INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

The paper was published in the proceedings of the 8th Australia New Zealand Conference on Geomechanics and was edited by Nihal Vitharana and Randal Colman. The conference was held in Hobart, Tasmania, Australia, 15 – 17 February 1999.

Comparative Evaluation of Compression Characteristics of Loose Sediments

T. Ford
B.E. (Civil)
Senior Geotechnical Engineer, Gutteridge Haskins & Davey Pty Ltd

Summary This paper describes the methods that were used to evaluate compressibility characteristics when oedometer testing yielded anomalous results. A residential development was constructed on quaternary alluvium which contains a 2.0m thick layer of very loose sediment. Surcharge monitoring showed higher than expected initial compression, and subsequent oedometer tests showed creep dominated behaviour, high rebound stiffness and apparent C_α/C_c ratios in the range 0.046 - 0.28. Three alternative methods of estimating the primary consolidation parameters C_c and M were used; by published correlations with index properties, by strength modeling using EFCPT data and by strain modeling using records of 15 settlement monitoring plates. Estimates of C_α were then made by adopting published correlations of C_α/C_c. It is concluded that for certain highly compressible inorganic sediments, design and interpretation of oedometer testing should be undertaken with great care, and a combination of field, laboratory analysis, and empirical correlations should be used to arrive at appropriate stiffness parameters.

1. NOTATION

The following symbols are used in this paper.

C_c = compression index = $\Delta e / \Delta \log \sigma_v'$
C_c = swelling index = $\Delta e / \Delta \log \sigma_v'$
C_α = secondary compression index = $\Delta e / \Delta \log t$
CR = compression ration = $C_c/(1+e_0)$
C_r = re-compression index = $\Delta e / \Delta \log \sigma_v'$
C_v = coefficient of consolidation
DD = dry density of soil
e = void ratio
e_0 = in-situ void ratio
f_s = sleeve friction of electric friction cone
FR = friction ratio of electric friction cone = f_s/q_c
Δh = settlement of compressible layer
k_v = coefficient of permeability in vertical direction
H_o = preconstruction thickness of a compressible layer with void ratio e_0
m_v = modulus of volume change
M = constrained modulus = $1/m_v$
M_0 = initial constrained modulus
PI = plasticity index (%)
q_c = electric friction cone penetration resistance
R_s = surcharge ratio = $1-\sigma_{vf}'/\sigma_{vs}'$
S_r = degree of saturation of soil
t = elapsed time from application or removal of vertical stress
t_{eop} = time for end of primary consolidation
w_L = liquid limit
w_0 = natural water content in percent of dry weight
σ_p' = apparent preconsolidation pressure
σ_v' = vertical effective stress
σ_{vf}' = final effective vertical stress
σ_{vs}' = maximum effective vertical stress due to surcharge
σ_{v0}' = in-situ or initial effective vertical stress
ε_a = vertical strain due to secondary compression

2. BACKGROUND

A 24 Ha residential "canals" development on the south coast of WA was constructed by filling low lying ground that was underlain at shallow depth by a 2.0m layer of very loose, saturated deltaic sediment, referred to in this paper as the "soft layer". It was desired to minimise future foundation settlements under structure loading, and to predict these settlements accurately enough to provide a site classification in accordance with AS2870.

The ground was improved by a 1.5m preload left in place for one month. Because of time constraints, construction of deep sewer lines and canals took place concurrently with preload placement and settlement monitoring. Thus while the interpretation of monitoring plate data was complicated by additional settlements caused by intermittent dewatering, the effective preload was increased by the dewatering operations.

3. SITE GEOLOGY

Pelican Point is a headland lying between the Collie River and Vittoria Bay on the south coast of WA. Soils are low lying river terraces overlying quaternary alluvium, to depths in excess of 12m. Groundwater reflects tide levels, and thus is generally at RL 0.0 (AHD) which is about 1m below the natural ground surface.

The top of the soft layer is 0.5m-1.0m below the water table. It is grey to black, and in common with many deltaic deposits, it exhibits marked stratification with more clayey lenses interspersed with silty sand lenses. The generalised subsurface profile is shown in Table 1 and a typical Electric

Friction Cone Penetration Test (EFCPT) plot is shown in Figure 1.

