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Analysis of full-scale field test results for the design of ground improvement at the New Bangkok International Airport
L'analyse des resultats des essais a grande echelle pour le project d'amélioration du sol au Nouveau Aéroport International de Bangkok

C. Athanasiu, A. Bye & J.A. Finstad
NOTEBY A/S, Consulting Engineers, Oslo, Norway

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ABSTRACT: In conjunction with the design of ground improvement by pre-loading with prefabricated vertical drains (PVD) for the New Bangkok International Airport (NBIA), three full scale tests were conducted at NBIA site at Nong Ngu Hao. Settlements, pore water pressures and lateral movements were recorded over a period of two years. A number of site specific conditions such as: non-hydrostatic "in situ" pore water pressure, high lateral displacements under fill load, stress-dependent permeability and relative high apparent pre-consolidation of the upper layers of Bangkok clay at NBIA made it a difficult to back-calculate the full scale test results. The paper describes how these specific conditions are simulated in the numerical analyses of consolidation process with PVD. The "calibrated" compression and consolidation soil parameters that lead to good agreement between results of full-scale field tests and of numerical analyses are discussed in view of their practical implications of the design.

RÉSUMÉ: Le project d'amélioration du sol par surcharge avec des drains verticaux prefabriqués a Nouveau Aéroport International de Bangkok a été "verifié " prealablement par troi essais a grande echelle sur le site même de l'Aéroport, a Nong Ngu Hao. Pendant les essais, les tassements , les pressions intersticièlles et les mouvements lateraux ont été enregistrées pour une periode de deux années. Quelleques conditions specifiques au site comme par example: pressions intersticièlles, "in situ", non-hydrostatiques, deplacements laterales importantes sous les charges de ramblais, la permeabilité variable avec les efforts effectives et grandes pressions de préconsolidation de l'argile de Bangkok dans les couches de surface doivent etre prises en consideration dans un calcul invèrs des resultats des essais. L'article present les analyses numeriques effectuées pour "calibrer" les parametres de tassement et de consolidation a utiliser dans le design par le calcul invèrs de resultats des essais à grande echelle.

1 INTRODUCTION

Three test embankments, TS1, TS2 and TS3 were constructed at the site of the New Bangkok International Airport (NBIA) at Nong Ngu Hao . The tests were conducted under the supervision of Asian Institute of Technology (Balasubramaniam et al., 1995). Settlements, pore water pressures and lateral deformations were measured in order to calibrate the soil parameters for the design of the ground improvement by pre-loading with prefabricated vertical drains. In addition the performance of different types of drains installed with different spacing was also investigated. The ground conditions at Nong Ngu Hao are summarised as follows. Beneath a thin drying crust of less than 2 m, the very soft to soft clay layers extend to a depth of 12 m. Further down, the clay turns gradually into medium to stiff to a depth of about 20 m. The stiff clay is underlain by the first sand layer

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with very fine to medium size sand, extending to about 30 m. The "in situ" pore water pressures show a marked piezometric draw down due to pumping from deep wells in Bangkok region.

