SETTLEMENT CHARACTERISTICS OF COODE ISLAND SILT

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ABSTRACT

Coode Island Silt is a Quaternary age alluvial deposit of the Yarra River delta in Melbourne. It is a highly compressible and only slightly overconsolidated deposit, which exhibits significant primary consolidation and creep settlements. This paper discusses the settlement characteristics of this deposit based on laboratory test data and field observations, mostly for a settlement assessment using conventional one dimensional consolidation theory. The rate of creep settlement based on field observations is also discussed.

1 INTRODUCTION

The geological setting of the Coode Island Silt (CIS) is well described in Neilson (1996). Although developments in the areas of CIS deposits have been carried out since late 19th century, most of the earlier developments were limited to port and isolated industrial facilities until the latter part of last century, possibly due to the highly compressible nature and lower strength of CIS. However, major developments have been carried out in these areas in the recent past, as these areas are located close to the city and there is a growing demand for such developments.

Foundation design for major structures in the areas of CIS, which involve pile footings, would not require a detailed settlement assessment of CIS, other than to check the potential for downdrag loads. However, ancillary facilities associated with the major developments such as roads, underground services etc may require a detailed settlement assessment. Engineering properties of CIS were presented in Ervin (1992) and Ervin (1996), which include most of the parameters discussed in this paper. This paper provides further details and additional information specifically with regard to settlement assessment in CIS.

Settlement assessment in soft clay deposits will require the following key parameters:

- Preconsolidation pressure (or over consolidation ratio)
- Compression and re-compression indices
- Initial void ratio / unit weight
- Coefficients of consolidation / hydraulic conductivity
- Effective drainage path length
- Secondary compression index

The first three parameters will be required to assess the magnitude of primary consolidation settlement, the next two will be required to assess the rate of primary consolidation settlement and the last parameter will be required to assess the long term secondary compression (creep) settlement. These parameters, except the effective drainage path length, are mostly assessed using laboratory testing of assumed representative samples. Some of these parameters are also obtained from in-situ testing, such as piezocone penetration testing (CPTU) using empirical correlations. These parameters can also be obtained from back analysis of data from trial embankments and past projects. An assessment of these parameters for the CIS deposit is the main focus of this paper.

2 PRECONSOLIDATION PRESSURE

Assessment of preconsolidation pressure using conventional one dimensional consolidation testing with incremental loading is somewhat subjective, unless the loading increments are kept reasonably small in the vicinity of the preconsolidation pressure.

Ladd and DeGroot (2003) highlight the benefits of constant rate of strain (CRS) testing (Wissa et al. 1971, ASTM D4186) to obtain preconsolidation pressure and compression index values compared to conventional one dimensional consolidation testing with incremental loading. They claim that the CRS test provides a continuous compression curve, continuous unambiguous values of coefficient of consolidation (c_v) and hydraulic conductivity (k_v) and the test can be completed in far less time. The only disadvantage of CRS test is that it is not suited to obtain a measure of the secondary compression index. The results of a CRS test on a Lacustrine Clay sample (moisture content 72%, Liquid Limit 75%, Plasticity Index 47%) from Northern Ontario, Canada is presented in Figure 1 to highlight the nature of the results that can be obtained from CRS test and the typical variations of various parameters in soft clay deposits. The symbol u_e in the figure represents excess pore pressure at the base of the sample during the CRS test. To the author's knowledge a CRS test has never been carried out on a CIS sample. The author believes that CRS tests be carried out on CIS samples in the future on projects where settlement in CIS is a critical aspect. The results from a CRS test will complement the existing knowledge on CIS characteristics.