Table 1. Typical Profile

Depth (m)	Description	q_c (MPa)
0 - 2.0	SAND, loose to medium	2 - 5
2.0 - 4.0	CLAYEY SILTY SAND with shell fragments (Soft Layer)	0.20
4.0 - 7.0	SILTY SAND, medium dense with sandy clay lenses	6 - 20
>7.0	SAND/SANDY CLAY, dense/very stiff	>20

During initial drilling, the SPT head fell through the soft material under it's own weight, ie N = 0, and occasionally, N = 1.

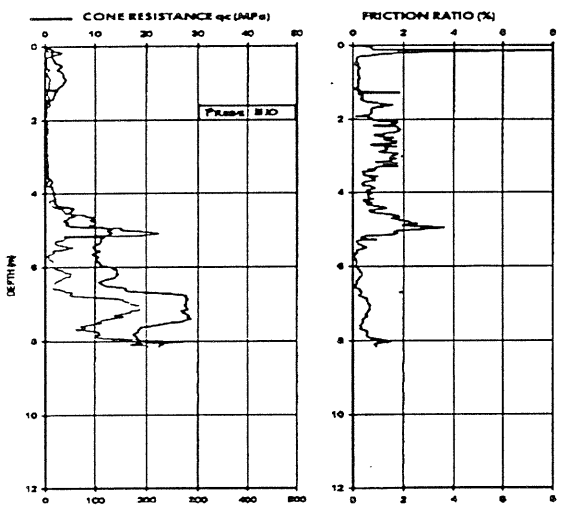

Figure 1. EFCPT Plot for Probe E10

4. GEOTECHNICAL PROPERTIES FROM LABORATORY TESTING

4.1 Composition and Index Properties

Atterberg limits on three samples and Particle Size Distribution tests on 12 samples, summarised in Table 2, classify the material as SC to CL, a low plasticity inorganic material that varies from fine grained sand with silt and clay, to sandy silty clay.

The relatively high sand content puts the soil into the "intermediate" category as defined by Nakase (1991), i.e. it's behaviour is partially cohesive and partially cohesionless.

Table 2. Index Properties & Grading

Parameter	Range of Values
w_L (%)	NP - 47
PI (%)	NP - 25
% passing 2.36 mm	99 - 100
% passing 0.075 mm	15 - 52

Miscellaneous other tests results were:

- Hydrometer analysis on the most plastic sample taken, sample SA2, gave 10% passing 2 υm.
- Organic content loss on ignition tests on 3 samples gave between 1.4 % and 5.5% organics.
- Carbonate Content on two samples SA1 and SA2 gave 9% and 8% carbonate content respectively.
- Pilcon shear vane tests on SA1 gave a peak shear strength of 20 kPa and a residual strength of 7 kPa, indicating a sensitivity of about 3.

4.2 Compressibility

A great difficulty was experienced in obtaining undisturbed samples during drilling, as the material simply slipped out of the tube.

Oedometer tests were carried out on four undisturbed samples, and Table 3 shows details of their initial condition.

Table 3 Sample Details

Sample	w_o (%)	S_r (%)	DD (t/m^3)	e_o
A7	27.9	116.4	1.60	0.62
SA1	34.7	100.1	1.38	0.92
SA2	59.6	98.0	1.01	1.61
SA3	51.2	97.4	1.11	1.40

Twenty four hour or less stage loading method was conducted on Samples A7. SA3 was subjected to sustained loading to 10,000 minutes or 5 log cycles of time for three stages . Sample SA1 and SA2 were subject to a combination of sustained loading, unloading, and recompression at various loads. The stress range which was of interest in this project was 40 kPa to 120 kPa, and unloading, re-compression, and creep testing was concentrated within this range.

Oedometer tests were interpreted by the standard graphical construction methods; on the e-logP curve for evaluation of σ_p', and Taylor's curve fitting method for t_{eop} . C_c and C_α were calculated from t_{eop} .

Figure 2 below shows e-log P curves for Samples SA1 and SA2 which represent the most sandy and most clayey materials sampled. It is evident from Figure 2 that both samples show a very stiff deformation response in recompression.

