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Technical session 1f: Prediction and performance

Séances techniques 1f: Prédiction et performance

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1 INTRODUCTION

There are 33 papers allocated to this technical session representing 18 countries. The majority of these papers compare theoretical predictions with measurements made either in the laboratory or in the field. However, a few of the papers consider only theoretical predictions and some concetrate on the accuarcy of emprirical relationships. They cover a wide range of geotecnical problems.

All of the papers are reviewed below and have been grouped in terms of subject matter. Due to space limitations it has not been possible to provide an extensive review of each paper. Consequently, only the main conclusions are stated and to some extent these have been influenced by the reporter's experience. He apologises to the athors if he has, in any way, misrepresented the content of their paper.

2 SETTLEMENTS ON SOFT CLAY

Thirteen of the papers in this technical session relate to the calculation of settlements on soft clay. They have been grouped into those that deal with laboratory research, analytical solutions, numerical analysis and empirical approaches.

2.1 *Laboratory based research*

The objective of the paper by Mesri et al (2005) is to determine the magnitude of the excess pore water pressure associated with the beginning of secondary compression. A paradox arises as it is often assumed that primary consolidation ends and secondary compression begins when excess pore water pressures become zero. However, if secondary compression occurs in a saturated soil, water must be expelled and consequently there must be a hydraulic head to promote water flow. Such a hydraulic head must be associated with an excess pore water pressure. The paper first presents some simple one dimensional consolidation theory to assess the magnitude of the pore water pressure at the beginning of secondary compression. Two scenarios are considered. In the first a compressible layer without vertical drains is considered and vertical flow of water is assumed. In the second, a compressible layer with vertical drains and radial flow of water is considered. The resulting equations for the maximum excess pore water pressure, u'_m at the end of primary consolidation are similar and take the form;

$$u'_m = \frac{\sigma'_v C_\alpha}{\beta C_c} \tag{1}$$

where σ'_v is the final consolidation pressure, C_α is the secondary compression index, C_c is the primary compression index and β equals 2.6 or 2.3 for a compressible layer with and without vertical drains respectively. This equation is based on the assumption that the duration of primary consolidation corresponds to 95% average degree of consolidation, as defined by Terzaghi's theory of consolidation for a linear distribution of initial excess pore water pressure with depth in a single homogeneous layer (Terzaghi et al., 1996). Unfortunately, no justification is provided for this key assumption. Based on this equation the authors show that for realistic values of C_α/C_c = 0.03 to 0.07 then u'_m/σ'_v= 1 to 3%.

Experimental data from laboratory oedometer tests are then presented to validate the above equation. Results from 86 pressure increments on 8 soft clays, 56 increments on 7 clay shales, and 13 increments on 2 fibrous peats are considered. The values of excess pore water pressure measured at t_p = the Casagrande t_{100} are compared with the excess pore water pressure computed using Eq. 1 (i.e. at t_p = the Terzaghi t_{95}). This comparison is reproduced here as Fig. 1.

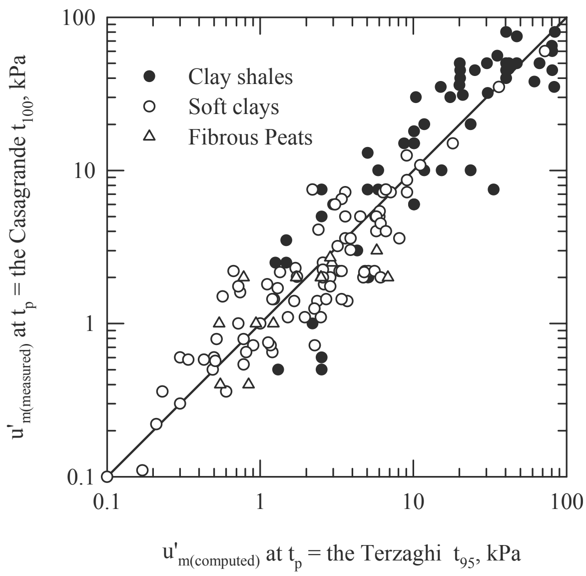

Figure 1. Computed and measured excess pore water pressure at EOP consolidation (after Mesri et al., 2005).

While there is scatter in the data on this figure, which according to the authors may be due to inaccuracies in measuring the excess pore water pressure, there is generally reasonable correlation between the measured and computed values of u'_m.

The analyses and experimental results suggest that for soft clay and silt deposits and for fibrous peats, which are rarely subjected to σ'_v values greater than 500kPa, the maximum excess pore water pressure at the end of primary consolidation is expected to be of the order of 1kPa and is not expected to exceed 10kPa. However, for clays and shales subjected to σ'_v values of 8 to 10MPa, u'_m may be as high as 100kPa.

In conclusion, the paper shows that for soft soils the maximum excess pore water pressure at the end of primary consolidation is likely to be extremely small and therefore consistent with the concept that secondary compression begins once the excess pore water pressures approximately reach zero. Or alternatively, only a very small hydraulic gradient is required to expel water during secondary compression

The paper by Tanaka et al. (2005) uses the interconnected oedometer (ICO) apparatus to investigate the effect of different specimen thicknesses on the consolidation behaviour of clay in a normally consolidated state and at a state of transition from the in situ stress (p'_{vo}) to the yield consolidation pressure (p'_c). The ICO apparatus essentially consists of a set of conventional oedometer samples (6cm in diameter and 1cm to 2cm thick) connected in series to form a larger sample. It has the advantage over the use of a single larger sample in that the friction effects between the specimen and the inside wall of the oedometer ring are minimised and that the strain rate in each of the constitute sub-samples can be monitored separately.

The clay tested was from Osaka and came from two different depths. The Ma10 clay came from an elevation of 125m whereas the Ma3 clay came from an elevation of 276m. The Ma3 clay was used to investigate consolidation behaviour between p'_{vo} - p'_c (i.e overconsolidated (OC)) and the Ma10 clay the behaviour under a stress range $2p'_c$ - $3p'_c$ (normally consolidated (NC)).

The use of samples from two different clay layers should be noted as it clearly could have an effect on the conclusions reached. It would have been more logical to use samples from the same clay layer for all of the testing. It should also be noted that the apparent overconsolidation noted for these clays is unlikely to be due to previous loading but due to ageing effects. This is particularly relevant to the OC tests performed on the Ma3 clay.

Figures 2 and 3 (taken from the paper) show the effect of sample height (H) on the consolidation behaviour in terms of normalized pore water pressure ($u/\Delta p$) and vertical strain (ε) plotted against normalized time (t/H^2). In the legend the number after the H refers to sample height in cm.