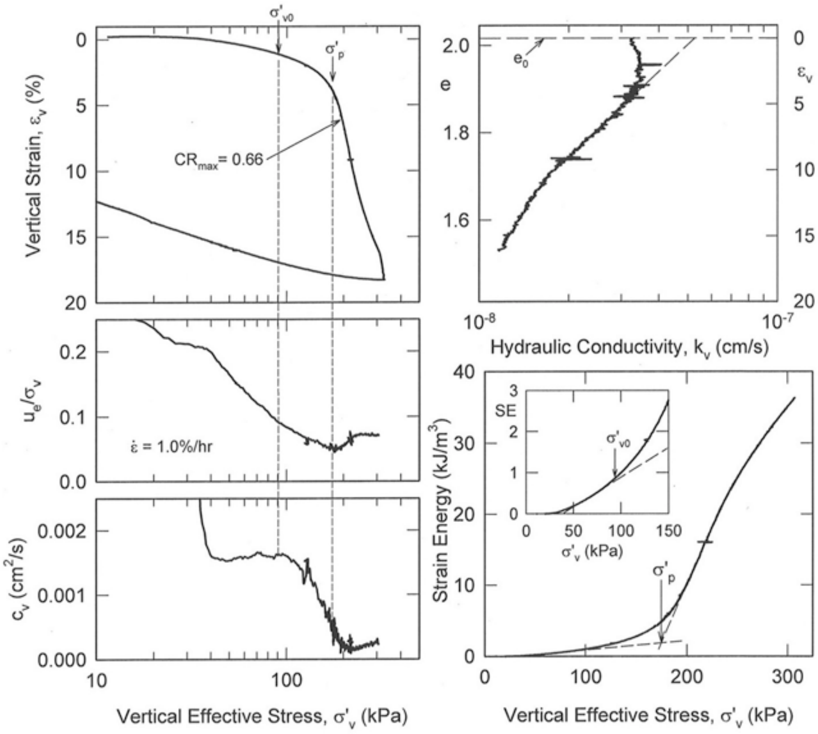

Figure 1: Results of a CRS test (adapted from Ladd and DeGroot, 2003)

The preconsolidation pressure in the CIS deposit in most parts of Melbourne is typically assumed to be about 10 kPa to 25 kPa higher than the effective vertical stress, although there is some evidence to suggest higher overconsolidation in the upper part of CIS deposit. There are also areas of no apparent preconsolidation pressure. It should be noted that past construction activities in the general area of the site can have a considerable influence on preconsolidation pressure. For example, stockpiling of materials in a construction site and groundwater drawdown associated with past underground construction activities can alter the preconsolidation pressure. Groundwater drawdown associated with the construction activities can also extend to considerable distances through old river channels as observed during the construction of Melbourne's City Link (1997-2001).

On a recent project in Port Melbourne, the observed settlements associated with some groundwater drawdown were significantly lower than expected. Further investigations revealed that the possible reason for this behaviour was groundwater drawdown in the area in the 1890s. A document from the Melbourne Metropolitan Board of Works (MMBW) archives with regard to the construction of the Melbourne Main Sewer in Port Melbourne in 1894 states "Over the greater portion of this contract the surface of the streets settled, and numerous houses along both sides of the street were damaged – in fact, more claims for damage were settled in this single contract than on all the rest of the contracts of the Board". Therefore for a realistic settlement assessment in CIS, it is important to understand the historical activities in the vicinity or even some distance away from the site, depending on the hydrogeological conditions.

Olson (1998), in his Terzhaghi lecture, suggested the use of undrained shear strength to assess the preconsolidation pressure. The undrained shear strength can be more reliably obtained using in-situ vane shear testing or cone penetrometer testing (CPT). This approach is also discussed in Ladd and DeGroot (2003).

The undrained shear strength (S_u) can be obtained from cone tip resistance (q_c) using the following empirical equation:

$$S_u = (q_c - \sigma_v) / N_k \qquad \textbf{(1)}$$

Where σ_v is the total vertical stress and N_k is an empirical constant. Ervin (1996) states that N_k values for CIS deposit vary between 10 and 15, but usually closer to 15.

Using the undrained shear strength – effective vertical stress relationship suggested by Ladd and DeGroot (2003) for normally consolidated inorganic clays and assuming a N_k factor of 15, the preconsolidation pressure (P'_c) can be obtained using the following equation:

$$P'_c = S_u / 0.22 = (q_c - \sigma_v) / 3.3 \quad \textbf{(2)}$$

Mayne at al (2009) show that an equation similar to the above ($P'_c = (q_c - \sigma_v) / 3$) gives a reasonable estimate of P'_c as shown in Figure 2. The value of 0.22 in Equation (2) may range from 0.20 to 0.25, depending on the soil type and plasticity. The scatter that can be observed in Figure 2 may be consistent with this range of values.

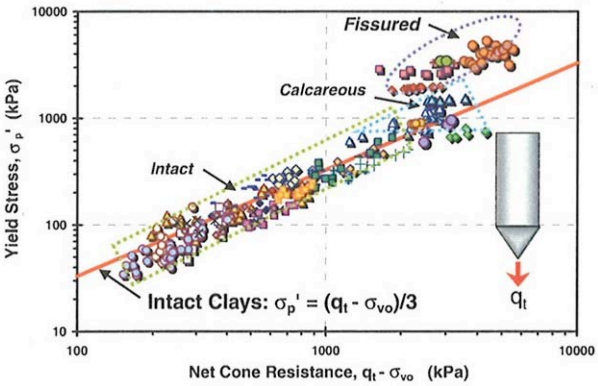

Figure 2. Preconsolidation pressure relation with cone tip resistance (adapted from Mayne et. al., 2009)

A typical q_c profile of a site in Southbank and an assessment of P'_c as per the above approach (using an N_k factor of 11) are shown in Figure 3. The results suggest that the P'_c varies with depth and an average P'_c of about 20 kPa above the effective vertical stress.