Figure 3 shows the settlement-time plots for SA1 in virgin compression over the stress range 40-80 kPa, and re-compression over the stress range 40-60 kPa.

Figure 4 shows the settlement-time plots for SA2 in virgin compression over the stress range 53-100 kPa, and re-compression over the stress range 53-75 kPa.

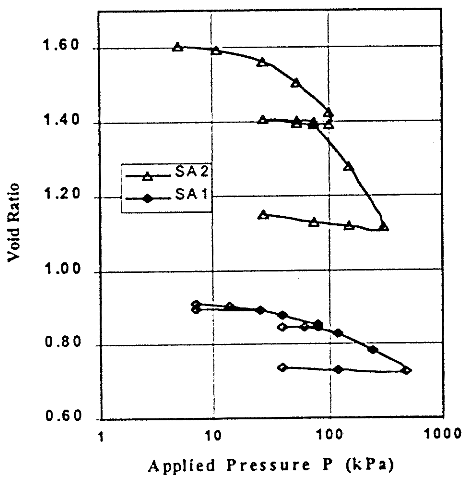

Figure 2. e-log P curves

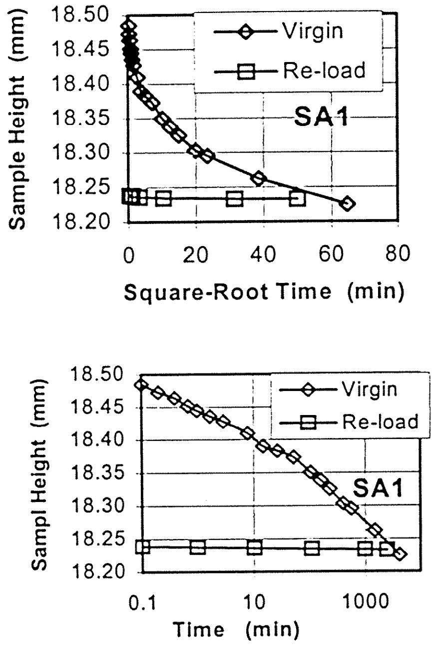

Figure 3. Settlement-Time Curves for SA1

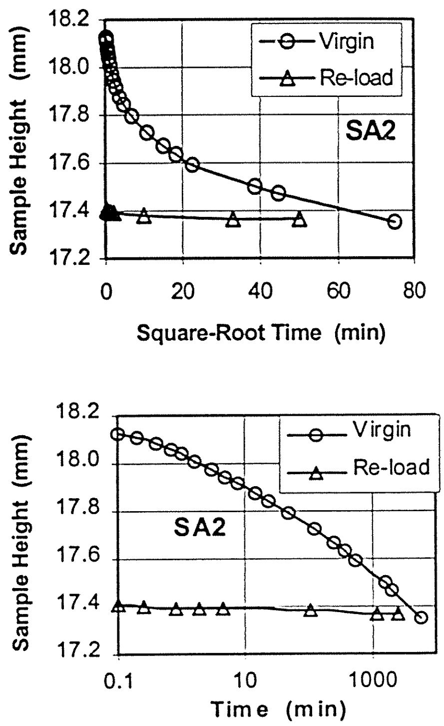

Figure 4. Settlement-Time Curves for SA2

The main characteristic of the virgin curves from the load stages 20-120 kPa was an apparent transition from the primary to the secondary curve without evident primary levelling off, and a gradual increase in slope on the settlement-log time plot.

C_α values in all cases were calculated over the time from end of primary consolidation (EOP), to the end of the stage, and are thus average values. As the C_α values were found to increase with sustained loading, it is expected that the average C_α will be somewhat higher than a C_α measured in the first post primary log cycle of time.

Tables 4 give selected parameters calculated from parameters from oedometer test results.

The most striking aspect of the values shown in Table 4 is that in only one case was the C_α/C_c ratio within the range of 0.03-0.05 expected for inorganic clayey material from Mesri & Feng's (1991) experiments. In 6 out of 9 cases the C_α/C_c ratio was well outside the range (02-0.10) reported by the authors for all geotechnical materials.

Other features of Table 4 are:

- values for k_v are far smaller than would be expected for stratified material with generally high sand content.
- Recompression stiffness is around 20 for SA1, the more sandy sample, and around 4 for SA2.