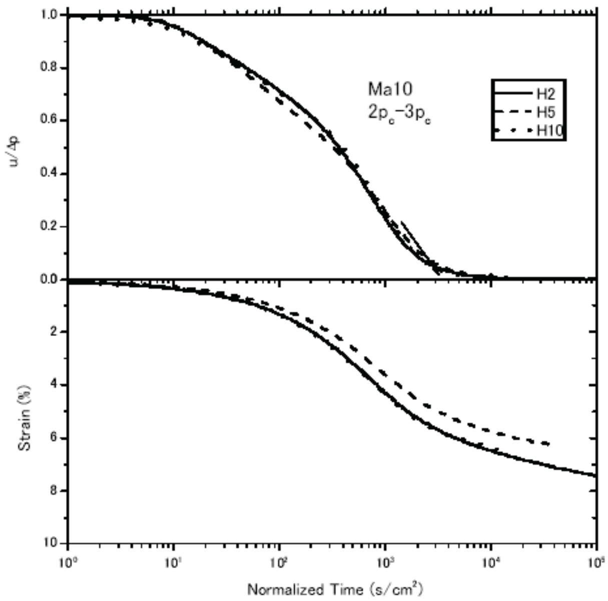

Figure 2. Relationship between elapsed time and $u/\Delta p$ or ε for Ma10 (at NC state) (after Tanaka et al., 2005)

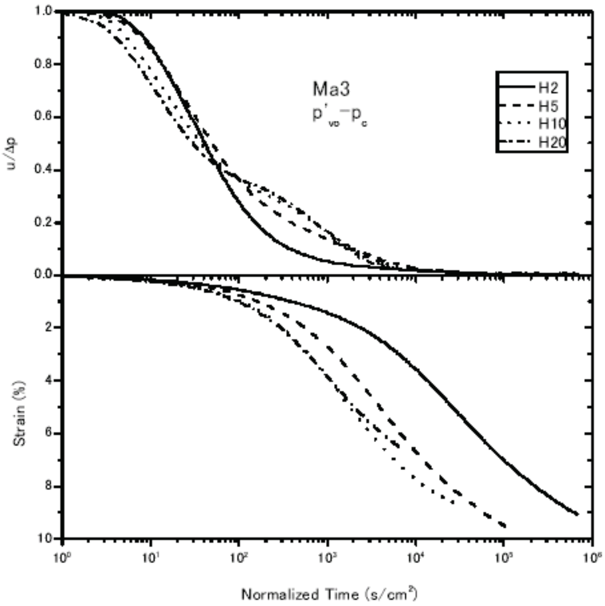

Figure 3. Relationship between elapsed time and $u/\Delta p$ or ε for Ma3 (at OC state) (after Tanaka et al., 2005)

In the case of NC clay, see Fig. 2, both the $u/\Delta p$ and ε can be normalized by t/H^2. In addition, the generated strain coincides well with the dissipation of excess pore water pressure and therefore occurs simultaneously with, or as a direct consequence of, a change in the effective stress.

In contrast, for the OC clay, see Fig. 3, neither $u/\Delta p$ or ε can be normalized by t/H^2. In particular, as the sample thickness decreases, larger values of t/H^2 are required to attain the same strain level. This in turn implies that as the thickness increases consolidation proceeds faster than that expected from the H^2 rule. In addition, it is noted that most of the strain is generated after the dissipation of the excess pore water pressure. This tendency being more prominent as the specimen thickness decreases.

The results of these tests are interesting in that they imply that creep behaviour does not begin until primary consolidation has completed for the NC clay, whereas for the OC clay creep occurs concurrently with primary consolidation.

By considering the strain rates in the constitute sub-samples making up the larger specimens of both NC and OC clay, the paper shows the applicability of the Isotache theory in which changes in strain are governed both by changes in mean effective stress and strain rate.

2.2 *Analytical solutions*

Settlement of a test embankment at the site of the New Bangkok International Airport is discussed in the paper by Indraratna et al (2005). The soft clay below the embankment is stabilised with vacuum-assisted prefabricated vertical drains. After discussing the site and the trial embankment an analytical solution, based on the Cylindrical Cavity Expansion theory to predict the extent of the smear zone along the length of the drains and predict the performance of the embankment, is presented. The solution incorporates the modified Cam-clay model to simulate soil behaviour and accounts for permeability varying as a function of voids ratio.

The analytical solution is then used to predict time-settlement curves at various depths below the original ground surface for the trial embankment. Due to potential air leaks in the vacuum system used on site, the analysis employed the pore water pressures measured in the drain placed immediately above the original ground surface as a boundary condition. The soil parameters were obtained from an extensive site investigation.

The excellent agreement between the analytical predictions and the measured behaviour is shown in Fig. 4.

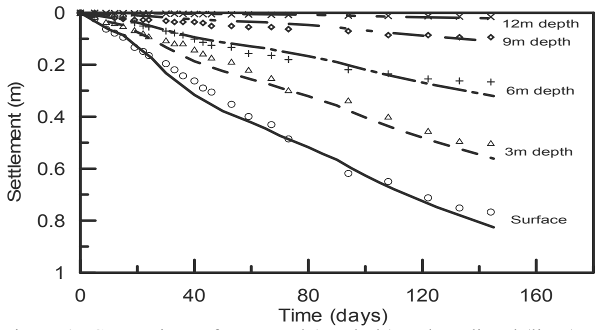

Figure 4. Comparison of measured (symbols) and predicted (lines) consolidation settlements at various depths (after Indraratna et al., 2005)

2.3 *Numerical analysis*

Long term settlements of three reclaimed islands in Osaka Bay are considered in the paper by Mimura et al (2005). Here the main emphasis is on the pre-yield modeling of the behaviour of the clay deposits in the foundation. As noted above, the apparent overconsolidation of these clays (OCR of 1.1 to 1.4) is due to ageing. Consequently, the authors argue that behaviour pre-yield should be modeled as elasto-viscoplastic and not elastic as is done conventionally. Only if the soil is unloaded should the behaviour be assumed elastic, see Fig. 5.

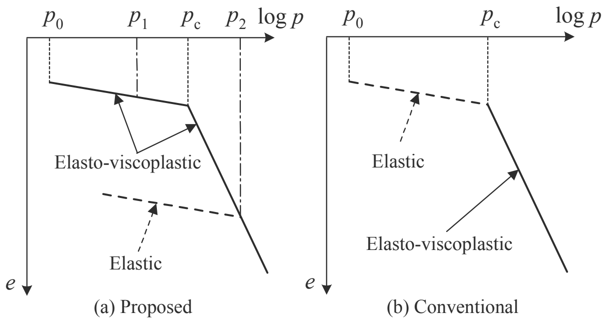

Figure 5. Compression model (after Mimura et al., 2005)

The paper presents the results from one dimensional finite element analyses for the three islands. The clay layers are modeled with a modified plane strain version of the elasto-plastic model proposed by Sekiguchi (1977). No details of the model or of the determination of soil parameters are provided. It is merely stated that the parameters "were determined rationally based on the prescribed procedure (Mimura et al., 1990)". Presumably the analysis involved coupled consolidation but again this is not clear.

Two analyses were performed for each island. One based on a conventional approach in which elastic behaviour (i.e. no creep) is assumed pre-yield and one where elasto-viscoplastic behaviour (i.e. with creep) is assumed, albeit with an appropriate compression index "$\kappa=\lambda_{QOC}$". It is not clear if this compression index is the same in both analyses or not. Not with standing this, it is implied that apart from the modelling of the pre-yield behaviour the two analyses are similar.

Comparisons are then made with field measurements and it is shown that the proposed model results in the better predictions. While this is generally true it should be noted that in two of the three cases the field data does not appear to be continuous from the beginning of construction (i.e. the field observations began after construction commenced). In the figures it has been assumed that the field settlement has an initial value (just prior to the measurements starting) in agreement with the results of the analysis based on the authors proposed model. This, in the reporter's opinion, tends to bias the comparison and is unhelpful.