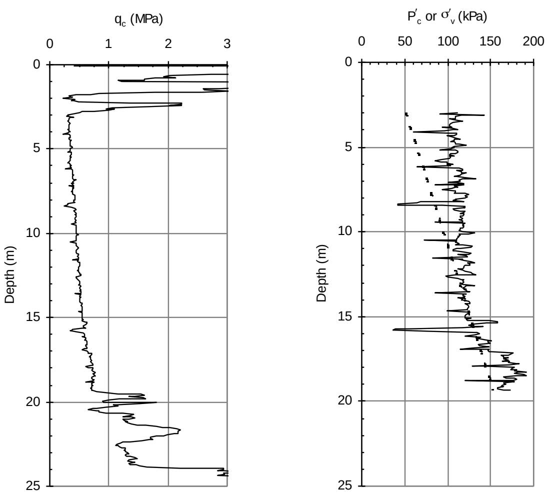

Figure 3: Assessment of preconsolidation pressure using cone tip resistance for a site in Southbank.

3 COMPRESSION AND RE-COMPRESSION INDICES

Reasonable estimates of compression and re-compression indices can be obtained from conventional oedometer tests. Ervin (1992) presented a plot of moisture content (w_n) versus compression index (c_c) values. This chart has been updated with additional data points from projects where the author was involved and is presented in Figure 4, in natural scale (Figure 4a) and log-log scale (Figure 4b) plots.

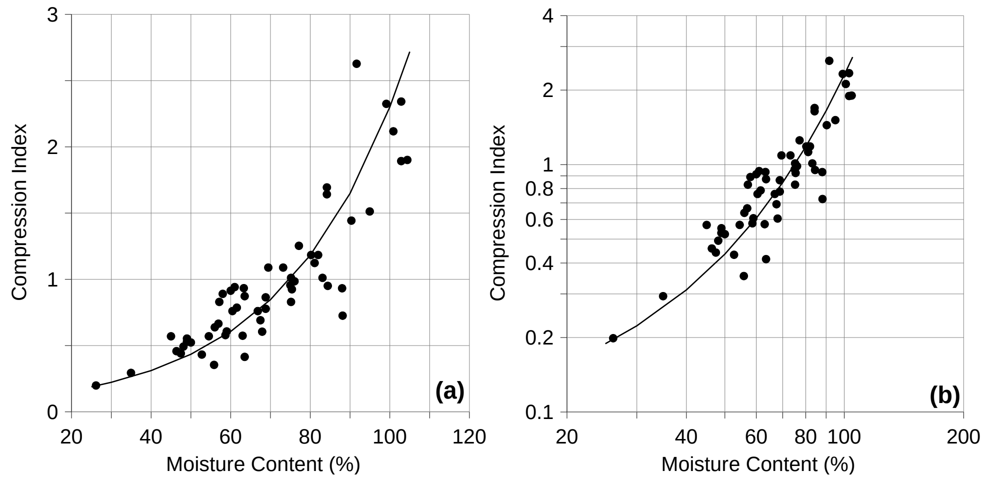

Figure 4 : Compression index versus moisture content

Correlations between compression index and moisture content for soft clays have been suggested in the literature (for example, Koppula (1981)). Data points presented in Figure 4 suggest that a reasonable correlation exists between moisture content and compression index for CIS as shown by the line, which can be represented as:

$$c_c = exp\ [(w_n - 75) / 30] \qquad \textbf{(3)}$$

The above correlation may be used as guide to obtain compression index values from moisture content values and also as a check for values obtained from oedometer tests. The author also found that this correlation is reasonably applicable for inorganic clays in other parts of the world. Figure 5 shows the above correlation for data points presented in Mesri et al (1997).

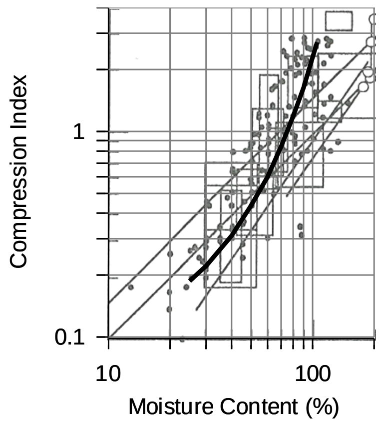

Figure 5: Compression Index - Moisture Content Correlation on Data Points from Mesri *et al.* (1997).

Generally c_c is assumed as a constant for settlement calculations over a given stress range. However, in reality c_c will reduce with reducing void ratio (or moisture content) as shown in Figure 4, which possibly is the reason for the curvy part typically observed in *e-log p* plots at higher stresses.

With regard to the validity of a unique relationship between moisture content and c_c, a question can be raised whether all the samples tested must have similar overconsolidation stress. The change in void ratio due to loading up to the preconsolidation pressure is expected to be small and hence it may not have a significant influence on this relationship. This could be one of the many reasons for the scatter in the data points in Figures 4 and 5.

Ervin (1992) found that the re-compression ratio, $c_r/(1+e_0)$, in CIS ranges between 0.004 and 0.067 with an average of about 0.021. The ratio of the re-compression ratio to the compression ratio ranged from about 1% to 33% with an average of about 8%. The additional data points that the author have collected do not alter these values.