Analysis of these results in the light of Mesri's work on secondary compression (1977, 1991, 1997) suggests that the true virgin stiffness response in some of the stage loading tests within the pressure range 40-120 kPa has been masked by soil ageing caused by sustained compression. The indicators that soil ageing has produced a partial recompression rather than a virgin response on next loading are;

- pressure increment ratios were not high enough to produce a defined inflection point in e-log t curves; and
- the increasing slope on the e-log t curves are typical of loads that are at or near preconsolidation pressure.

These features have been noted previously, and are evident in Figures 3 and 4.

Table 4. Selected Parameters from Oedometer Tests

Load Range kPa	C_v m^2/yr	k_v m/s	M (Mpa)	C_c	C_r	C_s	CR	C_α	C_α/C_c	ε_α
Sample A7										
50-100	26.3		5.4	0.047			0.030			
200-400	15.8		8.8	0.116			0.076	0.0054	0.046	0.0035
Sample SA1										
40-80	54.7	$2.3e^{-10}$	6.7	0.037			0.019	0.0057	0.154	0.0031
80-40						0.0037				
40-60			127		0.0016		0.0009	0.0002	0.125	0.0001
240-480	140	$2.9\ e^{-9}$	13.2	0.108			0.060	0.0078	0.073	0.0045
Sample SA2										
50-100	24.0	$1.1e^{-9}$	4.2	0.103			0.041	0.025	0.244	0.011
100-50						0.0156				
50-75			14.1		0.0245			0.0017	0.069	0.0007
150-300	12.3	$9.0e^{-10}$	4.26	0.269			0.118	0.0032	0.125	0.0151
Sample SA3										
60-120	18.0	$1.6e^{-9}$	3.4	0.13			0.059	0.030	0.229	0.0138
120-240	13.0	$7.9e^{-10}$	5.3	0.163			0.078	0.029	0.179	0.0144

The premise that soil ageing made C_c difficult to define, is supported by the fact that, of all the stage loading tests carried out, only Sample SA7 at 400 kPa gave reasonable values for C_α/C_c . All load durations on SA7 except for the last stage, were only a few hours long. Whether or not soil ageing fully explains all of the anomalous C_α/C_c ratios cannot be tested theoretically, because if C_c is in error in every case, the whole data set is thrown into error and theoretical calculations are meaningless.

Despite the above, in keeping with the C_α/C_c concept of secondary compression, regardless of whether loading is virgin or re-compression, one would expect that the C_α/C_c ratio for a particular soil remains constant. The high and variable C_α/C_c ratios for 3 out of 4 samples suggest that for an individual, supposedly virgin load stage, the C_α being measured was a virgin C_α, , but the C_c was a partial recompression (and varying) C_c . Alternatively some kind of yielding other than secondary compression may have taking place. Whatever the mechanism, it is clear that the compression parameters given by the oedometer tests were ill-defined. In view of this, it was decided to evaluate C_c by a number of alternative methods. These are discussed in the following sections of this paper.

4.3 Estimating C_c by Correlation

Suggested empirical correlations between Cc and index properties given in CUR (1996) and Bowles (1984) are:

$$C_c = 0.54(e_0 - 0.35) \qquad (1)$$
$$C_c = 0.01(w_0 - 7.55) \qquad (2)$$
$$C_c = 0.30(e_0 - 0.27) \qquad (3)$$

Using the index properties for the samples, "correlated C_c" values according to the three relationships above are shown in Table 5. The last column gives CR calculated from the average of the three formulae and the sample initial void ratios.

Table 5. C_c Estimated from Index Properties

	Lab C_C	C_C from Correlation			av. CR
Sample	Lab	(1)	(2)	(3)	
A7	0.05	0.14	0.20	0.10	0.07
SA1	0.04	0.31	0.27	0.20	0.13
SA2	0.10	0.68	0.52	0.40	0.20
SA3	0.13	0.57	0.43	0.34	0.14

The results of Table 5 show that true virgin C_c for all of the samples could be between 3 and 5 times larger than the laboratory measured C_c.