It is also worth remembering that the analyses were made after the observations were taken and consequently, without a detailed explanation concerning the determination of the input parameters and how sensitive the analysis is to the values chosen, it is difficult to judge the validity of the conclusions.

The use of finite element analyses to predict the behaviour of 5 pre-loaded highway embankments placed on uniform soft Ariake clay is the subject of the paper by Ohta et al. (2005). Coupled finite element analyses employing the elasto-viscoplastic constitutive model proposed by Sekiguchi & Ohta (1977) were performed. The initial soil parameters were selected based on the results from oedometer tests and the plasticity index, I_p using a procedure recommended by Iizuka & Ohta (1987). This appears to be based on a series of empirical correlations.

Four sets of analyses are reported. In the first series all 5 embankments are analysed using the initial estimate of soil properties and the as constructed height of the embankment above the original ground surface to simulate the embankment loading. The same analyses were then repeated, but with the actual fill thickness used to simulate embankment loading. These analyses therefore accounted for the settlement of the original ground surface during construction. It is not stated how this was achieved in the analysis or whether large displacement analyses were performed. In the third series of analyses the simulation was further improved by accounting for the recovery of the ground water level as a result of the dewatering pumps being switched off. In the fourth series of analyses the critical state strength, M, was increased based on the results from triaxial tests on Ariake clay sampled from other sites. It is implied that such results are more appropriate than the value of M determined from the empirical correlation based on I_p. It is not clear if the other parameters empirically related to the value of M were also changed.

The predictions of the four series of analyses are compared to the field data which comprised settlements of the original ground surface under the centre line of the embankment, pore water pressures in the foundation clay under the centre line of the embankment and lateral movements with depth on a vertical section near the toe of the embankment.

For each embankment the analyses improved (i.e. the predictions become closer to the field data) as they became more realistic (i.e. from series 1 to 4). However, for all embankments the lateral soil movements measured on a vertical section near to the toe of the embankment were not well predicted.

2.4 *Empirical approaches*

Several papers essentially present empirical equations for predicting settlements with time. Some of these methods rely on initial measurements made over a relatively short time period to predict long-term settlements.

The paper by Lansivaara (2005) is one such paper and promotes the use of the settlement potential approach of Janbu (1991). The settlement potential, S, is defined as

$$S = \frac{d\delta}{dt}t \qquad (2)$$

where δ is the settlement and t is time. Many case records have shown that, after some time, the settlement potential obtains a constant value that it keeps for a long period of time. In the paper it is shown that this observation is consistent with the assumption of a constant secondary compression index, C_α. The constant value of S usually occurs after a consolidation time of only a few years and long before the end of primary consolidation. This implies that the time-settlement process is actually governed by creep deformations under the later stages of the consolidation process. Two alternatives as to how S varies before it becomes constant are discussed and shown to be equally valid. One is based on the classical result from consolidation theory that settlement develops linearly with the square root of time and the other is based on a hyperbolic relation between settlement and time.

The proposed approach is then applied to three case studies and compared with the approaches of Asaoka (1978) and Korhonen (1977). Results for the Murro embankment are reproduced here in Fig. 6. All three methods used the first year of settlement observations to predict the following 9.4 years of settlement.

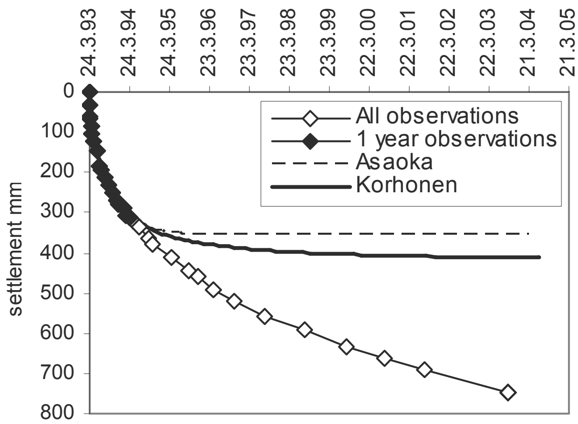

Figure 6. Settlement predictions at Murro test embankment (after Lansivaara, 2005).

The proposed approach aggrees well with the measurements. It is concluded that the proposed method is superior as it provides reliable predictions based on relatively short observation times.

The paper by Ravaska (2005) is also concerned with the problem of predicting future settlements based on available observations. However, in contrast to the above paper, it proposes that a simple power law of the form

$$\delta = at^{b} \tag{3}$$

is sufficient, where a and b are curve fitting parameters. As this equation is based on the well known square root of time fitting method proposed by Taylor (1948) for evaluating the coefficient of consolidation from oedometer tests, it is not surprising that during primary consolidation the exponent b takes a value near to 0.5 (i.e. typically 0.4-0.6).

The advantage of the method over the alternatives, including the three discussed above, is its ease of use and that a minimum of only three observations are needed to derive the parameters. By using field records of settlement during primary consolidation, it is shown that the method is as accurate as the settlement potential method discussed above and considerably better than the methods proposed by Asaoka (1978) and Korhonen (1977).

The potential shortcoming of the method is that it only provides accurate predictions of future settlements during primary consolidation if the parameters a and b are determined from early observations (i.e. those made early in the primary consolidation stage). To use the equation to predict creep settlements that arise during secondary compression, the observations used to determine the appropriate parameters a and b must be available from this same stage of the settlement process. That is the equation is unable to predict the complete time-settlement curve based on one set of parameters.

The paper by Juarez-Badillo (2005a) also proposes an equation for the time-settlement curve. This equation is derived from the Principle of Natural Proportionality. It takes the form:

$$\delta = \frac{\delta_\infty}{1+\left(\frac{t}{t_{50}}\right)^{-\chi}} \tag{4}$$

where δ_∞ is the ultimate settlement at $t=\infty$, t_{50} is the time when $\delta=0.5\delta_\infty$ and χ is the coefficient of proportionality.

To use the equation values of the parameters δ_∞, t_{50} and χ must be obtained. These may be obtained from three field measurements. The equation is used to predict the settlement of the Kansai International Airport. It is compared with the observed field measurements taken over a period of 10 years and shown to be in good agreement. However, its ability to predict longer-term movements which are dominated by secondary compression (creep) is unproven.

In the paper by Akai & Tanaka (2005) the results from reliability analysis performed on the settlement performance of the first stage construction of the Kansai International Airport in Osaka Bay is presented. The analysis is based on the settlement measurements taken over a nine-year period starting from the end of 1993, and also based on soil investigation results for the upper most clay layer (Ma12). To account for creep settlements (i.e. secondary compression), a simple hyperbola is used to represent the time-settlement curve. This is the same approach as used by Korhonen(1977) as mentioned above. The coefficients for this equation are obtained empirically by matching the curve to some of the field settlement observations. In essence, the main objective of the paper is to present a quantitative analysis based on a probabilistic approach accounting for the ambiguities of the measured settlement data and soil testing data.

It is concluded that the total settlement in 2043 (49 years from the opening of the airport) will be 14.3m, with ±0.5m error for a reliability index of 1.0 and ±1.0m error for a reliability index of 2.0. The paper also evaluates the performance of the passenger terminal building at the airport. It is shown that, even though adjustments have been made to the column levels by jacking and inserting shims, at regular intervals (i.e. 2 to 3 times a year), the differential settlement of the building is such that some of the distortional design criteria have been exceeded.