4 INITIAL VOID RATIO

For a saturated soil, the void ratio (e) can be calculated from specific gravity (G_s) and moisture content (w_n) as:

$$e = G_s\, w_n \qquad \textbf{(4)}$$

Information presented in Ervin (1992) and other available data suggest that the average specific gravity of CIS is about 2.6. Based on this, for moisture content ranging from 40% to 100%, the saturated unit weight will vary from about 14 kN/m^3 to about 17.5 kN/m^3. Typically, the saturated unit weight of CIS is assumed to be 16 kN/m^3, which corresponds to a moisture content of about 58% and a void ratio of about 1.5.

5 COEFFICIENT OF CONSOLIDATION

The vertical coefficient of consolidation (c_v) is generally defined as:

$$c_v = k_v / (\gamma_w \; m_v) \qquad \textbf{(5)}$$

Where k_v is the vertical hydraulic conductivity, γ_w is the unit weight of water and m_v is the coefficient of volume compressibility.

The results of the CRS test presented in Figure 1 show the variations of k_v and c_v values that can be typically expected in a soft clay. The CRS test provides a continuous record of these values over a range of applied stress. The k_v value decreases with increasing stress (or reducing void ratio) and in the normally consolidated stress range, log of k_v is proportional to the void ratio. The variation in the c_v value in the normally consolidated stress range is not significant, as the reduction in m_v offsets the reduction in k_v. Generally c_v is assumed as a constant for a given stress range in the consolidation settlement calculation, which is considered to be reasonable. For CIS, the c_v values obtained from oedometer tests in the normally consolidated stress range were found to vary from about 0.1 m^2/year to about 2 m^2/year.

The horizontal coefficient of consolidation (c_h) values, which can be obtained from pore pressure dissipation tests during piezocone testing, is used to estimate c_v. using a c_h/c_v ratio The c_h value may be considered as more reliable compared to the oedometer c_v values, as it would represent the permeability characteristics of a larger area. The c_h values assessed in CIS typically range from about 1 m^2/year to 20 m^2/year, which is about 5 to 10 times higher than the oedometer c_v values in the normally consolidated stress range. It should be noted that the c_h assessment in-situ and c_v assessment in the lab may not be at the same void ratio. Plots of two dissipation tests in CIS are presented in Figure 6. Plot (a) was from a site in Southbank, which suggests c_h of about 2 m^2/year and Plot (b) was from a site in Docklands, which suggests a c_h of about 15 m^2/year.

Generally in alluvial deposits there is an inherent anisotropy and the hydraulic conductivity in the horizontal direction (k_h) is higher than that in the vertical direction (k_v). Ervin and Morgan (2006) indicated that in CIS, k_h/k_v ratio could be more than 100, based on field and laboratory permeability tests. As discussed earlier, it should be noted that variation in the hydraulic conductivity with void ratio may be far greater than that of the coefficient of consolidation.

Day and Woods (2007) suggest k_h/k_v ratio in the CIS deposit could be in range of 3 to 5. However, they also claim that the k_h/k_v ratio appears to be near unity based on the back analysis of the observed settlement behaviour in areas with and without wick drains at a site near the mouth of Moonee Ponds Creek at Victoria Dock.

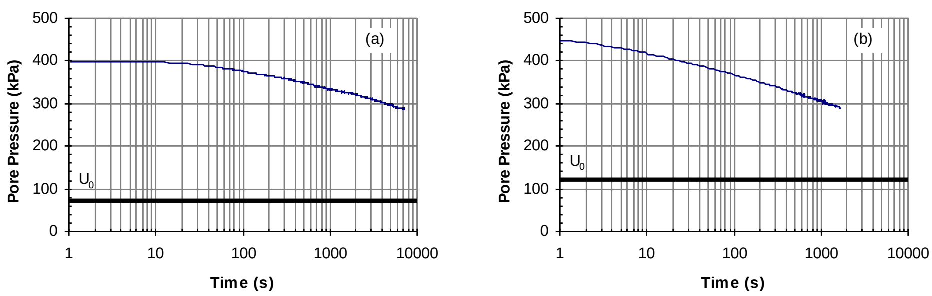

Figure 6: Results of pre pressure dissipation tests in CIS at two sites

There are many case histories in the literature suggesting faster consolidation in the field than that assessed from laboratory c_v values. For example, Leroueil (1988) compiled settlement data from 16 embankments and found that, on average, c_v values obtained from field data were 20 times higher than the laboratory values. Ozcoban et al (2007) presented a unique case history of Ailbey Dam construction near Istanbul in Turkey, where the construction spanned over a period of over 15 years (1967 to 1983) and the monitoring data over 25 years. They also found that the laboratory c_v values were to be about 25 times smaller than those obtained from field settlement measurements. It should be noted however, the rate of settlement (or consolidation) will depend on both the coefficient of consolidation and the length of effective drainage path. It is possible that shorter drainage paths due to the presence of thin sandy layers that naturally occur in alluvial depositional environments may contribute to faster field consolidation. Some field evidence and discussion on the presence of sand layers in CIS is presented below.