5. EFCPT TESTING

Figure 6 shows average cone resistance q_c of the soft layer against friction ratio (FR) on interpretation curves adapted from those published by Robertson (1990). Results of 58 probes are shown; 12 comprising pre-surcharge testing and 46 post-surcharge probes. The location of the probes are shown on the Site Plan (Figure 7). It can be seen from Figure 6 that the soft layer material falls in the range of fine grained sensitive soil, and that measurable improvement in cone resistance was brought about by surcharging

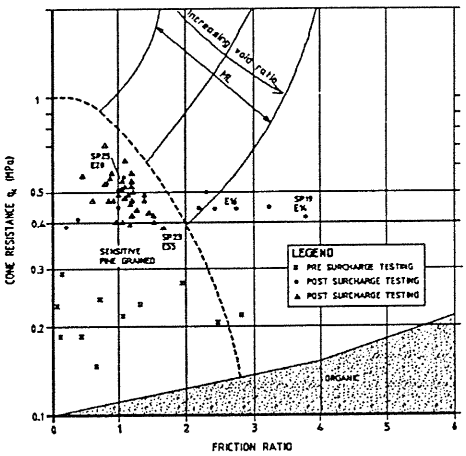

Figure 6. Cone Resistance vs Friction Ratio

6. SETTLEMENT MONITORING

Settlement plates were installed on the surface of the natural ground prior to permanent fill placement, and an initial level reading taken. Regular monitoring began only after the design ground levels had been reached and the 1.5m surcharge placed. A number of plates were re-installed on the final (design) ground surface to monitor post surcharge creep.

Figure 8 shows the settlements plotted against time, of fifteen monitoring plates. In all cases the sudden increases were associated with de-watering of trenches or canals located within a distance of 20m

Figure 9 shows records of post surcharge creep monitoring plates. Within the tolerances of normal leveling, the no significant trend of either rebound or settlement was observed within the seven month period. This observation supports the laboratory evidence that recompression stiffness is very high.

6. MODELING

6.1 From q_c

Settlement and strength modeling of the soil profile beneath a number of settlement plates was carried out, using information from pre-surcharge and post-surcharge EFCPT probes taken near settlement plates.

Three separate loading events were identified; filling to design ground level, surcharge application, and canal de-watering. Fill heights and de-watering levels were worked out from the construction program.

The models used a simplified stress path analysis, calculating settlement for the first loading event using M_o, and then calculating new σ_v' and M values to use for the next loading event, and so on until a theoretical final stress state was reached, which gave a theoretical parameter M_f and a backfigured q_{cf}. Theoretical q_{cf} was then compared with the measured post surcharge q_{cf}.

The following empirical relationships were used in the analysis:

$$M_0 = 3\, q_{c0} \tag{4}$$

from Meigh (1987) and others

$$M_i = m \times \sigma_v' \tag{5}$$

from Janbu (1967, 1985), where Mi is constrained modulus for intermediate loading events, and m is a dimensionless modulus number which ranges from 9-18 for normally consolidated (NC) clays and from 25-50 for NC silts. A value of 18 for m gave good agreement in the model.

The initial and final states of the a typical analysis is The initial and final states of a typical analysis are shown in Table 6.

Table 6. Model Results for Plate SP25

	Initial			Final			
Layer	σ_{vo}	q_{co}	M_o	Δh (mm)	M_f	q_{cf} (1)	q_{cf} (2)
Sand	10	3	10	8	25	6	8
Soft Layer	35	0.2	0.6	197	1.5	0.5	0.5
Lower Sand	53	2	7.6	13	12	3	3
Dense Sand	67	7	26	2	40	10	8

q_{cf} (1) Theoretical q_{cf} (2) Measured

The modeling in every case gave good agreement with field settlements and q_c values, and indicated that M is far smaller than obtained from laboratory tests.

6.2 From Strain

Using settlement data, and the geometry of the profile from EFCPT probes at each settlement plate, fifteen plate positions were analysed and M, CR, C_v and k_v for the soft layer were calculated at each position. Estimates of ε_α were then made from CR by adopting a value of 0.05 for the ratio ε_α/CR (Mesri, 1977). It was assumed in the calculation that 100% consolidation had been achieved in the one month of pre-load. The calculations were based entirely on the known (cumulative) stress increase, the measured total settlement, and the inferred initial thickness of the soft layer. The following relationships were used in the calculations:

$$M = \Delta\sigma_v' / (\Delta h/H_0) \tag{6}$$

$$CR = (\Delta h/H_0)/\log(\sigma_{vf}'/\sigma_{vi}') \tag{7}$$

and C_v and k_v were calculated from the usual equations, assuming two way drainage, and a consolidation time of 1 month.