Settlement calculations based on the results of laboratory oedometer tests and one-dimensional Terzaghi consolidation theory is the subject of the paper by Aalto et al. (2005). Apparently in Nordic countries the calculation of settlement on soft clays is normally carried out using the Ohde-Janbu stress-strain model, the parameters of which, modulus number m and stress exponent β, are obtained by fitting the model function to a set of oedometer data. Settlement is then calculated using these parameters instead of the compression index C_c which is more commonly used throughout the world.

By using the results from oedometer tests the paper shows that the parameters m and β are stress level dependent and for Finish clays derives some empirical equations which account for this stress dependency. These relations for m and β, along with a stress dependent permeability (published elsewhere) are then used to back analyse the settlements of the Murro test embankment in Finland. Comparison of the predictions with the field data shows excellent agreement. It should be noted that, as creep has not been considered, the analysis is only appropriate to primary consolidation.

The monitored settlements of biodigestor tanks forming part of a waste-water treatment plant is the subject of the paper by Dapena et al (2005). These tanks are founded on 14 reinforced concrete slabs that rest on a deposit of soft clay alluvium that has a thickness of between 15m and 20m. Previously the clay had been loaded by a layer of blast furnace slag which was up to15m deep. At the location of the tanks most of this slag was removed and consequently the loading from the tanks caused stresses in the clay that were well below the preconsolidation pressure.

No soil parameters are provided in the paper. However, measured settlements are reported. These indicate that the observations produce an approximately linear relationship when plotted as settlement against the square root of time. They also indicate that the compressibility of the clay is highly variable, with settlements ranging from 28mm to 136mm. It is noted that these values are high and that settlements are still ongoing, even though the loading does not produce stresses that exceed the preconsolidation pressure.

Another case study is presented in the paper by Gottardi & Tonni (2005). This involved the construction of a 6.7m high, 40m diameter, vertical walled cylindrical test bank on the Pleistocene silty sediments that under-lay the Venice lagoon. The test not only helped in the development of a reliable geotechnical model for the foundation design of the submersible gates intended to protect Venice and its lagoon environment against flooding, but also for the investigation of the potentials of in-situ testing.

In the paper, predictions of settlement based on two well established empirical formulae (i.e. those of Lunne & Christophersen (1983) and Kulhawy & Mayne (1990)) for obtaining the one dimensional constrained modulus from the results of cone penetration test data are compared with the measured behaviour. It is shown that the predictions severely underestimate the measured behaviour. The paper then uses the field data, and in particular the results from sliding deformeter instruments, to determine average site-specific coefficients for the two empirical formulae for each of the soil sub units in the foundation. These coefficients are smaller in magnitude than those usually recommended and vary from one sub unit to another. Not surprisingly, when used to calculate the total settlement they produce values in good agreement with the field data.

The paper hypothesizes that the discrepancy probably arises due to the highly stratified, rather complex, silty subsoils, whose behaviour is not typical of coarse-grained materials nor of fine-grained materials. In particular their partially drained behaviour under penetrometer testing is difficult to interpret and their time rate consolidation during the load test was very high, comparable with the loading bank construction time. Consequently secondary settlements cannot be clearly separated from the primary component.

The paper by Kuzakhmetova (2005) provides a qualitative description of some of the problems of predicting settlements of embankments constructed on soft soil. In particular it discusses the determination and influence of the presonsolidation pressure on the stress at which "filtration consolidation" begins. It is implied that this latter quantity is the minimum stress that must be applied before water is expelled from the soil. This leads the author to propose an approach for determining the active depth below the embankment (i.e. the depth above which settlements will occur) and within this zone the depth above which water will be expelled.

3 FOUNDATIONS

There are seven papers concerned with the behaviour of foundations. These are conveniently divided into those dealing with shallow foundations and those dealing with piled foundations.

3.1 *Shallow foundations*

The paper by Griffiths and Fenton (2005) investigates the reliability of shallow rectangular footings against serviceability limit state failure in the form of excessive settlement. The soil is modelled as a three dimensional spatially random linear elastic material in which the Young's modulus, E, varies but the Poisson's ratio is fixed at 0.3. A single value for the mean value of E combined with a range of standard deviations and correlation lengths are used assuming that the distribution of E is lognormal. Rigid, rough rectangular footings with aspect ratios of 1:1, 2:1 and 3:1 are studied. The analyses used the three-dimensional random finite element method which combines finite element analysis with random field theory. The Monte-Carlo approach is used to obtain the settlement statistics.

The paper concludes that the smaller the aspect ratio of the footing the higher the probability that the settlement will exceed its design value (obtained from a deterministic analysis). No comparisons are made with either laboratory model tests or field data.

The performance of three dimensional (3D) finite element models in geotechnical engineering is discussed by Mestat et al. (2005). The paper briefly reviews the limitations of two-dimensional (2D) analyses and states that for predicting displacements they are usually conservative. When reviewing 3D analyses the paper dismisses the use of the Fourier Series Aided Finite Element Method for non-linear problems. This is contradictory to the experience of the reporter who has shown that such methods are accurate and considerably more efficient in terms of computer resources than conventional 3D analyses, see Potts and Zdravkovic (1999, 2001).

The paper then argues that with present day computer resources its is not possible to analyse 3D problems with a sufficiently refined mesh (or more precisely a mesh with the same element density as traditionally used in 2D analyses). A footing and tunnel problem are used as examples.

Some results from a bench marking exercise are then presented. The problem to be solved is that of a vertically loaded square footing located near to the crest of a slope. The constitutive models for the soil and footing and their associated parameters are fully specified as is the main geometry. All participants had to do was select the distances to the boundaries of the mesh, produce a 3D finite element mesh and simulate the loading of the footing.

Results from four participants are presented. Three different finite element/finite difference software codes were used. A comparison of the results indicates significant differences, especially when approaching the limit load. Further sensitivity studies, varying the mesh density, indicted that it was not possible to arrive at a converged solution. For example, at a set footing load the footing displacement continued to increase as the mesh density increased. Consequently, it was not possible, with the computing resources available, to employ a mesh with a sufficient number of elements to overcome mesh discretisation errors.

The paper concludes that, while 3D numerical analyses are possible and in principal desirable, they should be used with caution. More experience with such analyses and an increase in available computing resources are required before the situation can improve.

The paper does not present any comparisons with field data. While the reporter is sympathetic to the conclusions given in the paper, he fells that they are too pessimistic. In his experience the results from 3D finite element analysis can be highly informative, especially in providing insight into mechanisms of behaviour. As noted by the authors, such analyses must of course be performed by personnel that have experience in the use of numerical analysis.

Predicting the settlement of shallow footings is also the subject of the paper by Bovolenta and Berardi (2005). Here it is recognized that the dependence of soil stiffness on both stress and strain level is of paramount importance if realistic predictions of settlement are to be achieved. By comparing the results from triaxial tests and plate loading tests on dry uniform Ticino sand under normally consolidation conditions, the paper develops two empirical relationships. The first of these expresses how the ratio $(\delta/B)/\varepsilon_a$ varies with stiffness ratio E'_s/E'_o, where B is footing width, ε_a is axial strain in the triaxial test, and E'_s and E'_o are the mobilised and initial secant Young's moduli respectively. The second expresses how the ratio of field stiffness to laboratory stiffness $(E'_s/E'_o)^{\text{field}}/(E'_s/E'_o)^{\text{laboratory}}$ varies with $\delta/B=\varepsilon_a$. Both these expressions are stress level dependent.