6 EFFECTIVE DRAINAGE PATH

Terzaghi's consolidation theory indicates the degree of consolidation as inversely proportional to the square of the drainage path. That is, if the drainage path is reduced by half, the degree of consolidation will increase by four times and therefore the effective drainage path length is an important parameter in assessing the rate of consolidation. However, the effective drainage path length cannot be readily assessed from field testing, especially if the sandy layers in the clay deposit are thin. An estimate of the effective drainage path length may be obtained from back analysis of observed settlement performance.

Thin layers and lenses of fine sands are known to be present in the CIS in many parts of Melbourne. Table 1 summarises the thin sandy layers encountered in a borehole near Swan Street Bridge where detailed logging of continuous tube samples was carried out. The CIS was encountered between about 4.2 m and 20.8 m depths in that borehole.

Table 1: Details of sandy layers in CIS in a continuously sampled borehole

Depth (m)	Thickness (mm)	Nature
5.80	400	Sandy silt
8.61	8	Fine sand
8.69	5	Fine sand
8.79	17	Fine sand
8.89	3	Fine sand
11.88	30	Fine sand
12.18	10	Fine sand
12.42	10	Fine sand
13.41	15	Fine sand
15.53	15	Fine sand
15.56	5	Fine sand
16.30	100	Fine to medium sand
16.92	10	Fine sand
17.60	200	Fine to medium sand
18.12	7	Fine sand
19.00	100	Fine to medium sand

The author found that the rate of primary consolidation settlement in CIS can be approximated as about 50% in about 6 months and about 95% in about 3 years as shown in Figure 7 based on his experience at some sites. This behaviour could be obtained by various combinations of c_v and effective drainage path length values. For example a c_v of 10 m^2/year and an effective drainage path length of 5 m, or a c_v of 1.6 m^2/year and an effective drainage path length of 2 m would result in a settlement-time curve similar to that presented in Figure 7. It should be noted however, that this behaviour may not be applicable to CIS everywhere. Within the Yarra delta, there could be areas of less turbulent depositional environment, where CIS might have been deposited as a more uniform silty clay deposit. Day (2010) indicated that the rate of settlement was much slower than that suggested by Figure 7 in the areas of Beacon Cove, Moonee Ponds Creek and Webb Dock, where he was involved in settlement monitoring in CIS deposits. Karlsurd (2009) also indicated that settlement behaviour in soft clays in Norway, which were deposited in lake type low energy depositional environments, generally matches the assessment based on laboratory c_v values.

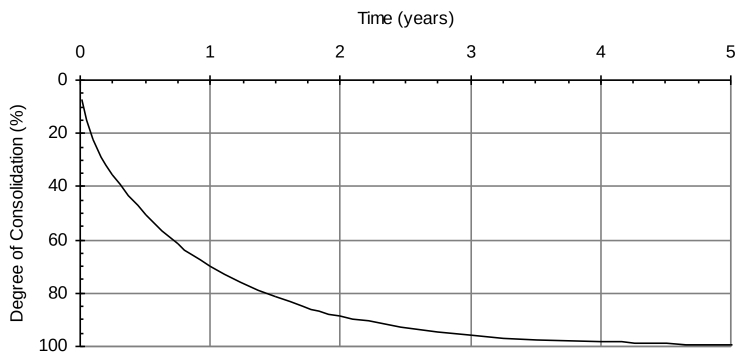

Figure 7: Approximate degree of primary consolidation versus time in CIS deposits.

7 SECONDARY COMPRESSION INDEX

Ervin (1992) presented a comprehensive assessment of the creep characteristics of CIS. A summary plot of secondary compression index (c_α) vs log (applied pressure, P_a / effective overburden pressure, P'_0) presented in his paper is reproduced in Figure 8. Although there is considerable scatter of the data points, an apparent trend of increasing c_α with increasing applied stress was noted in his paper.

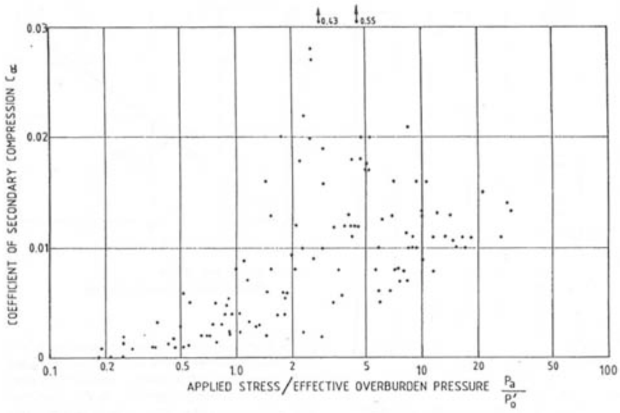

Figure 8: Secondary compression versus applied stress (adapted from Ervin, 1992)

Ervin (1992) also noted that historical measurements of creep settlements over the last century suggest an apparent linear trend rather than diminishing rate on a linear time scale (or linear trend on a log time scale) as postulated in classical soil mechanics. It is possible that construction activities in the past century might have caused small settlements, even in some cases the activities might not have been in close vicinity of the site (for example, minor groundwater drawdowns, storage of materials over a limited period), which would have contributed to this apparent linear trend.