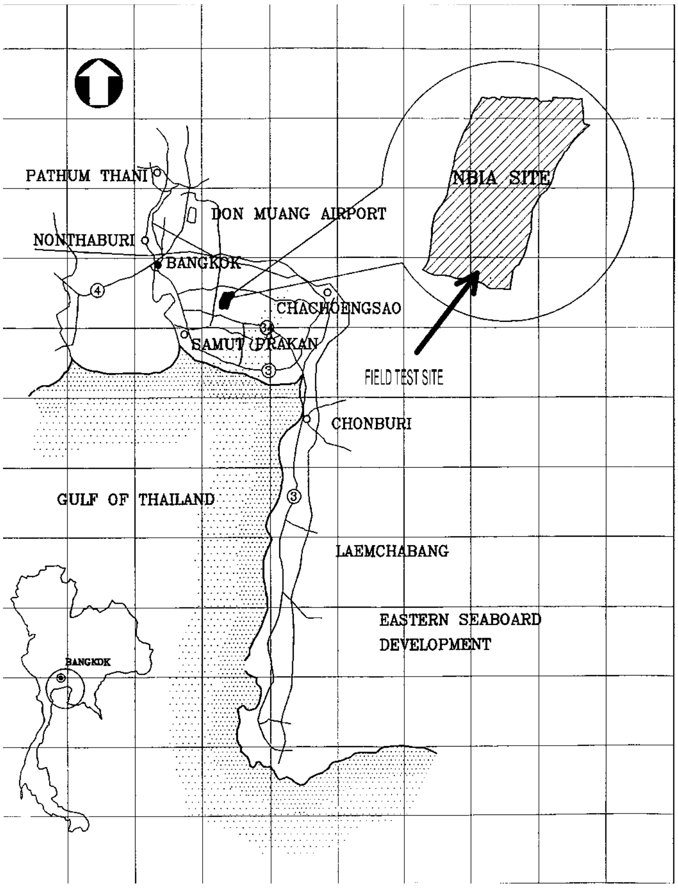

Fig.1 Field test area

The 40 m x 40 m embankments have side slopes of 3:1 with 5 m wide berms. For TS1 Flodrain at 1.5 m spacing were used to accelerate the rate of consolidation while Castle Board drains at 1.2 m spacing, and Mebra drains at 1.0 m spacing were used for TS2 and TS3 respectively. All drains were 12 m long in order to avoid any hydraulic connections between the drains and the deeper sand layer.

The paper presents the site specific problems encountered with back calculation of test results and the method used to calibrate the soil parameters for design purposes.

2 SITE SPECIFIC CONDITIONS

"In situ" pore water pressure. The initial measurements of the "in situ" pore water pressure reflects the excavation of the upper 0.6 m for preparation of field tests resulting in a pore pressure 8 – 10 kPa lower than hydrostatic. In depth, the difference to hydrostatic pressure increases due to low pressures in the underlying sand stratum. The pore water pressures just after the installation of the drains show an increase in pore water pressure towards hydrostatic due to drain conductivity. The result of this is a reduction of effective stresses and an increase in pre-consolidation of the soil.

Correction of pore water pressure readings due to large settlements. The piezometers settle with the surrounding soil and their new depth under the ground water surface will increase the pore water pressure readings. The corrected excess pore water pressure read by a piezometer installed at depth z is thus:

$$\Delta u = u_{measured} - \gamma_w \cdot (z + s_t) \tag{1}$$

where: $u_{measured}$ is the pore water reading at a time t; z and s_t are the installation depth from the ground water table and the settlement of the layers below the depth z at time t respectively; γ_w – unit weight of water.

Correction of settlement measurements due to immediate settlements and to large lateral movements. The measured settlements at soil surface and at different depths consist of immediate settlements, consolidation settlements, settlements due to large lateral deformations. The large lateral movements measured by inclinometers are mainly due to low factor of safety during construction of the fills which resulted in low shear stiffness of the clay. The vertical deflection of the soil surface is estimated from the condition that the volume of settlement should be equal to the soil volume moved laterally. The measured settlements were thus corrected by subtracting the deflections due to lateral movements before comparing to back calculated settlements.

Disturbance from drain installation. Drain installation disturbs the structure of the clay making it more compressible. It is found that reducing the past pre-consolidation pressure to account for disturbance effect gives best agreement with the observed settlement development.

3 METHOD OF ANALYSIS

The method of analysis used to back calculate the test results had to account for the 3-D effects of the test fills as compared with the purely 1-D consolidation under surcharges over large areas during pre-loading, in addition to the site specific conditions.