These expressions are then combined with the familiar elastic settlement equation $\delta/B=q.I_s.(1-\mu'^2)/E'$ and the decay in stiffness obtained from an appropriate triaxial test, and used to obtain predictions for four field cases. In all cases the predictions are in excellent agreement with the measured behaviour. As an example, the comparison between the predicted and observed behaviour of 3m square footing is reproduced in Fig. 7.

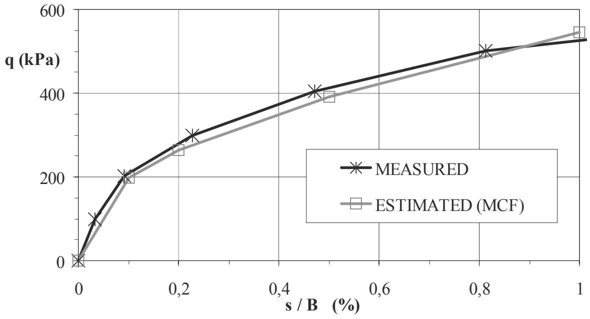

Figure 7. Computed and measured load-settlement curves.(after Bovolenta & Berardi, 2005)

The prediction of the dynamic behaviour of rigid block type machine foundations is considered in the paper by Puri et al. (2005). At present there are two accepted design approaches available, namely the elastic half space method and the impedance-compliance function approach. However, there are, according to the paper, few case studies in which predictions have been compared with field measurements.

This paper attempts to rectify this situation by presenting comparisons of the computed and observed response of two concrete block foundations (1.5m x 0.75m and 3.0m x 1.5m in plan and both 0.7m high). Both foundations were subjected to vertical excitation and the natural frequency and the damped amplitudes of vibration determined for a range of excitation frequencies. The dynamic soil properties of the foundation soil were determined from a range of laboratory and in-situ tests. These were then used with the two design approaches to predict the behaviour of the foundations.

Comparison of the predictions with the measurements indicated that both design approaches were able to predict the natural frequencies of vibration to within 15 to 20% of the measured values. In contrast, neither design method was accurate in predicting the damped vibration amplitudes. However, while the elastic half space method predicted values 2-4 times those measured, the impedance function method showed “a somewhat more reasonable agreement with the observed values”.

Predicting the settlement of a 40-story tower block founded on volcanically derived soils/rocks (i.e. a deposit consisting of interbedded lava flows, volcanic tuff and pumice layers) is considered in the paper by Justo et al (2005). The site investigation included both laboratory and in-situ tests and the results indicated that under the induced loading of the tower the foundation behaviour would remain elastic.

Elastic moduli obtained from unconfined compression tests on core samples instrumented with strain gauges were dismissed as being two high and therefore unrepresentative of in-situ conditions. Instead, lower bound values were obtained from pressuremeter tests and upper bound values were derived from the total rock mass rating using an empirical relationship. These two sets of parameters were then used in three-dimensional elastic finite element analysis in which both the foundations and the structure were modeled. The results of the analysis using the lower bound stiffness values gave a maximum settlement of 17mm, whereas only 1mm of settlement was predicted using the upper bound stiffness values.

Measurements were made on site and the paper uses the results from extensiometer measurements, together with the theory of Steinbrenner, to back calculate Young’s moduli. These values lie between the upper and lower bound values described above. The maximum measured settlement of 4.4mm is also between the predicted values quoted above. No further comparison is made between the settlements predicted by the finite element analyses and the field measurements as the authors claim that this will form the topic of a forthcoming paper.

3.2 *Deep foundations*

The paper by Arrua et al (2005) examines the application of reliability-based design to piles constructed in collapsible Argentinean Loess. In this material the soil resistance is highly dependent on the degree of saturation. For example, under partially saturated conditions shaft friction is much higher than when the loess is fully saturated. This gives problems when a pile is constructed in loess with a low degree of saturation and then at a later date the degree of saturation increases. In such a scenario the available shaft friction decreases and it is possible that load originally taken by the shaft has to be transferred to the base of the pile. This results in pile settlement that can have detrimental effects.

The paper applies a reliability based design approach to both floating and end bearing piles. The friction angle and cohesion of the loess are considered as two random variables. Bearing capacity and settlements are then computed considering the mean and standard deviation of both. Both short-term analysis, based on total stress parameters (pile installed in partially saturated loess), and long-term analysis, based on effective stress parameters (pile installed in saturated loess), are considered.

The paper concludes that reliability analysis enables geotechnical engineers to better understand the behaviour of piles in loess. The magnitude of the safety factor, determined by a deterministic analysis, does not give accurate information on the reliability of the pile. For example, for the same probability of failure, the paper shows that the safety factor for floating piles should be almost 2.5 times that used for end bearing piles.

It is also concluded that, for a given vertical load, floating piles have a higher probability of exceeding a settlement of 0.02m than end bearing piles for both short and long term conditions. In addition, the probability that settlements exceed 0.02m is higher for long-term than for short-term conditions.

The paper by Juarez-Badillo (2005b) uses the principle of natural proportionality to derive equations for the load displacement behaviour of piles. Vertically loaded piles and pile groups in tension and compression and laterally loaded piles are considered.

The equations are empirically fitted to test data using some of the measurements, past experience and in some cases a trial and error approach. Many field cases are considered and it is shown that the equations provide a good fit to the measured behaviour. However in all the cases presented the equations are applied once the field data was known. It would have been more convincing if the equations were applied using some of the earlier results from a field test and then used to predict the remainder of the field test before it had been completed.

4 EMBANKMENT DAMS

Two papers are concerned with embankment dams. Both compare predictions using the PLAXIS finite element software with field measurements.

The paper by Sadrekarimi and Kia (2005) considers a 70m high central clay core dam constructed in the north west of Iran. The dam was instrumented with a variety of devices to monitor pore water pressures and vertical and horizontal movement, but the paper concentrates on the settlements of the dam during construction. The monitored behaviour is compared with two finite element predictions. It is claimed that in one of these the Mohr-Coulomb model is used to simulate soil behaviour and in the other the Soft Soil Creep model is used. However, the paper does not provide parameters for the Soft Soil Creep model for the embankment shell material and no properties are provided for the foundation at all. It also specifies a range of values for some of the parameters. It is therefore unclear to the reporter as to what properties were actually used in each of the two analyses. In addition, the finite element mesh and boundary conditions employed in the analyses are not described.

Comparison of the predicted and measured vertical settlement of the dam is presented. For dam construction up to approximately three quarters its final height both finite element analyses give predictions in good agreement with the measured behaviour. The predictions from the Mohr-Coulomb model underestimate, and those from the Soft Soil Creep model overestimate, the measured settlements. At this stage in the dam construction impounding begins and there is a sudden increase in reservoir level.

