

# PRELOAD DESIGN, PART 1 – REVIEW OF SOIL COMPRESSIBILITY BEHAVIOUR IN RELATION TO THE DESIGN OF PRELOADS

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## ABSTRACT

The method of treating soft soils by preloading has been used for over a century, and is still widely used today as one of most common form of ground improvement technique. Yet, every now and again, post-construction settlements have been observed to be more than those predicted after preloading. The author believes, in most cases, the poor preload performance is probably associated with lack of understanding of the time-dependent compressibility behaviour of the soft soils. And in particular, the behaviour of secondary consolidation (or creep) is still not well understood despite extensive research and numerous constitutive models that have been developed. The availability of powerful commercial computer programs does not help if they are used indiscriminately when the fundamental principles are not well understood.

Part 1 of this paper provides a review of the factors that influence creep. The dependency of creep on stress level and stress history expressed in terms of the over-consolidation ratio (OCR) is discussed, followed by a discussion on the commencement of creep. A brief overview of time-dependent consolidation and creep settlement analysis methods is provided, followed by a summary of the preload design approach given by Mesri (1991) that illustrates the possibility of the occurrence of higher creep rate some time following preloading.

In Part 2 of this paper, an analytical approach based on Bjerrum's (1967) time line model, or principle of "artificial aging" will be presented for preload design to limit post construction settlement, and a preload design example is discussed to illustrate the importance of geological and stress history on post-preload settlement behaviour.

## 1 INTRODUCTION

Preloading is one of most common and economical forms of ground improvement. If the intention of the preload is to eliminate the post-construction primary consolidation, the design of the preload is relatively straightforward. In this case, the preload must result in an effective stress increase, over the entire compressible stratum, that is, at least equal to the final effective stress expected under the design loading condition. However, if it is required to reduce post-construction secondary consolidation (creep) to a specified limit, a surcharge is required to be applied to preconsolidate the soil. The surcharge must produce a preconsolidation pressure greater than the final effective stress. The resulting over consolidation ratio will inevitably vary with depth and this complicates the preload design procedure, particularly for deep soft soil profiles. In this paper, the term "preloading" will be used interchangeably with the term "surcharging" to have the same meaning of preconsolidation to reduce post-construction creep settlement.

The author believes that one of the least understood aspects of soil behaviour in soil mechanics, creep, is responsible for either the success or failure of preloads. Of course, primary consolidation is also important, but at least its behaviour is better understood and more readily predictable and measured, unless masked by simultaneous creep during primary consolidation. Therefore, before launching into designing preloads, a good understanding of time-dependent settlement behaviour of soft clays, and the influence of geological and stress history on creep strain rates, is considered to be of critical importance.

Due to inconsistencies in published literature, text books and the test procedure of Australian Standard AS1289.6.6.1 (1998), the following terminology to describe creep will be used in this paper to avoid confusion:

$C_{\alpha}$  = creep index, and not the creep strain rate (or coefficient of secondary consolidation as defined in AS1289.6.6.1, 1998).

$C_{\alpha\epsilon}$  = creep strain rate, or coefficient of secondary consolidation as defined in AS1289.6.6.1 =  $\Delta H/H$  per log time cycle (note: some text books and AS1289.6.6.1 uses the symbol  $C_{\alpha}$  for this quantity),  $C_{\alpha\epsilon} = C_{\alpha}/(1+e_0)$  where  $e_0$  = initial void ratio.

## 2 BACKGROUND INFORMATION

### 2.1 GEOLOGICAL HISTORY AND COMPRESSIBILITY OF SOFT CLAYS

Traditionally, time-dependent settlements of soft clays are separated into two components: (a) primary consolidation under increasing effective stress, and (b) secondary consolidation (or creep) under constant effective stress.

In relation to primary consolidation, Terzaghi first published his consolidation theory in 1925 (Terzaghi, 1925) which described pore water pressure dissipation. The end of primary consolidation for a soil sample at various effective stress levels is normally presented in the form of a void ratio versus log of applied effective stress ( $e - \log p$ ) plot, from which the re-compression and compression indices ( $C_r$  and  $C_c$ ) may be obtained to describe the compressibility of the soil.

About a decade later, laboratory tests and field observations reported by Buisman (1936) and Taylor (1942) clearly indicated the effect of time on the compressibility of clays. Buisman found that settlements increased linearly with log of time under constant effective stress for clay and peat loaded in the field and in the laboratory. The diminishing consolidation with time under constant effective stress is referred to as creep.