Typically, the rate of creep or long term settlement in CIS appears to vary between about 5 mm to 10 mm per year except in areas where it is relatively thin. Table 2 summarises measurements of creep or long term settlement in CIS reported in the literature and two sites where the author had some involvement.

Table 2. Long Term Settlement Observations

Reference	**Area**	**CIS thickness (m)**	**Period of settlement observation (years)**	**Rate of Long Term Settlement (mm/year)**	**Comments**
Golder Associates (1976)	South Melbourne	15 - 21	16	6 – 13 (mostly close to 10)	Based on MMBW records of building levels between 1960 and 1976
McDonald (1988)	South Melbourne	16	2	18	Settlement observed after about 5 years of some new fill placement, area under continued traffic loading
McDonald and Cimino (1987)	South Melbourne	16	unknown	10	Area used for container and lumber storage
Hutchison and Lamb (1999)	South Melbourne	18 – 20	96	7 – 9	5 data points over 96 years
	South Melbourne	5 – 15	4.5	1.5 - 7	Lower rate may correspond to lesser CIS thickness and vice versa
Ervin et al (2006)	South Melbourne	16	1	8	Performed to obtain background rate of settlement
Author's Involvement	Swan Street Bridge	15	40	10	Based on asphalt coring observations, area under continued traffic loading
Author's involvement	Docklands	15	3	8 - 15	Minor recent filling, under continued traffic loading

As indicated in Ervin (1996), in the absence of strong field evidence of reducing rate of long term settlement, the current conventional wisdom is to agree that a regional creep or long term settlement in the range of 5 mm to 10 mm per would occur in CIS deposits. Creep settlement would be negligible near the fringes of the units where it is relatively thin, say less than 3 m.

8 SOME FIELD OBSERAVATIONS

Settlement observations made at two sites are presented below, just to give some examples of field settlement behaviour in CIS and especially the rate of settlement discussed in Section 6.

8.1 SWAN STREET BRIDGE

The area at the western abutment of Swan Street Bridge, which comprises about 15 m thick CIS, has undergone settlements due to groundwater drawdown. This area has been instrumented with vibrating wire piezometers, magnetic extensometers and surface settlement markers. The area also has been subjected to passive recharge wells to limit the groundwater drawdown in the CIS. Figure 9 presents a schematic plan and cross section of the area and the instruments and the measured responses until about end of 1998. Most of the groundwater drawdown occurred in the bottom 6 m of the CIS deposit.

The parameters obtained from oedometer tests of CIS samples and CPT results from the site are presented in Table 3.

Table 3. CIS parameters for a site near Swan Street Bridge

Parameter	Value
Moisture Content	48% - 63%
c_c	0.52 – 0.57
c_r	0.05
$P'_c - \sigma'_v$	~ 0
c_v	0.4 – 1.2 m^2/year -