At completion of construction the settlements from the Mohr-Coulomb model are much less than those measured, whereas those from the Soft Soil Creep model are in good agreement with the observations. It is implied by the authors that this difference is a consequence of the Soft Soil Creep model being able to reproduce creep behaviour, whereas the Mohr-Coulomb model could not. However, this appears to contradict an earlier statement in which it is claimed that after construction was complete no further settlements had been measured, thus indicating that creep was not a major factor influencing settlement.

Matesic and Kvasnicka (2005) present results from an analysis of a 20m high central clay core dam constructed on the Istria peninsula in the Adriatic Sea. The purpose of the analysis was to predict the past behaviour of the dam, validate this by comparison with field measurements and then assess future modifications to the dam. Dam construction began in 1981 and was completed three years later. However field measurements did not commence until two years after construction was completed (i.e.1986).

The Mohr-Coulomb model is used to simulate the dam shoulders and filter layers and the Hardening Soil model to simulate the dam core and the foundation. The parameters were determined from laboratory and in-situ tests.

The paper shows good agreement between the predicted and observed settlement and lateral movement of the dam crest. During dam construction a portion of the dam failed due to foundation instability. This occurred because the dam was built too quickly, not allowing for the required dissipation of excess pore water pressures. The failure was successfully back analysed, giving confidence in the soil parameters used and in the calculated current factor of safety of the dam (i.e. F=1.4). In the reporter's opinion this stability part of the paper would have benefited from a clear description on how the undrained strengths quoted where reconciled with the effective stress parameters input into the finite element analysis.

5 RETAINING WALLS

The design and construction of a massive quay wall in the harbour of Antwerp in Belgium is the subject of the paper by Van Alboom and Baertsoen (2005). The L shaped reinforced concrete quay walls are of the gravity type, with a height of 30m and a base width of 22.5m to 24.0m. They are constructed in a deep excavation that has to be dewatered. However, to prevent subsidence to adjacent infrastructure and housing, re-charge wells are also used in combination with a 1km long hydraulic cut-off wall some 50m deep.

The paper describes the nature of the geotechnical investigation, the soil profile and the geotechnical parameters selected for design. It also describes the monitoring programme that was installed and in particular that related to the measurement of wall movements.

The measured behaviour of the wall is then compared to predictions from finite element analyses, of which few details are given. From the information provided it appears that the soils were modeled using a linear elastic perfectly plastic Mohr-Coulomb model.

The predicted movements do not match well with the measured behaviour. In order to improve matters the design parameters were revisited and evidence, from cone penetrometer tests, was found to justify increasing the stiffness of the backfill by a factor of approximately 5. A further numerical analysis was performed and while the results are an improvement, they are still not in good agreement with the measured behaviour.

In a further attempt to simulate the measured movements, a sensitivity analysis was performed varying the strength and density of the backfill, the roughness of the interface between the vertical face of the quay wall and the backfill, and by introducing an interface along the slopes of the excavation pit. However, it was not possible to select a combination of parameters that gave predictions in good agreement with the measurements. In particular, none of the analyses predicted the backward movement of the wall during the dredging stage of construction, as measured in the field.

Back analysis of two sections of a diaphragm wall that was constructed in Colombes (France) is discussed in the paper by Serrai et al. (2005). The wall is supported by two rows of anchors and retains alluvium. The authors note that good agreement between field measurements of lateral wall movements and predictions from subgrade reaction coefficients was obtained and that this comparison has been presented in a previous publication. The present paper presents the results from additional back analysis using the finite element method. The objective being to establish the most appropriate constitutive model and its associated parameters. The PLAXIS software has been used and analyses performed using the Mohr-Coulomb, Hardening Soil and Soft Soil models. Results from these analyses have been calibrated against the measured lateral wall movements and the bending moments determined from the analysis using subgrade reaction coefficients.

An extensive discussion on the selection of appropriate soil parameter is given. For example, it is concluded that to obtain reasonable predictions with the Mohr-Coulomb model, which assumes constant elastic properties, the Young's modulus, E should be set equal to four times the Young's modulus determined from a pressuremeter test. In contrast, for the Hardening Soil model in which the stiffness variation with strain follows approximatly a hyperbolic function, the input parameter E_{50} should be set to two times the Young's modulus determined from a pressuremeter test. It also concludes that for accurate back analyses to be performed field measurements at all major stages of construction are required in order to fully validate any analysis.

In contrast to the above paper, Vanoudheusden et al. (2005) conclude that analyses based on the subgrade reaction method (SGRM) do not provide realistic predictions of retaining wall behaviour. They compare SGRM predictions made during the design of retaining walls that support an excavation for a metro station with measurements made during its construction. The station is located close to a canal in an area that has been designated a world heritage site by UNESCO. Consequently, ground movements and their effect on adjacent structures and services was of concern. Therefore the retaining walls are supported by two rows of concrete struts and two levels of ground anchors.

The SGRM simulated soil behaviour with elastic perfectly plastic springs, the parameters for which were derived from results based on a site investigation. A comparison of the SGRM predictions with the measured behaviour indicated that the maximum measured lateral wall movement was 240% greater than that predicted and the maximum measured forces in the top and bottom rows of struts was approximately 400% and 130% greater than that predicted respectively. However, the maximum predicted bending moments and the anchor forces were comparable. As a consequence of the underestimation of the strut forces by the SGRM during design, an extra row of struts were added immediately above the top row during construction.

After construction a sensitivity analysis was undertaken using the SGRM. While it was possible to select spring parameters that enabled a match between prediction and measured behaviour during the early stages of construction (i.e. up to construction of the second level of anchors), the predictions for the later stages diverged significantly from the measured behaviour. It was concluded that the SGRM could not deal with a wall supported simultaneously by two levels of struts and two levels of anchors.

Finite element analyses were then performed using the PLAXIS software and the Mohr-Coulomb model. The soil parameters were based on the same site investigation data that was used to determine the spring parameters for the SGRM analyses. These analyses gave lateral wall movements in good agreement with those measured and therefore considerably larger than those from the SGRM. The maximum bending moments, however, were similar to those predicted by the SGRM. The prop forces predicted in the top row of struts were approximately 50% larger than those measured.

A further finite element analysis was performed in which the full excavation was modelled. Note, for the first finite element analysis and the SGRM analyses symmetry was assumed about the vertical centre line of the excavation. Results from this analysis were similar to the previous finite element analysis except that the prop forces were lower, which meant that they under-estimated the measured values in the early stage of construction, but over-estimated them in the later stages.

This case is interesting in that it compares results from SGRM and finite element analysis with field measurements. It also, perhaps not surprisingly, concludes that the SGRM approach cannot deal with the complexities associated with two levels of struts and anchors acting simultaneously. One reason given for this is the inability of the SGRM to realistically simulate soil arching. The reporter is unclear as to why the finite element and SGRM analyses gave similar wall bending moments, as the associated lateral wall deflections were very different both in magnitude and distribution. In this respect a figure comparing the predicted and measured bending moment distribution down the wall would have been helpful.

Another field case is described in the paper by Simon and Barras (2005). This concerns the failure of a quay wall in the port of La Rochelle (France). The wall was constructed in 1982 and consists of 1.2m diameter steel tubes at 1.7m centres, connected by steel sheet piles. The retained height is 23.5m and the wall is embedded 4m into limestone. Each steel tube is anchored back to a sheet pile anchor wall installed 30m behind the main wall. To complicate matters one of the rails for the dock crane is supported on the capping beam that is constructed on top of the main wall, whereas the other rail is supported on a row of vertical piles and is connected to the capping beam with structural members. This arrangement resembles a conventional relieving platform.