The vertical stresses under the centre of embankment are calculated using the Theory of Elasticity solution for rectangular areas. The procedure is incorporated in the program SETTL (NOTEBY – 1997). The ground surface area of test embankment is divided into a number of rectangular elements. The load from the fill is applied as uniform load over each element. The vertical and horizontal stresses at the centre of each layer under the embankment centre are calculated using superposition principle, by adding the contribution of each loaded element. The end of primary (EOP) settlements are then calculated for each nodal point of the surface elements by summing up the compression of each soil layer under the node. Fig.2 illustrates the results from SETTL for test fill TS3, showing the surface loading and the corresponding end of primary (EOP)

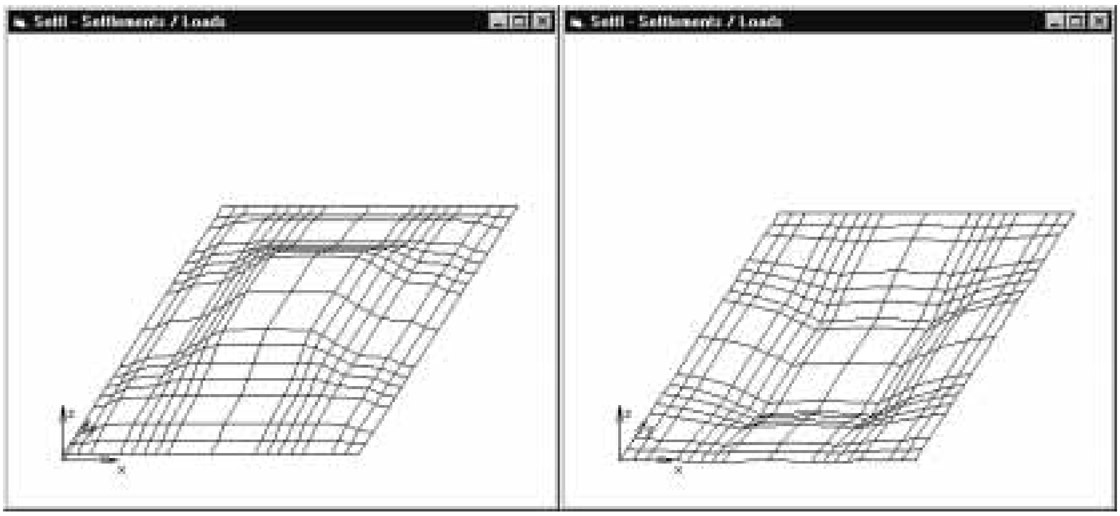

Fig. 2 Test fill TS3. Surface loading and settlements.

settlement pattern. The program SETTL is used to calculate the stress diagrams for different construction sequences of test embankments. These diagrams are then used as load increments for analysis of consolidation process.

The consolidation process is then analysed using the program CONS1-D (NOTEBY, 1996) for the axis of embankment. The program CONS1-D uses finite difference method to solve separately the differential equations for 1-D consolidation and for radial consolidation. Variable coefficients of vertical and radial consolidation are used in the program. The vertical and radial consolidation degrees with respect to pore water pressure, U_v and U_r are calculated for each depth. The resultant consolidation degree is obtained according to Carillo, 1944:$1\text{-}U=(1\text{-}U_v)\cdot(1\text{-}U_r)$ and the resulting pore pressure is then $u=(1\text{-}U)\cdot u_o$, where: u_o is the initial pore pressure due to applied total vertical stress at the depth into consideration. The radial consolidation degree can alternatively be calculated using the close form solution proposed by Hansbo (Hansbo, 1994):

$$U_r = 1-\left(1+\frac{t\cdot\lambda}{\alpha\cdot D^2}\cdot\sqrt{\frac{u_o}{D\cdot\gamma_w}}\right)^{-2} \tag{2}$$

where: t-the time elapsed since the application of the load; D-diameter of the soil cylinder around one drain;λ-consolidation coefficient for radial consolidation; α-coefficient depending on the ratio D/d ; d-diameter of the drain.