It is well established that soft soils will undergo significantly greater consolidation when loaded beyond the preconsolidation pressure,  $p_c$  (or  $\sigma_c'$ ). The preconsolidation pressure is defined as the effective pressure at which the break in slope occurs in the ( $e - \log p$ ) plot, and is also called the “yield stress” in some references. The latter term is used to avoid the common misconception that the presence of a preconsolidation pressure in soft clays is due only to past loading of the soil to a higher stress level (e.g. due to erosion). Chandler *et al.* (2004) concluded that the yield stress is a function of the structure of the clay, and suggested that it is likely to be greater than the maximum geological stress which it has sustained, whether the clay is normally or overconsolidated. In addition to geological stress history, the structure of the clay after deposition can be affected by environmental factors such as pore fluid chemistry (e.g. salinity), chemical and biological bonding and desiccation. The structure of clay deposits is also affected by creep and it has been shown (Taylor, 1942, Crawford, 1964 and Bjerrum, 1972) that the longer the soil is allowed to creep under constant stress after initial deposition, the higher its  $p_c$  will be.

### 2.2 THE CONCEPT OF PRELOADING

Although the term preloading may imply loading soft soils to a stress level equivalent to the in-service condition under self weight and design applied loading, it is also often used to describe the process of surcharging the soils above their anticipated in-service stress level so as to reduce post-construction creep as well as primary consolidation. The application of surcharge is inferred when the term preloading is used in this paper.

The ratio of the recompression index to the compression index,  $C_r/C_c$  is typically 0.1 to 0.2, and therefore, when a soil has previously been loaded to or beyond the final stress resulting from the proposed design load, the expected settlement due to primary consolidation when the design load is applied will be only 10% to 20% of that of a normally consolidated soil (i.e. reloading within the over-consolidated stress range). However, if the final stress is at or close to the preconsolidation pressure, significant creep settlement may still occur over time, and such creep settlement may not be acceptable over the design life of the development.

An important aspect to bear in mind when designing the preload is that the stress history and stress levels in a soil profile are not constant with depth. This is normally overcome by subdividing the soil profile into sub-layers and examining each of the sub-layers.

### 2.3 TIME-DEPENDENT CONSOLIDATION THEORY AND ITS APPLICATION IN PRELOADING

About a decade after Terzaghi presented his theory of one-dimensional consolidation, Taylor developed one of the first time-dependent consolidation models (Taylor, 1942). The early effort of Taylor attempted to model creep during primary consolidation. However, the effects of creep compression on preconsolidation pressure were generally not considered before Bjerrum's development of the time lines theory (Bjerrum, 1967; 1972). Bjerrum's conceptual model is often referred to as the concept of “artificial aging” as illustrated in Figure 1.

Figure 1 shows that if a soil sample in a laboratory oedometer test is loaded from its *in situ* effective stress value  $p_0$  at point “a” to point “c”, which represents the end of primary consolidation and “d”, which represents the point at which the surcharge is removed after a certain time period, then the stress level is reduced to that of point “e”, the creep rate will now be significantly lower.

In Figure 1, Bjerrum showed parallel time lines for creep (i.e. independent with stress level) and a reducing creep strain rate equivalent to one order of magnitude reduction with each log time cycle. The validity of these assumptions will be explored later in this paper.

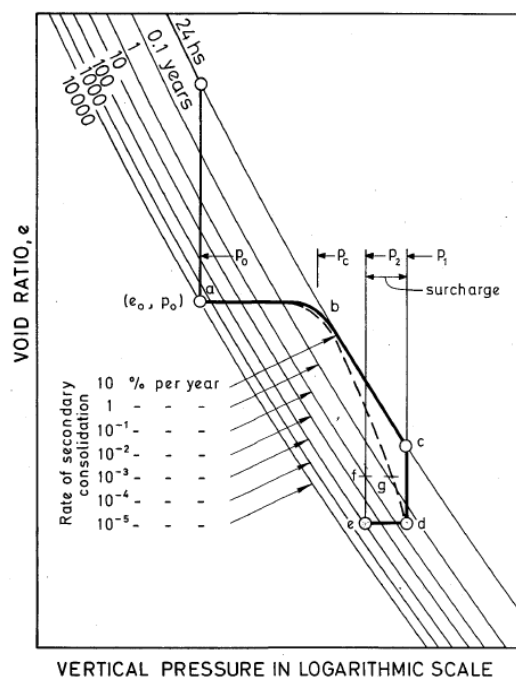


Figure 1: Principle of designing a preload to reduce the rate of creep (Bjerrum, 1967; 1972).

