FINDINGS

The major findings are:

1. The alluvial sediments, deposited by the Canning and Murray Rivers in Western Australia, are predominantly mixes of very soft clays and silts. Their undrained strengths are typically 10 to 20 kPa. Their undrained moduli of Elasticity are generally less than 1.5 MPa.

2. Sand embankments constructed over these alluvials will undergo large settlements. An 8m high embankment constructed in 2 stages settled 1.5m. Settlements within each metre of the alluvials for each metre of embankment loading varied from 15 to 20mm for the two case histories presented.

3. Embankment construction may have to be carried out in stages to allow sufficient time for the alluvials to consolidate to prevent a slip failure.

4. Coefficients of consolidation calculated from oedometer testing are significantly lower than those back calculated from field measurements and determined from screw plate testing. Reliance on laboratory derived parameters will result in significant under-estimates of the rate of consolidation and could result in the installation of unnecessary (and expensive) drains to accelerate settlement.

5. Progressive monitoring of settlements and pore water pressures are recommended to determine the most appropriate rate of embankment construction.

6. The screw plate test can provide reasonably accurate in situ determinations of the load vs deformation vs time properties of very soft alluvials.

7. The radial consolidation model proposed by Janbu and Senneset (1973) was found to be more appropriate for estimating rates of settlement than the vertical consolidation model proposed by Kay and Avalle (1982).

8. The screw plate test is simple, representative of normal loading conditions and utilises robust equipment. The test can be carried out for approximately the same cost as the oedometer test. The soil tested by the screw plate is significantly greater in volume and less disturbed than the soil tested in the oedometer.

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