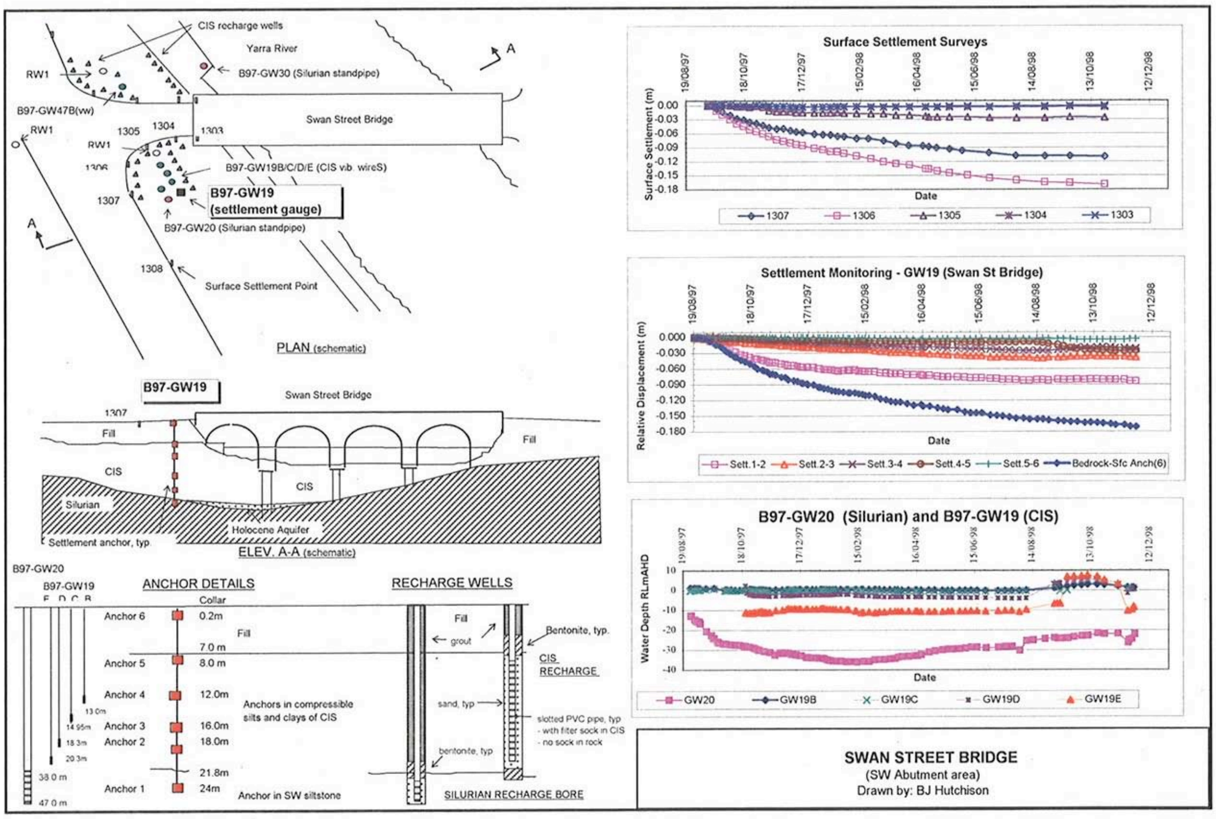

Figure 9: Details of settlement measurements near Swan Street Bridge.

Back analysis of the pore pressure and settlement data suggests a c_v value of about 10 m^2/year for assuming drainage at the top and bottom of the CIS, which is considerably higher than the laboratory c_v values. Alternatively, if we assume an average c_v of about 0.8 m^2/year (average of laboratory values), the effective drainage length should be about 1.4 m within the bottom 6 m of the CIS, which can be argued to be realistic based on the presence of sand layers at the site described in Table 1.

Back analysis of the settlement data also agrees well with the typical rate of settlements suggested by Figure 7, that is about 50% primary consolidation settlement in 6 months and about 95% settlement in 3 years, which is shown in Figure 10.

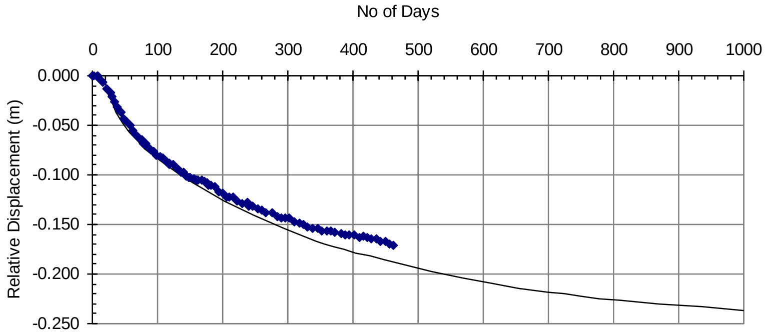

Figure 10: Measured and assessed settlement versus time

8.2 SITE IN DOCKLANDS

A project site in Docklands required the placement of up to 1 m fill for the construction of new roads associated with the development. The CIS at the site is about 23 m thick. The parameters obtained from oedometer tests of CIS samples and CPT results from the site are presented in Table 4.

Table 4: CIS parameters for a site in Docklands

Parameter	Value
Moisture Content	56% - 61%
c_c	0.61 – 0.64
c_r	0.03 – 0.09
$P'_c - \sigma'_v$	~ 10 kPa
c_v	0.3 – 1.4 m^2/year -
c_h	10 – 17 m^2/year

Typical results of settlement monitoring for about a 10 months period are shown in Figure 10. Back analysis of the settlement data again agree well with the typical rate of settlements suggested by Figure 7, that is about 50% primary consolidation settlement in 6 months and about 95% settlement in 3 years, which is shown by the line in Figure 11.

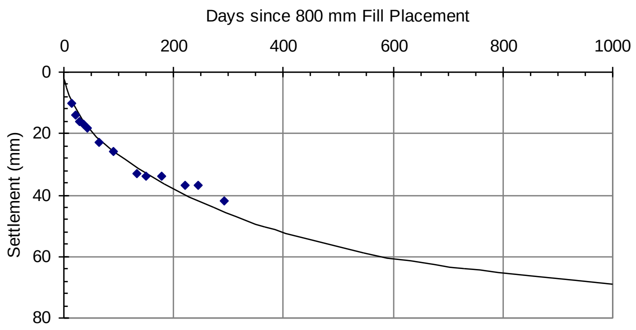

Figure 11: Measured and assessed settlement versus time

9 CONCLUDING REMARKS

Key parameters of CIS that will be required for a settlement assessment using conventional one dimensional consolidation theory are discussed in the paper. It appears that a reasonable assessment of the magnitude of settlement can be made by simple moisture content measurements and using the moisture content-compression index relationship presented. A detailed assessment of the tip resistance obtained in cone penetration test can provide an insight into the preconsolidation stress. It is believed that the depositional environment of the CIS and the associated minor variations in its composition would have an influence on the primary consolidation settlement-time behaviour. The author's experience on some of the sites suggests that settlement-time behaviour in CIS may be approximated as about 50% in about 6 months and about 95% in about 3 years. However, this will not be true for areas where CIS was deposited in lake type low energy depositional environments.