Results of the analysis are shown in Table 7, and many intermediate stages of the calculation, such as calculation of initial effective stress etc. have been omitted for brevity, however the surcharge ratio achieved at each plate location is given.

Examination of Table 7 shows that:

- the CR values are much higher than those given by the oedometer tests.
- the CR values are similar in range to the values predicted in Table 5.
- Correspondingly, modulus values are much less than given by the oedometer, but fit better with correlations to EFCPT .
- C_v and k_v values depend entirely on the estimated time to EOP. Both C_v and k_v are probably too low for the known grading of the material.

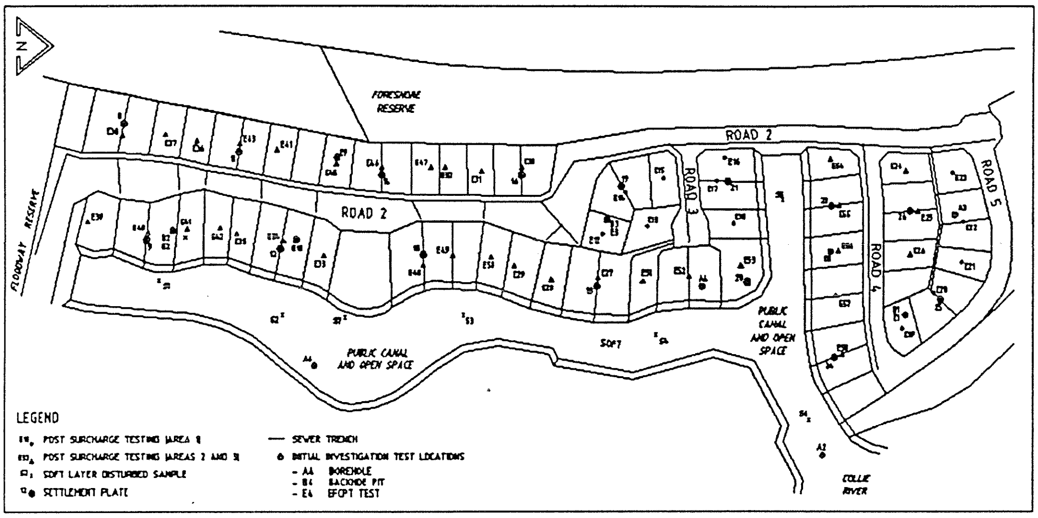

Figure 7. Site Plan

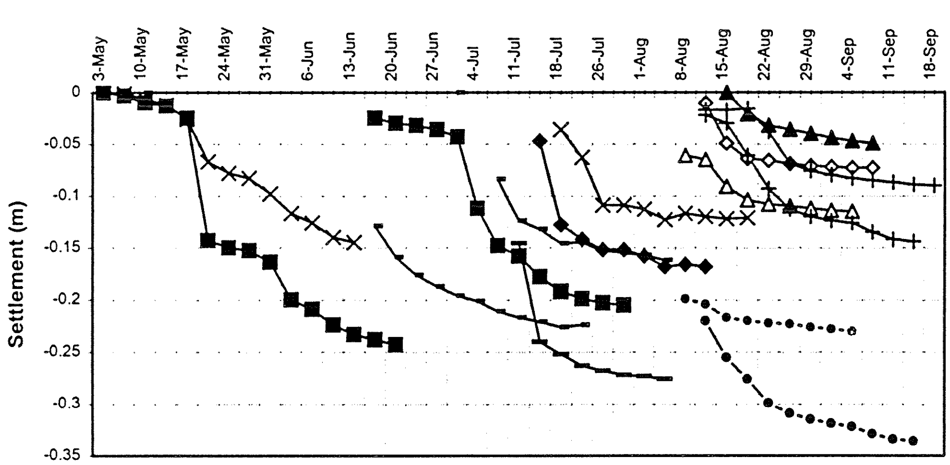

Figure 8. Settlement Plate Records

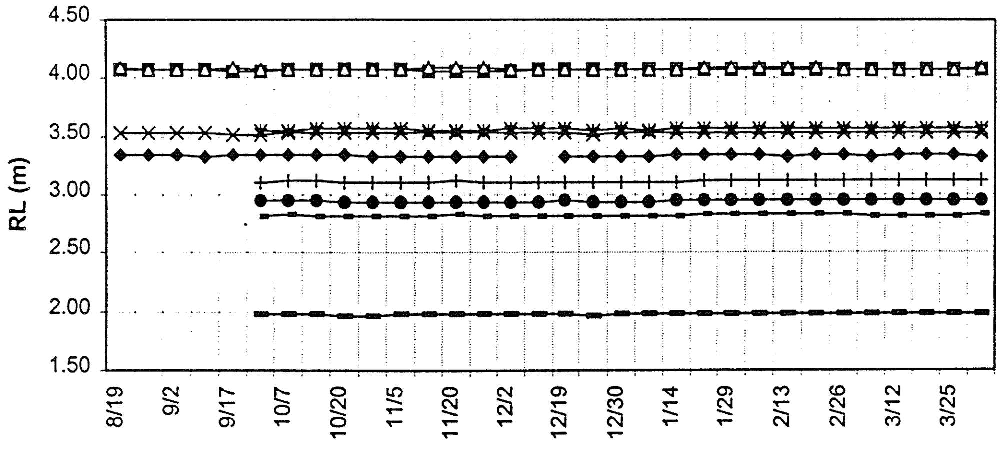

Figure 9. Creep Monitoring Plate Records

Table 7. Field Evaluation of Soft Layer Virgin Compression Characteristics

Plate	H_0 (m)	Δh (m)	$\Delta\sigma_v'$ (kPa)	$\Delta h/H_0$	M (MPa)	CR	C_v (m^2/year)	k_v ($\times 10^{-9}$ m/s)	ε_α	R_s
SP19	2.78	0.178	61.8	0.064	0.96	0.133	23	7.5	0.0067	0.20
SP21	2.80	0.102	61.8	0.037	1.69	0.082	24	4.3	0.0041	0.31
SP25	2.75	0.153	65.4	0.056	1.18	0.109	23	6.0	0.0055	0.32
SP8	2.61	0.214	88	0.082	1.08	0.103	20	5.9	0.0051	0.46
SP9	1.46	0.064	78	0.044	1.79	0.051	6	1.1	0.0025	0.37
SP11	2.30	0.098	79.6	0.043	1.87	0.064	16	2.6	0.0031	0.42
SP12	1.69	0.092	79.8	0.054	1.47	0.064	9	1.8	0.0033	0.36
SP14	2.29	0.190	88.0	0.083	1.06	0.079	16	4.6	0.0040	0.37
SP15	1.97	0.067	79.8	0.034	2.35	0.043	12	1.5	0.0021	0.34