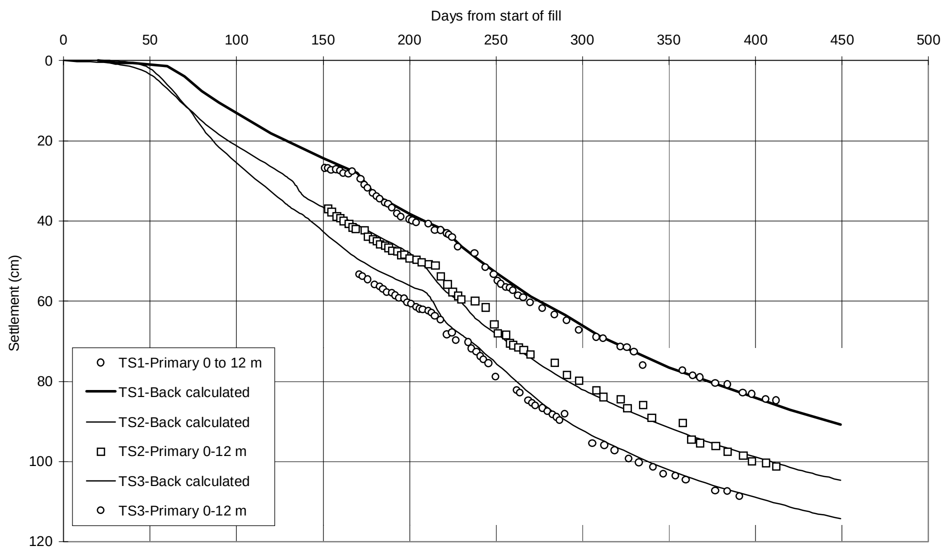

Fig.3 Comparison between measured and calculated settlements.

The program takes into account the site specific conditions as follows. If the "in situ" pore pressures are less than hydrostatic, the installation of the drains causes the pore pressure in the soil above the drain tips to change from "in situ" to hydrostatic, without any change in total stress, reducing the net effective stress that causes settlement. The program accounts for load changes due to either submerging the fill under the ground water table or lowering the water table as settlement occur by special drainage system in the surcharge fill. The load history is specified as a number of load increments (proportions of total load) each increment being applied at a specified number of time intervals. At every new loading increment, the total stress, the initial pore water pressure and the actual pore water pressure are increased by the load increment, dp. If the total stress diagram changes from one load increment to another, incremental diagrams may be specified instead of proportions of total load diagram.

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4 BACK CALCULATION OF TEST RESULTS

The back calculation of test results was performed in two stages: first, constant coefficients of consolidation with respect to void ratio were used with somewhat simplified method of consolidation analysis; later on, in the design process void ratio dependent consolidation coefficients were used. In both stages the smear zone was not modelled and average coefficients of consolidation in horizontal direction, c_h* were used. The main purpose of back calculation was to calibrate the compression parameters, $C_c/(1+e_o)$ and $C_r/(1+e_o)$ and the consolidation parameters: c_h*and λ*. The compression index $C_c/(1+e_o)$ was determined from correlation with water content. Fig. 3 shows the comparison between measured and back calculated settlements from 0 to 12 m for TS1, TS2 and TS3. The best agreement was obtained during the stage two back calculations when using consolidation coefficients variable with void ratio (i.e. effective stress). A plot of c_h variation with effective stress at different depths is shown in Fig.4 for TS3. It clearly shows the effect of pre-