The remarks provided above should be considered to make a general assessment only. Although these remarks may hold for CIS in some parts of Melbourne, local variations are highly likely. There is no substitution for a detailed geotechnical investigation and monitoring, if settlement in CIS is identified as a major issue for a project.

10 ACKNOWLEDGEMENTS

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11 REFERENCES

ASTM D4186 – 06. Standard Test Method for One-Dimensional Consolidation Properties of Saturated Cohesive Soils Using Controlled-Strain Loading. American Society for Testing and Materials, Standard D4186.

Day, R. A. and Woods, P. (2007). Verification of Consolidation Parameters of a Near-Normally Consolidated Clay by Back-Analysis of and Instrumented, Wick-Drained Reclamation. Proc. 10th ANZ Conf. on Geomech., Vol. 2, pp 54-58.

Day, R. A. (2010). Personal Communications.

Ervin, M. C. (1992). Engineering Properties of Quaternary Age Sediments of the Yarra Delta. Engineering Geology of Melbourne, Olds and Seddon (editors).

Ervin, M. C. (1996). Engineering Properties of Coode Island Silt. Paper presented in Aust. Geomech. Society Vic. Div. seminar titled "Building in Coode Island Silt".

Ervin, M. C. and Morgan, J. R. (2006). Groundwater Control around a Large Basement. Aust. Geomech. J., Vol. 41, No. 3, pp 51-62.

Ervin, M., Benson, N., Morgan, J. and Pavlovic, N. (2006). Melbourne's Southbank Interchange – A Permanent Excavation in Compressible Clay. Aust. Geomech. J., Vol. 41, No. 3, pp 25-40.

Golder Associates. (1976). Geotechnical Investigation, South Melbourne. Internal Report.

Hutchison, B. J. and Lamb, I. A. (1999). Construction Management of Hydrogeological Aspects of Melbourne's City Link Project, Proc. 10th Aust. Tunneling Conf., pp 35-45.

Koppula, S. D. (1981). Statistical Estimation of Compression Index. ASTM Geotechnical Testing J., Vol. 4, No. 2, pp 68-73.

Karlsurd, K. (2009). Personal Communications.

Ladd, C.C. and DeGroot, D.J. (2003). Recommended Practice for Soft Ground Site Characterization – Arthur Casagrande Lecture. Proc. 12th Pan-American Conf. on Soil Mech. and Geot. Engr., Massachusetts Institute of Technology, MA, USA.

Leroueil, S. (1988). Recent Developments in Consolidation of Natural Clays. Can. Geot. J., Vol. 25, No.1, pp 85-107.

Mayne, P. W., Coop, M. R., Springman, S. M. Huang, A. and Zonberg, J. G. (2009). Geomaterial Behaviour and Testing – State of the Art paper. Proc. 17th Int. Conf. on Soil Mech. and Geotech. Engr., Vol. 4, pp 2777-2872.

McDonald, P. (1988). The Real World of Embankment Settlement. Proc. 5th ANZ Conf. on Geomechanics, pp 110-117.

McDonald, P. and Cimino, D.J. (1984). Settlement of Low Embankments on Thick Compressible Soil. Proc. 4th ANZ Conf. on Geomech., Vol. 1, pp 310-315.

Olson, R. E. (1998). Settlement of Embankments on Soft Clays – The Thirty-First Terzaghi Lecture. J. of Geotech. and Geoenviro. Engr., ASCE, Vol. 124, No. 8, pp 659-669.

Ozcoban, S., Berilgen, M.M., Kilic, H., Edil, T.B. and Ozaydin, I.K. (2007). Staged Construction and Settlement of a Dam Founded on Soft Clay. J. of Geotech. Eng., ASCE, Vol. 133, No.8, pp. 1003-1015.

Mesri, G., Stark, T. D., Ajlouni, M. A. and Chen, C. S. (1997). Secondary Compression of Peat with or without Surcharging. J. of Geotech. and Geoenviro. Engr., ASCE, Vol. 123, No. 5, pp 411-421.

Neilson, J. L. (1996). The Geological Setting of the Coode Island Silt. Paper presented in Aust. Geomech. Society Vic. Div. seminar titled "Building in Coode Island Silt".

Wissa, A.E.Z., Christian, J.T., Davis, E.H. and Heiberg, S. (1971). Consolidation at Constant Rate of Strain. J. of the Soil Mech. And Found. Div., ASCE, Vol. 97, No. 10, pp 1393-1413.