#### 2.4 DEPENDENCY OF CREEP ON STRESS LEVEL

Just as soil compressibility during primary consolidation is stress level-dependent and history-dependent, such dependency also applies for creep. The creep dependency on stress level is true for all materials, including steel and plastic, and also stiff compacted fill as reported in Waddell and Wong (2005). The creep strain rates for most engineering materials are also strongly dependent on operating temperature (e.g. plastic, glass and metal). For soils below the ground, the temperature is relatively constant and the dependency of creep on temperature may therefore be ignored.

Examples of the dependency of creep on stress level in soft soils have been presented by Nash *et al.* (1992) and Ewers and Allman (2000). The test results by these researchers are reproduced in Figure 2 below.

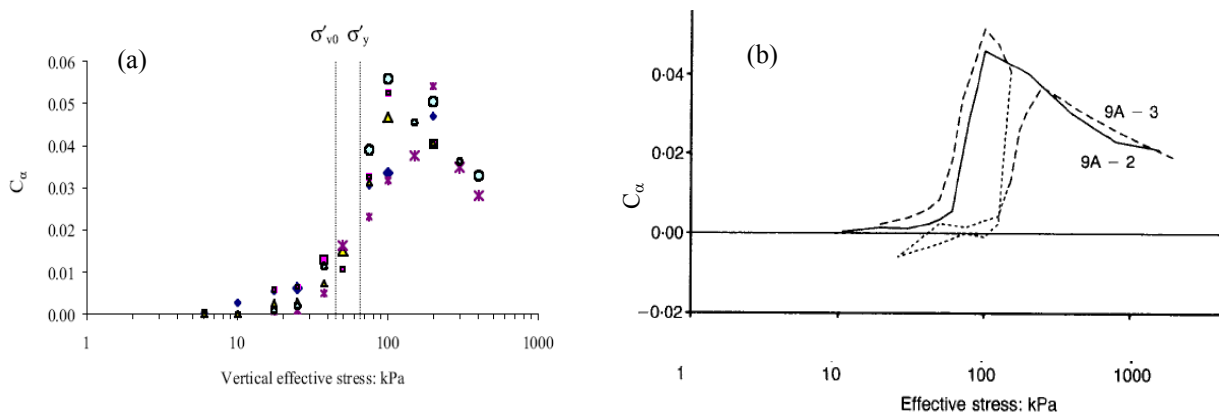


Figure 2: Dependency of Creep on Effective Stress (a) Ewers and Allman (2000) – soil samples from Teven Road site of Ballina Bypass Project, NSW, Australia (b) Nash *et al.* (1992) – soil sample from Bothkennar soft clay test bed, UK. (Note: Vertical Axis is  $C_{\alpha}$ , not  $C_{\alpha\epsilon}$ ).

2.5 DEPENDENCY OF CREEP ON STRESS HISTORY

Compelling evidence has been presented by Mesri and Godlewski (1977) of the relationship between  $C_{\alpha}$  and  $C_c$ . They found that at any  $(\sigma_v', t)$  during secondary compression, the ratio  $C_{\alpha}/C_c$  is a constant in both the recompression and compression ranges. In the above ratio  $C_{\alpha}/C_c$ , the value  $C_c$  is also used to describe the recompression index depending on the stress range. In other words, if  $C_c$  is replaced by  $C_r$  in the recompression range, and  $C_r$  is 0.1 to 0.2 times  $C_c$ ,  $C_{\alpha}$  will also reduce proportionally.

A clear illustration of the stress history dependency of creep has been presented by Magnan *et al.* (2001) as reproduced in Figure 3.

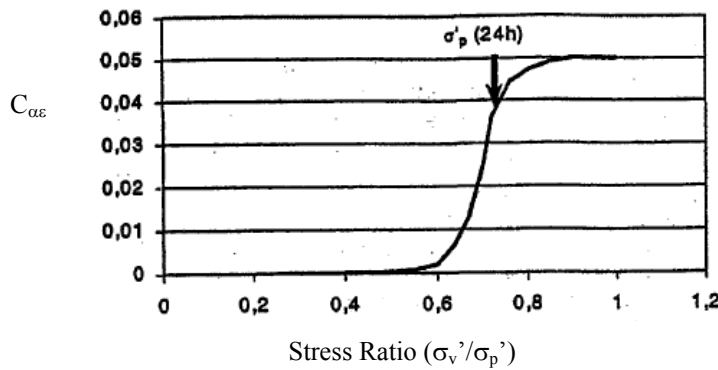


Figure 3: Stress History Dependency of Creep (Magnan, 2001).  
(Note:  $C_{\alpha\epsilon}$  here is creep strain, not Mesri’s Creep Index  $C_{\alpha}$ )