SP16	2.30	0.104	76.6	0.045	1.70	0.065	16	2.9	0.0033	0.26
SP18	2.21	0.107	81.6	0.049	1.68	0.05	15	2.7	0.0025	0.37
SP20	2.63	0.031	67.2	0.012	5.68	0.023	21	1.1	0.0010	0.32
SP23	2.97	0.270	69.0	0.091	0.76	0.185	26	11	0.0092	0.30
SP24	2.49	0.092	70.8	0.037	1.92	0.059	19	3	0.0030	0.34
SP26	3.17	0.175	48.6	0.055	0.88	0.166	30	11	0.0084	0.12

- the values for ε_α are generally smaller than most of the values given by the oedometer tests, despite the fact that pressure increments in the field were often greater than 2, while pressure increments in the laboratory were generally around 1.

7. CONCLUSIONS

From evaluations of compression characteristics of the Pelican Point soft layer by various methods, the following conclusions were made:

- The soft layer is a highly compressible material that exhibits fast primary compression and subsequent creep dominated behaviour.
- C_c and M values derived from standard oedometer testing and interpretation did not generally represent true virgin characteristics. Test yielded depressed values for C_c, C_v and k_v, and elevated values for M and C_a/C_c. The reason for the anomalies could not be fully explained by the author, although it is evident that soil ageing because of sustained compression has contributed to the curious results.
- The high recompression stiffness displayed in laboratory tests, and negligible post surcharge creep movements, indicate that a recompression modular ratio of at least 4 would be conservative, but appropriate for design purposes.
- Field evaluation of C_c and M from field strain modelling gave values that were in good agreement with values correlated from Index properties.