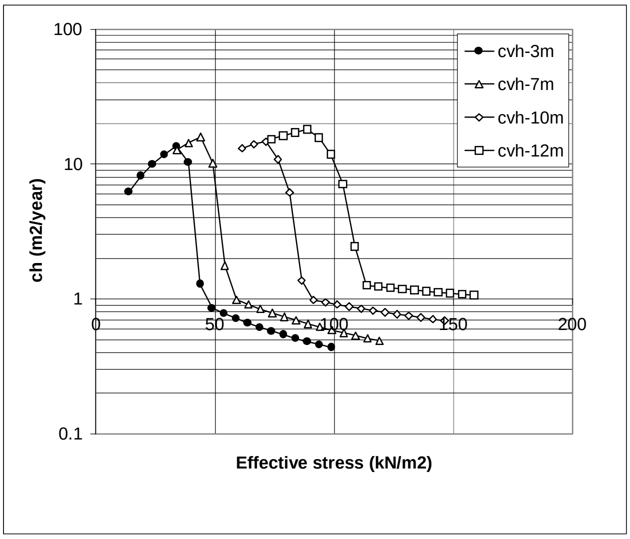

Fig. 4 Variable coefficients of radial consolidation

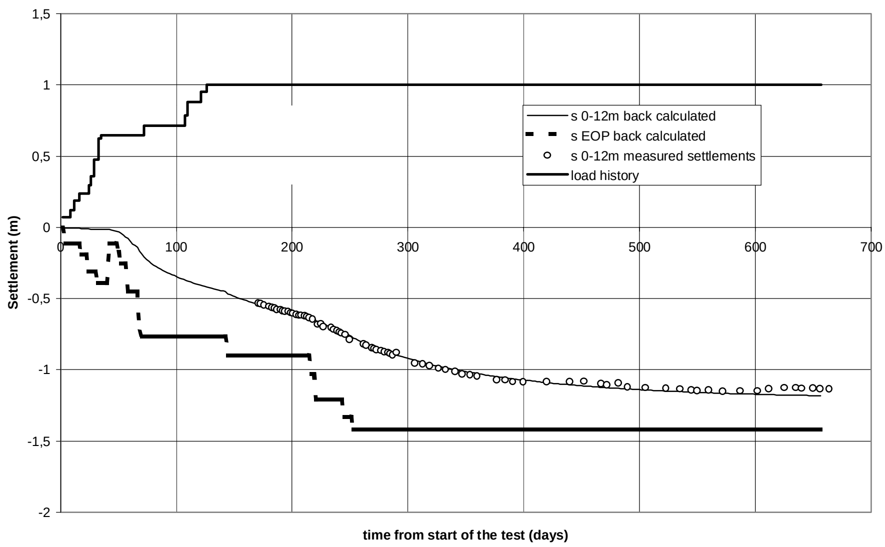

Fig. 5 Comparison between measured and back calculated settlements – TS3.

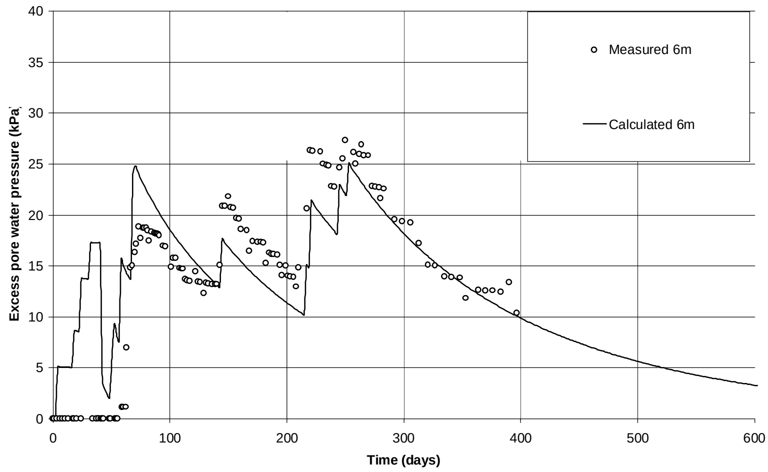

Fig. 6 Comparison between measured and back calculated pore pressures – TS3

consolidation at different depths. The mean value of c_h over the range of effective stress for each depth agrees well with the constant coefficient of consolidation obtained in first stage back calculation. Figs. 5 and 6 show the comparison measured / back calculated settlements from 0 to 12 m depth and pore water pressures, respectively for TS3.

5 CONCLUSIONS

Back calculations of the full-scale field test results at NBIA were performed in order to calibrate the soil parameters for the design of ground improvement by pre-loading with PVD. The best agreement between measured and back calculated settlements and pore pressures is obtained when 3-D and site specific are accounted for.

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