The stress ratio reported by Magnan *et al.* is the inverse of the Over-Consolidation Ratio (OCR), and if an empirical exponential relationship such as Equation 1 below is used to describe varying  $C_{\alpha\epsilon(oc)}/C_{\alpha\epsilon(nc)}$  with varying OCR, then one would obtain a gradual transition of creep strain rate near the preconsolidation pressure rather than an abrupt change from an overconsolidated value to a normally consolidated value.

$$\frac{C_{\alpha\epsilon(oc)}}{C_{\alpha\epsilon(nc)}} = \frac{(1 - m)}{e^{(OCR-1)n}} + m \tag{1}$$

In Equation 1 ‘m’ and ‘n’ are constants. Constant ‘m’ represents the minimum value of  $C_{\alpha\epsilon(oc)}/C_{\alpha\epsilon(nc)}$  or  $C_{\alpha(oc)}/C_{\alpha(nc)}$  when the OCR is large. From Mesri (1991) ‘m’ will be equivalent to the ratio  $C_r/C_c$ . The magnitude of ‘n’ controls the rate of reduction of  $C_{\alpha(oc)}/C_{\alpha(nc)}$  with OCR. Alonso *et al.* (2000) presented data for the reduction in the rate of creep with OCR for an inorganic clay from which the exponents ‘m’, and ‘n’ for Equation 1 to fit the data was found to be approximately 0.1 and 12. The creep reduction versus OCR relationship using ‘m’ = 0.1 and ‘n’ = 6 is shown in Figure 4 together with the back-figured data from Alonso *et al.* (2000). In the absence of data, the ‘n’ value of 6 is probably a reasonably conservative value for organic clays for preliminary assessment purposes. Of course, site specific data should be used if test data is available.

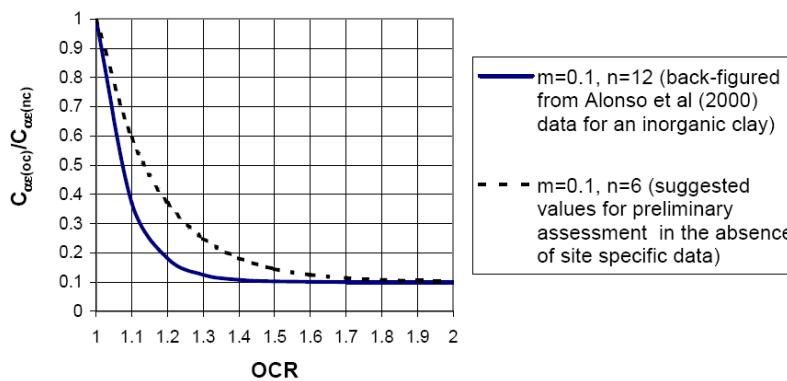


Figure 4: Creep versus OCR Relationships.

At this point, it is worth mentioning that the standard bilinear interpretation of recompression and compression in the typical ( $e - \log p$ ) plots is only a simplification for computational purposes. Real soils are likely to exhibit a curved transition below and above the preconsolidation pressure, (i.e. the slope describing the compression index is increasing gradually with increasing stress level around the preconsolidation pressure). In many soils the ( $e - \log p$ ) plot also flattens at high stress levels, and in such situations a secant  $C_c$  value may need to be used at the appropriate stress level. Therefore, Mesri and Godlewski's findings are indeed consistent with the creep - stress level/history dependency observations reported by others.

In other words, while  $C_{\alpha}$  may be considered as a constant (and proportional to  $C_c$  or  $C_r$ ) at any particular stress level and loading history, its magnitude does vary, depending on stress history, stress level and time. This finding contradicts with the parallel creep lines presented in Bjerrum's conceptual model shown in Figure 1. However, it does provide a clearer explanation for soft soils that have been deposited at relatively shallow depths (i.e. low stress levels) for tens of thousands of years but do not necessarily have high preconsolidation pressures as would otherwise be interpreted by using Bjerrum's figure. That is, at low stress levels, the creep strain rate is correspondingly low even when normally consolidated. Furthermore, creep itself causes an over-consolidation effect which further reduces the creep strain that is illustrated in Bjerrum's conceptual model of artificial aging. Although it is the author's view that the creep lines shown in Figure 1 are probably not parallel, Bjerrum's conceptual model is nevertheless very powerful for developing an understanding of the principle of artificial aging and preloading, and may be adopted provided the correct creep strain values are selected on the basis of stress level and stress history.