In February 2001 the wall suddenly moved horizontally at its top (by approximately 35cm) over a length of 100m. Further movements were arrested by placing a sand berm in front of the quay. Field investigations indicated that the movements arose due to the breakage of several of the anchor tendons. This mechanism of behaviour was corroborated by the results of two dimensional finite element analyses using the PLAXIS software. The history of construction was simulated in the analyses and it was shown that substantial anchor loads were generated by deflection of the anchor tendons due to backfill settlement. In addition, adjacent to the main wall, the tendons supported the overlying backfill generating both tensile forces and bending moments in the connection with the main quay wall.

Further numerical analyses were performed in order to develop remedial measures. These consisted of a combination of controlled temporary excavation behind the main wall and prestressing through temporary anchors as a way to restoring initial alignment of the quay. The wall behaviour during these operations agreed well with the predictions from the numerical analysis. Permanent stabilisation required the installation of a new anchor wall behind the existing one and new anchor tendons.

The measurement of bending moments in embedded concrete retaining walls is discussed in the paper by Clark and Richards (2005). Using the example of a propped contiguous bored pile retaining wall constructed in Ashford in the United Kingdom as part of the Channel Tunnel Rail Link, they compare bending moments derived from inclinometer measurements with those obtained from vibrating wire strain gauges.

They show that care must be exercised when processing the inclinometer data. They also show how the strain gauge data can be used to determine if cracking of the concrete piles has occurred. By comparing the bending moments derived from the strain gauge data with those from the inclinometers for different stages of excavation in front of the wall, they show that while no concrete cracking occurs good agreement is obtained, however once cracking occurs there are substantial differences. This is especially so at the top and bottom of the wall, where the strain gauges indicate low bending moments compared to those derived from the inclinometer data. Also, in these regions, the inclinometer data is sensitive to the order of the polynomial used to represent the lateral movements recorded.

The viscous deformation of geogrid-reinforced sand is discussed in the paper by Kongkitkul and Tatsuoka (2005). They present results from tension tests on samples of geogrid and plane strain compression tests on samples of sand and geogrid reinforced sand. All samples indicate viscous behaviour that can be simulated within the framework of a non-linear three-component rheology model. In particular the geogrid reinforced sand exhibited significant viscosity. The test results and their analysis by the three-component model indicated that the tensile force in the geogrid decreased with time during sustained loading of the reinforced sand. This has important implications for the design of reinforced walls and embankments both in terms of ultimate and serviceability limit states.

6 NEURAL NETWORKS

The paper by Koelewijn and Maccabiani (2005) presents three examples of the use of artificial neural networks.

The first example describes the use of a neural network to convert model parameters. In the Netherlands most settlement predictions are based on the Koppejan model that combines one dimensional consolidation theory with a theory for creep deformation. It requires two material parameters, namely C_p, the primary compression index and C_s, the creep coefficient. While there is much experience with the use of this approach, there are several major drawbacks. This has lead to the development of an alternative approach based on isotache theory which requires three material parameters *a*, *b* and *c*. While this new approach provides more accurate predictions of settlement than the Koppejan model, its widespread adoption has been hindered by engineers lack of familiarity, especially with the likely values of the parameters *a*, *b* and *c*.

In the paper two approaches to convert values of C_p and C_s to *a*, *b* and *c* are considered. The first uses traditional two dimensional regression analysis and the second uses a neural network. Both approaches use the results from 572 oedometer tests. It is shown that the neural network approach produces better correlations between the sets of parameters. In addition, these correlations were derived in a fully automated and unsupervised manner. The example shows that automated neural networks may have a wider application for highly automated tasks in geotechnical engineering.

The second example considers the application of neural networks to assess the safety classification of dikes. It uses historical data including many case records from the 1950's when there was a severe flood surge that destroyed many dikes. A total of 90 dikes were used in the study. The example shows the possibilities of using neural networks to learn from past experience, even if this is not well documented and information on some of the most important parameters is missing (i.e. subsoil properties and information on wave direction). It also indicated the advantages of combining neural networks with expert knowledge.

The third example investigates the use of neural networks to assess the safety of embankments. Apparently in the Netherlands every embankment has to have its safety re-evaluated every five years. Due to the large number of embankments this is a formidable task. The paper shows how a neural network can be used to determine the safety factor of an embankment based on 50 input parameters that describe the geometry, ground water conditions and subsoil stratification and material properties. This leads to a tremendous increase in efficiency over traditional approaches without loss of accuracy.

The paper by Rankine & Sivakugan (2005) uses artificial neural networks to predict the performance of paste backfills when used to support old mine stopes. In order to provide stability, paste fill must remain stable during the extraction of ore and minerals from neighboring stopes. Consequently, the amount of cement that is mixed with the mine tailings to form the paste is pivotal. As filling costs for a mine typically represent approximately 20% of all mining costs, with the cement costs constituting approximately 75% of that amount, it is also important to optimize the amount of cement used.

The paper describes how three-dimensional FLAC analyses were used to determine the maximum vertical stress in a stope as a function of its geometry (i.e. width, breadth and height). By assuming that this stress does not exceed the uniaxial compressive strength of the backfill enables the strength requirement of the paste to be estimated. This information, along with data collected from various sources on paste backfill, including its grain size, composition, curing time and uniaxial compressive strength, is then used to construct artificial neural network models. These can be used to predict the paste backfill strength and consequently the optimum cement content of the paste mix depending on the user defined input parameters for the stope geometry, material bulk density, curing time and solids density. The paper presents an example of the application of the method to a real field case where the potential of the approach to provide commercial savings is clearly shown.

7 SLOPE STABILITY

Slope stability in Nepal is the subject matter of the paper by Tiwari and Pokharel (2005). Due to the fragile geology, steep terrain, and improper construction practices, many of the Nepalese highways are suffering from landslide problems. In order to establish systematic landslide investigation and stabilization practices, the Nepalese government has recently studied four landslides in detail. Three of these are described in the paper.

In all cases two exploratory boreholes were sunk in order to determine the position of the slip surface at discrete positions. These positions were then used in combination with the results from finite element analyses to estimate the location of the complete slip surface. Conventional Limit Equilibrium stability analyses were then used to estimate the increase in stability as a result of various stabilization options. In all cases stabilization was achieved through a combination of toe weighting, a toe retaining wall of some form, surface drainage and deep drainage from boreholes. It is claimed that both the investigation and stabilization techniques have general applicability.

Unfortunately, the paper does not describe the finite element analysis in any detail. In particular, it is not clear if they were two or three dimensional analyses or how the initial slope conditions (i.e stresses) were simulated. As no results from the finite element analyses are presented in the paper it also unclear how they were used to determine the position of the slip surface.

8 TUNNELS

In the paper by Huang et al (2005) both the long and short term ground surface settlements above Shanghai metro line No. 2 are discussed. The tunnel has an external diameter D=6.2m and the average depth to the tunnel centre line is 1.8D. It was constructed in soft clay using an earth pressure balance tunneling machine.