On the basis of the foregoing, the parameter set calculated by field strain modeling was adopted as the most appropriate for design purposes. Predictive settlement analyses were carried out using recompression values of the virgin parameters in Table 7, appropriately modified to reflect the anticipated future loading.

In conclusion , it can be said that there exist in WA and in other places highly compressible sediments whose compression characteristics cannot easily be analysed by standard oedometer testing and interpretation. Factors which affect the measured compression characteristics are likely to be sample disturbance, rate of strain, scale effects and a fast (less than 24 hr) primary compression stage.

The practising engineer on a limited budget may not have easy access to specialised equipment and operator, and laboratory test results using standard means could yield misleading parameters.

For loose inorganic clayey sediments with high void ratios, it is recommended that correlations with index properties, field testing, and settlement measurements be used as a check on oedometer test results.

9. REFERENCES

Aboshi, H. (1991) On the Prediction of Consolidation Settlement Using Laboratory Data, *Geo-Coast'91*, p 1029

Asaoka, A. (1978) Observational Procedure of Settlement Predictions, *Soils and Foundations*, Vol 18, No. 4, p 87

CUR (1996) *Building on Soft Soils*, AA Balkema/Rotterdam/Brookfield

Bowles, J.E. (1984) *Physical and Geotechnical Properties of Soils*, McGraw Hill, 1984

Burland, J.B. and Wroth, C.P. (1975) Settlement of Buildings and Associated Damage, *UK Building Research Establishment*, CP33/75

Jamilkowski, M, Lerouil, S. and Lo Presti, D.D.F (1991) Design Parameters from Theory to practise, *Geo-Coast'91*, p877

Jamilkowski, M. and Leroueil, S. (1991) Exploration of Soft Soil and Determination of Design Parameters, *Geo-Coast'91*, p969

Janbu N. (1967) Settlement Calculations Based on Tangent Modulus Concept, *Bulletin No.2, Soil Mechanics and Foundation Engineering Series*, The Technical University of Norway, Trondheim, p 57

Janbu, N. (1985) Soil Models in Offshore Engineering, 25th Rankine Lecture, *Geotechnique*, Vol 35, No. 3, p 241

Karmon, M. (1991) Soil Improvement of Soft Clay Ground, *Geo-Coast'91*, p 1043

Lamb, T.W. and Whitman, R.V. *Soil Mechanics*, John Wiley & Sons, 1979

Lerouil, S. and Jamolkowski, M. (1991) Exploration of Soft Soil and Determination of Design Parameters, *Geo-Coast'91*, p 969

Meigh, A.C. (1987) *Cone Penetration testing Methods and Interpretation*, CIRIA Ground Engineering Report: In-situ Testing, Butterworths,

Mesri, G., Stark, T.D., Ajlouni, M.A., & Chen, C.S., (1997) Secondary Compression of Peat with or without Surcharging, *Journal of Geotechnical and Environmental Engineering*, May 1997, p 411

Mesri, G. and Feng, T.W. (1991) Surcharging to Reduce Secondary Settlements, *Geo-Coast'91*, p 359

Mesri, G. (1991) Prediction and Performance of Earth Structures on Soft Clay, *Geo-Coast'91*, p 1011

Mesri, G. and Godlewski, P.M. (1997) Time and Stress-Compressibility Interrelationship, *ASCE Journal of the Geotechnical Engineering Division*, p 417

Nakase, A. (1991) The Importance of Geotechnical Engineering in Coastal Development, *Geo-Coast'91*, p 867

Robertson, P.K., (1990) Soil Classification using the Cone Penetration Tests, *Canadian Geotechnical Journal*, Vol. 27, p 151

Shogaki, T. and Kogure, K (1991) Evaluation of Consolidation Parameters in Standard Consolidation Test, *Geo-Coast'91*, p 81