## 2.6 WHEN DOES CREEP COMMENCE?

For ease of computation, primary consolidation and creep are generally considered separately, and from the laboratory consolidation test result, commencement of creep is considered at the point of intersection of the straight line slopes drawn through the primary consolidation phase and creep phase of the test. This may be sufficiently accurate for a thin laboratory sample which may typically complete its primary consolidation phase in about 24 hours and during which the creep component is small. For a thick soft soil deposit in the field, this assumption may lead to errors in interpretation of monitoring results.

If we refer back to Sections 2.3 to 2.5 and consider the relationships between creep strain rate and "time lines", effective stress, and overconsolidation ratio, it would be intuitive that creep would commence as soon as additional effective stress is imposed, albeit it may be slow to start with due to the fact that an increase in total vertical stress produces an equal increase in porewater pressure initially. But as the pore pressure dissipates with time, the effective stress will increase and the effective stress state of the soil, "artificial age" and OCR will begin to alter according to Bjerrum's concept shown in Figure 1. It is also important to note that the dissipation of pore pressure (and therefore the rate of effective stress increase in creep) is non-uniform in a soil profile, with a faster rate of pore pressure dissipation near drainage boundaries. Therefore, the onset of creep is likely to commence sooner and at a faster initial rate near drainage boundaries than those further away. The concept of simultaneous creep and primary consolidation has been explored by many researchers, and Mesri *et al.* (1994) described the change in void ratio with time in the following equation:

$$\frac{de}{dt} = a_{vs} \frac{d\sigma_v'}{dt} + a_{vt} \quad (2)$$

In Equation 2, the rate of compression  $de/dt$  is determined by the two compressibility parameters  $a_{vs}$  and  $a_{vt}$  which describe the rate of effective stress increase in the primary consolidation phase and the creep phase respectively in a simultaneous manner. It is only after all the excess pore pressure has dissipated and  $d\sigma_v'/dt$  by definition is zero that the rate of compression is equal to  $a_{vt}$  alone during the creep phase.

Furthermore, because the rate of effective stress increase (i.e.  $d\sigma_v'/dt$ ) is equal to the rate of pore pressure decrease ( $-du'/dt$ ), Mesri *et al.* (1994) used this equality and rearranged Equation 2 to form Equation 3 as follows:

$$-\frac{du'}{dt} = \frac{\frac{de}{dt} - a_{vt}}{a_{vs}} \quad (3)$$

Equation [3] shows that an increase in either  $a_{vs}$  or  $a_{vt}$  slows pore pressure dissipation. These simple equations clearly show that both primary consolidation and creep occur simultaneously and that they affect each other. They also provide conceptual explanations for the following common observations:

For pressure increments within the recompression range, the porewater pressures dissipate rapidly because  $a_{vs}$  is small.

As primary consolidation takes place and the effective stress increases with time, the effective stress may increase from within the overconsolidated range to the normally consolidated range, and the value of  $a_{vs}$  abruptly increases, and there is a dramatic reduction in the rate of porewater dissipation and primary consolidation.

As the effective stress increases, so does the creep rate,  $a_{vt}$ , which has the effect of delaying porewater dissipation according to Equation 3. This point is important as there is a common misconception that residual porewater pressures are being observed during the creep phase, when in fact primary consolidation is incomplete.

### 3 TIME SETTLEMENT ANALYSIS METHODS

As discussed in the preceding sections, preload involves loading, unloading and reloading and by doing so the soft soil profile undergoes a complex change in structure and porewater pressure dissipation and the inter-related primary consolidation and creep parameters will change with time.

Ideally, design of preloads should take into account the simultaneous nature of primary consolidation and creep. There has been significant progress in recent years towards development of time-dependent modelling of simultaneous primary consolidation and creep. The various approaches include:

- Macro-mechanical approach (also known as the phenomenological approach) based on fitting of mathematical expressions to experimental evidence (Fedá, 1992).
- Micro-mechanical approach, which attempts to model soil behaviour at the particulate or molecular level (Mitchell *et al.*, 1968; Kuhn and Mitchell, 1992).
- Meso-mechanical approach, which combines the macro- and micro-mechanical approaches (Fox *et al.*, 1994).
- Rheologic mechanical model such as the Rajot Model (Rajot, 1992) which uses a series of elastic springs and sliders to represent the primary consolidation component and a series of non-linear springs and non-linear dashpots to model the creep component.
- Elasto-visco-plastic approach based on a set of constitutive models including softening and hardening functions that describe the mechanical response of the soil based on experimentally observed soil behaviour. Early work in this approach included Perzyna's visco-plastic over-stress theory (1963) and Olaszak-Perzyna's (1966) theory of non-stationary flow surfaces. The latter included the concept of a yield surface that changes in time due to creep behaviour to model the time-dependent compression behaviour of soft soils. More recent developments include Modified Cam-clay model (Yin and Graham, 1999) and the "a,b,c Isotache" model (den Haan, 1996; den Haan and Sellmeijer, 2000). Most of these models use a hardening rule that is principally based on Bjerrum's time line concepts although, instead of using an instant time line model, either a total strain rate model or a creep rate model is used.

For readers who are interested in further details of various computer models for time-dependent analysis of simultaneous consolidation and creep of clay, a good summary of the types of analysis available and their capabilities has been provided by Perrone (1999), together with development of his computer model CONSOL97.

The commercially available PLAXIS finite element analysis program (PLAXIS Version 8, 2002) has a Soft-Soil-Creep model and utilises a form of strain rate controlled time-dependent consolidation and creep analysis that also incorporates concepts of Modified Cam-clay and viscoplasticity.

One advantage of using the Soft-Soil-Creep model of PLAXIS is that the input used may be obtained from the usual parameters that are measured by conventional one-dimensional laboratory testing. However, the constitutive model is not readily understood by practising engineers and it will be shown in Part 2 of this paper that the analysis results are dependent on the geological time assumed since deposition of the soft soil, as well as the initial OCR profile of the soil.

### 4 PRELOAD DESIGN FOR EMBANKMENTS CONSTRUCTED ON SOFT SOILS

Unfortunately, many of the time-dependent analysis models described in Section 3 are not easily understood by practising geotechnical engineers and some are still in the research stage and not available for commercial use. In any case, indiscriminate use of research or commercial software, without understanding the assumptions and limitations of these models, has the potential to result in serious errors.

Therefore, despite recent advances in time-dependent analysis of simultaneous consolidation and creep for soft soils, most settlement analysis and preload designs in practice are still being carried out using traditional analysis methods that treat primary consolidation and creep in separate, consecutive phases.

In the author's opinion, it is important to be able to use fundamental principles and conceptual models that are readily understood, for use as either a preliminary design tool, or as a check on more complex analysis methods that are more difficult to understand. Bjerrum's time line approach forms a readily understood conceptual model that may be used to gain insight into the stress path effect on post-surge compression behaviour of soft soils. This approach will be extended to include stress level and OCR dependency in Part 2 of this paper.

Mesri (1991) presented his design approach for surcharging to reduce post-construction creep that incorporates the use of a "secant" creep index,  $C_{\alpha}$ " based on experimental results. Their method is summarised in Figures 5 and 6, and Equations 4 and 5.

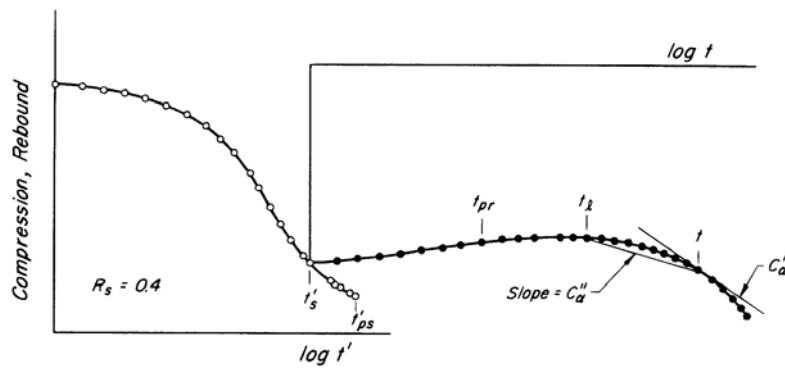


Figure 5: Settlement Behaviour – Surcharge Removal (Mesri, 1991).