Results from plane strain finite element analyses using a visco-plastic constitutive model based on the Modified Cam Clay model are compared with field measurements. The boundary conditions employed to model tunnel construction, although stated, are not precise and no soil properties are quoted (a reference to a paper is given).

A comparison between the measured ground surface settlements and those predicted from the finite element analysis show good agreement, especially for the long-term condition. Both the measurements and the predictions indicate that the magnitude of the long-term settlements are nearly twice those occurring in the short-term. The results are reproduced in Fig. 8.

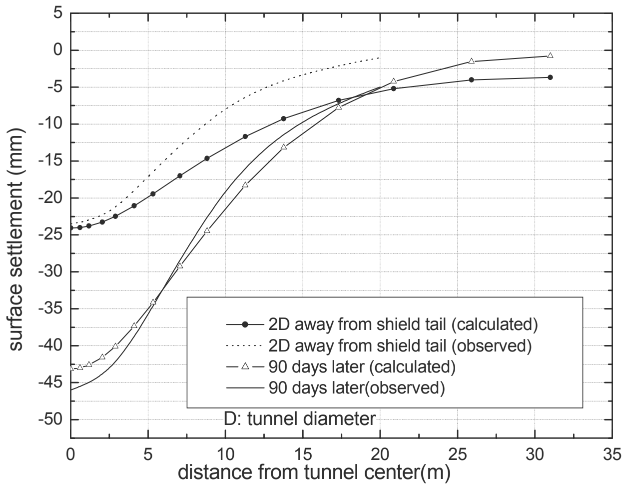

Figure 8. Comparison of calculated and in-situ measured surface settlements (after Huang et al., 2005)

Results from a finite element sensitivity study varying the degree of over excavation (i.e. volume loss) occurring during tunnel construction are also presented. It is shown that increasing the amount of over excavation increases both the ground surface settlements occurring during the short term (i.e. during tunnel construction) and in the long term (i.e. in the period after tunnel construction). A comparison of these latter results with the field measurements indicate that in the field over excavation has a larger effect (i.e. approximately twice that recovered from the numerical analyses).

9 PARTIALLY SATURATED SOILS

A simple testing technique to provide reliable data of the Soil Water Characteristic Curve (SWCC) is described in the paper by Bicalho et al (2005). Importantly, the test enables an accurate estimate of the SWCC to be obtained when the soil contains occluded air bubbles and therefore the air phase is discontinuous. The test involves the use of a modified triaxial cell connected to a flow pump. It allows continuous SWCC data to be obtained and consequently only one test is required to describe drying and wetting cycles of the SWCC for a particular soil. However, the interpretation of the test is based on the assumption that the overall volume of the soil remains constant. Consequently, further research is needed to investigate the effect that volume change will have on the derived SWCC.

Results from tests on a silt soil during the wetting process when occluded air bubbles are present are compared with a theoretical relationship for the SWCC derived using the occluded state theory proposed by Schuurman (1966). Few details of the derivation of this relationship are provided as these have been published by the authors elsewhere. The agreement between the experimental measurements and the theory is excellent, see Fig. 9, providing encouraging evidence of the validity of the occluded state theory and its inherent assumptions.

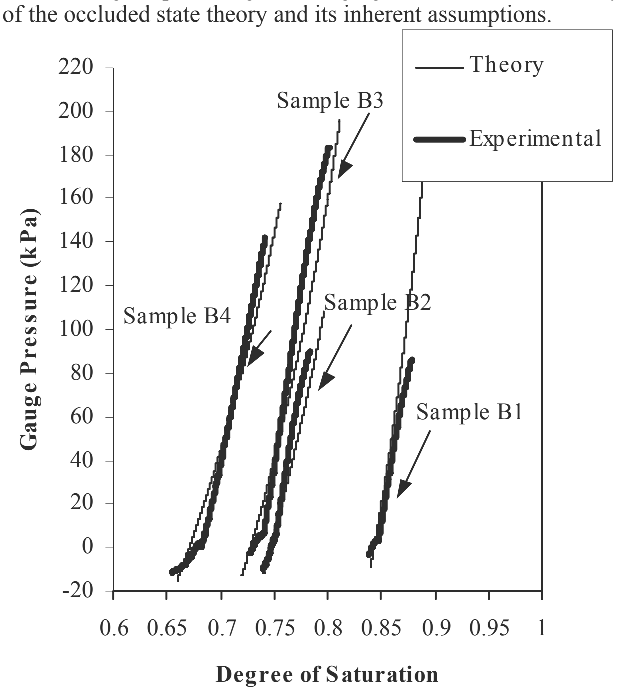

Figure 9. Water pressure measured and predicted from theory (Bonny Silt) (after Biacalho et al., 2005)

10 DISCUSION

Some of the papers reviewed above present information that, at first sight, may seem contradictory and warrant further debate. In this respect the reporter notes the following:

There has been a long running debate in the literature as to whether creep occurs concurrently with primary consolidation (i.e. during excess pore water pressure dissipation) or only after this has completed. The paper by Tanaka et al. (2005) provides additional data in this respect implying the former occurs for overconsolidated clays while the latter occurs for normally consolidated clay.

Several of the papers (i.e. Lansivarra (2005), Ravaska (2005), Juarez-Badillo (2005a) and Akai & Tanaka (2005)) propose empirical equations for predicting the long-term settlements on soft clay. A comparison of the strengths and weaknesses of these different approaches would be enlightening.

In the paper by Serrai et al (2005) it is claimed that predictions of retaining wall behaviour based on the subgrade reaction method are in excellent agreement with field measurements. However Vanoudheusen et al (2005) conclude that this approach could not reproduce the observed behaviour of their multi-propped wall. It would be interesting to debate the limitation of this approach and establish for what types of wall it is applicable.

Some of the papers (Serrai et al (2005, Vanoudheusden (2005)) indicate that using the Mohr-Coulomb model with finite element analysis produces good predictions, others show this not to be so and that more sophisticated models are needed (Alboom & Baertsoen (2005), Sadrekarimi & Kia (2005)). Some guidance on the use of this simple model would be helpful.

Some of the problems of performing three-dimensional numerical analysis have been highlighted by Mestat et al (2005). As such analyses are becoming more common the experiences of other users could be of interest.

Any of the above topics would be relevant for the discussion session at the conference.

PAPERS IN TECHNICAL SESSION 1F

Aalto, A., Vepsalainen, P. and Ravaska, O. 2005, Settlement calculation with stress dependent parameters.
Akai, K. and Tanaka, Y. 2005, Ex-post-facto estimate of performance at the offshore reclamation of Airport Osaka/KIA.
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Juarez-Badillo, E. 2005b. The principle of natural proportionality applied to the behaviour of piles.
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Mestat, P., Bourgeois, E., Humbert, P. and Riou, Y. 2005. Performances of 3D finite element models in geotechnical engineering.
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Sadrekarimi, J. and Kia, M. 2005. Performance of a large dam, filed measurements and analytical approach.
Serrai, K., Plumelle, C. and Schmitt, P. 2005. Analysis of measured deflections of a diaphragm wall in Colombes using finite element calculations.
Simon, B. and Barras, P. 2005. How a distressed quay wall could be moved back in place.
Tanaka, H., Udaka, K. and Nosaka, T. 2005. Consolidation behaviour around p_c value by inter connected iedometer test.
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