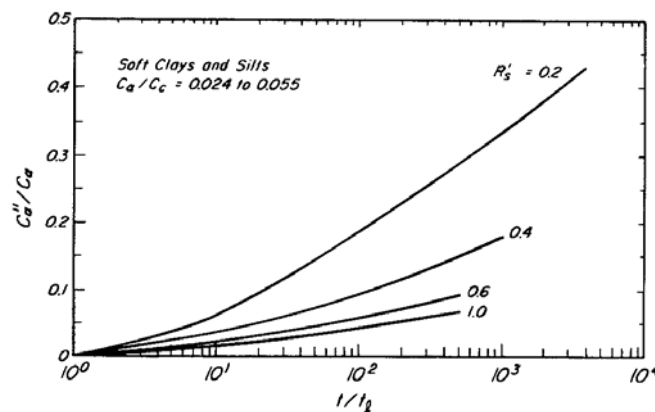


Figure 6: Post Surcharge  $C_{\alpha}$  Relative to OCR and Time Effects (Mesri, 1991)

In Figures 5 and 6, Mesri (1991) defined the stress history and time-dependent parameters,  $R_s'$  and  $t_r/t_{pr}$  that are used to assess the secant creep index,  $C_{\alpha}$ " as follows:

$$R_s' = \text{OCR} - 1 \tag{4}$$

$$t_r/t_{pr} = 100(R_s')^{1.7} \tag{5}$$

In Equation 5,  $t_{pr}$  and  $t_r$  are the time at completion of primary rebound and recommencement of creep, following removal of the preload. The time for completion of primary rebound (100% dissipation of negative porewater pressures) may be computed using traditional consolidation theory. The primary rebound time is generally fairly rapid due to unloading in the recompression range as discussed in Section 2.6.

By trial and error, using different surcharges, the final OCR and  $R_s'$  values may be calculated and once  $C_{\alpha}$ " is obtained from Figure 5 and  $t_r$  obtained from Equation 5, the post-construction creep settlement may be obtained from the equation:

$$s = \frac{C_{\alpha}''}{(1 + e_o)} h_o \log \frac{t}{t_i} \quad (6)$$

Equation 6 should be integrated for all sub-layers that may have different  $h_o$ ,  $C_{\alpha}''$ , and  $t_i$  values.

The possibility of relatively large on-going creep following preloading is fairly obvious from Figure 5, although the underlying reasons are still not readily conceptualised via Mesri's experimental data. In Part 2 of this paper, the use of Bjerrum's time line concept will be discussed and applied on an example problem to illustrate the likely reason for observed post-construction being greater than that predicted following preloading.

## 5 CONCLUSION

In designing a preload to reduce post-construction settlement, it is important to have a good basic understanding of the principles involved and soil consolidation behaviour. The following points have been highlighted in this paper:

- The geological history as well as loading history has a strong influence on compressibility behaviour of soils.
- Creep behaviour is one of the most important, yet least understood, aspects in preload design.
- Creep is dependent on stress level, geological and stress history.
- It is likely that creep occurs simultaneously with consolidation, particularly with thicker soft soil deposits.
- Various constitutive relationships using time-dependent, simultaneous consolidation and creep models, have been developed, ranging from fitting mathematical expressions to experimental data, to complex numerical models using elasto-visco-plastic models that can be used for assessing preloading and post-preload settlement behaviour.
- Despite recent advances in time-dependent settlement analysis and the availability of powerful commercial software, the underlying assumptions and basis of such software may not be readily understood by practising engineers. The author believes that in order to reduce the risk of serious errors in preload design, it may be better to use simpler models that incorporate more readily understood soil behaviour. Bjerrum's time line model provides such an approach that is still widely used for preload design today.
- Mesri's experimental data (Mesri, 1991) shows the potential for increasing rate of creep sometimes following preloading, under certain time and surcharge ratio conditions.

In Part 2 of this paper, an analytical approach based on Bjerrum's time line model (Bjerrum, 1967; 1972) is developed for preload design, and a preload design example is discussed to illustrate the importance of geological and stress history on post preload settlement behaviour.

## 6 REFERENCES

- Alonso, E.E., Gens, A. and Lloret, A. (2000), "Precompression design for secondary settlement reduction" *Geotechnique*, 50, No. 6, pp. 645-656.
- Australian Standard AS 1289.6.6.1 (1998), "Method of testing soils for engineering purposes. Method 6.6.1: Soil strength and consolidation tests-determination of the one-dimensional consolidation properties of a soil-standard method", Standards Australia
- Bjerrum, L. (1967). "Engineering geology of Norwegian normally consolidated marine clays as related to settlements of buildings", 7<sup>th</sup> Rankine Lecture, *Geotechnique*, 17(2), pp. 81-118.
- Bjerrum, L. (1972). "Embankments on Soft Ground", Proceedings of Speciality Conference, Performance of Earth and Earth-Supported Structures, American Society of Civil Engineers, New York, NY., Vol II, pp 1-54.
- Buisman, L. (1936) "Results of long duration settlement tests", Proceedings, First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Massachusetts, Vol. 1, pp. 103-107.
- Chandler, R.J., de Freitas, M.H. and Marinos, P. (2004) "Geotechnical Characterisation of Soils and Rocks: a Geological Perspective" The Skempton Conference, Thomas Telford, London, pp 67-102.
- Crawford, C.B. (1964) "Interpretation of the consolidation test", *Journal of the Soil Mechanics and Foundations Division, Proceedings of the ASCE*, 90(5), pp. 87-102.
- den Haan, E.J. (1996) "A compression model for non-brittle soft clays and peat" *Geotechnique* 46(1), pp. 1-16.
- den Haan, E.J., and Sellmeijer, H.J.B. (2000) "Calculation of Soft Ground Settlement with an Isotache Model" *Geotechnical Special Publication Number 112, Soft Ground Technology*, Edited by James L. Hanson and Ruud J. Termaat, Proceedings of the Soft Ground Technology Conference, the Netherlands, pp. 94-104.
- Ewers, B. and Allman, M.M. (2000) "Secondary Compression of Soft Clay from Ballina" *GeoEng 2000*

- Feda, J. (1992) "Creep of Soils and Related Phenomena" Elsevier Science Publishing Co., New York.
- Fox, J.P., Edil, T.B., Malkus, D.S. (1994) "Discrete element model for compression of peat" *Computer Methods and Advances in Geomechanics*, Siriwardane and Zaman (editors), Balkema, Rotterdam.
- Kuhn, M.R. and Mitchell, J.M. (1993) "New Perspectives on Soil Creep", *Jnl. of Geotechnical Engineering*, 119(3), pp. 507-524.
- Magnan, J. P., Bertaina, G., Khemissa, M., and Reiffsteck, Ph. (2001). À propos des indices de fluage déterminés à l'œdomètre, *15<sup>th</sup> International Conference on Soil Mechanics and Geotechnical Engineering*, Istanbul, vol. 1, Balkema, pp. 203-206.
- Mesri, G. (1991) "Prediction and performance of earth structures on soft clay", *Proceedings of The International Conference on Geotechnical Engineering on Coastal Development*, Yokohama, 2, G2.1-G2.16.
- Mesri, G., and Godlewski, P.M. (1977) "Time and Stress-Compressibility Interrelationship", *Jnl of the Geotechnical Engineering Division, ASCE* (103), No. GT5, pp. 417-430.
- Mesri, G., Lo., and Feng, T. (1994). *ASCE Specialty Conference on Geotechnical Engineering*, Special Publication No. 40, vol. 1, pp. 8-76.
- Mitchell, J.K., Campanella, R.G., and Singh, A. (1968) "Soil Creep as a rate process", *Jnl. Geotech. Engineering Division, ASCE*, 94(1), pp. 232-253
- Nash, D.F.T., Sills, G.C. & Davison, L.R. (1992) "One-dimensional consolidation testing of soft clay from Bothkennar" *Geotechnique* 42, No. 2, pp 241-256.
- Olaszak, W. and Perzyna, P. (1966) "The constitutive equations of the flow theory for a non-stationary yield condition", *Applied Mechanics, Proc. Eleventh International Congress of Applied Mechanics*, pp545-533, Springer – Verlag.
- Perrone, V. (1999) "One Dimensional Computer Analysis of Simultaneous Consolidation and Creep of Clay", PhD Thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.
- Perzyna, P. (1963) "Fundamental Problems in Viscoplasticity", *Advances in Applied Mechanics*, Vol. 9, pp243-377, Academic Press, New York
- PLAXIS (Version 8, 2002) *Finite Element Code for Soil and Rock Analysis, User Manual*
- Rajot, J.P. (1992) "A theory for the time dependent yielding and creep of clay", Ph.D. Thesis, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.
- Taylor, D.W. (1942) "Research on Consolidation of Clays", Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, Serial 82.
- Terzaghi, K. (1925) *Erdbaumechanik*, Franz Deuticke, Vienna.
- Waddell, P.J. and Wong, P.K. (2005) "Settlement of Deep Compacted Fill" *Australian Geomechanics Jnl.* Dec 2005 pp57-72.
- Yin, J.H. and Graham, J.(1999) "Elastic viscoplastic modelling of the time dependent stress-strain behaviour of soils" *Canadian Geotechnical Jnl.*, 36, pp736-745.

