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Settlement prediction of embankments stabilised with prefabricated vertical drains at Second Bangkok International Airport
Prédiction d'aménagement au second aéroport international de Bangkok, de remblais stabilisés à l'aide de drains verticaux préfabriqués

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ABSTRACT: This study describes the settlement prediction including the effect of smear on three full scale test embankments raised on soft clay foundations, which are improved by installation of vertical drains at the site of the Second Bangkok International Airport. In the analysis, the classical axisymmetric solution for consolidation by vertical drain has been converted into an equivalent 2-D plane strain analysis. It is revealed that the inclusion of smear effects in single drain analysis improves the settlement prediction significantly, at the centerline of the embankment.

RÉSUMÉ: Cette étude décrit la prédiction de soubassement en incluant les effets d'enduit sur trois remblais testés à l'échelle, bâtis sur des fondations d'argile meuble et consolidés grâce à l'installation de drains verticaux au second aéroport international de Bangkok. La solution axi-symétrique classiquement employée pour la consolidation par drain vertical a été analysée en terme de pression plane bidimentionnelle équivalente. Il apparait que l'utilisation d'un enduit, pris en compte dans l'analyse d'un drain, améliore la prédiction d'aménagement de façon significative au niveau de la ligne médiane du remblai.

1 INTRODUCTION

In the past two decades, vertical sand compaction piles and prefabricated band drains have been used extensively for the purpose of soft ground improvement. The classical solution of vertical drains has been well documented (Barron, 1948; Hansbo, 1981) and has been generally used in the prediction of settlement using vertical drains. Due to the increasing popularity of plane strain finite element analysis, several models of equivalent plane strain solution have been introduced (Cheung, 1991, Hird et al., 1992). Hird et al., 1992, extended the axisymmetric solution of vertical drains including smear into a 2-D, plane strain analysis. Indraratna and Redana (1997), following Hird et al. (1992) and Hansbo (1981) extended the plane strain finite element analysis to include explicitly the effect of smear around the drains.

The occurrence of a smear zone is almost inevitable during the installation of vertical drains by means of a mandrel. Barron (1948), suggested the concept of reduced permeability which is equivalent to lowering the overall value of the coefficient of consolidation. Hansbo (1979) introduced a zone of smear with a reduced value of permeability. In this study, a smear zone with reduced coefficient of permeability is explicitly determined around the vertical drain, adopting a 2-D plane strain analysis. The analytical model in conjunction with the modified Cam-clay theory is then employed to predict the settlements and excess pore pressures along the embankment centre-line.

2 PLANE STRAIN MODELLING

Indraratna and Redana (1997) showed that if the radius of the axisymmetric influence zone of a single drain (R) were taken to be the same as the width (B) in plane strain (Fig. 1), then the converted plane strain ratio of the horizontal smear zone permeability, k'_{hp} to the undisturbed permeability k_{hp} could be given by:

$$k_{hp} = \frac{k_h\left[\alpha+(\beta)\frac{k_{hp}}{k'_{hp}}+(\theta)\left(2lz-z^2\right)\right]}{\left[\ln\left(\frac{n}{s}\right)+\left(\frac{k_h}{k'_h}\right)\ln(s)-0.75+\pi\left(2lz-z^2\right)\frac{k_h}{q_w}\right]} \tag{1}$$

In Eqn (1), after ignoring higher order terms,

$$\alpha = \frac{2}{3}-\frac{2b_s}{B}\left(1-\frac{b_s}{B}+\frac{b_s^2}{3B^2}\right) \tag{2}$$

$$\beta = \frac{1}{B^2}\left(b_s-b_w\right)^2+\frac{b_s}{3B^3}\left(3b_w^2-b_s^2\right) \tag{3}$$

and $$\theta = \frac{k_{hp}^2}{k'_{hp}Bq_z}\left(1-\frac{b_w}{B}\right) \tag{4}$$

The converted half width of drain (b_w) and the half width of smear zone (b_s) in plane strain are represented by the following expressions:

$$b_w = \frac{\pi r_w^2}{2S} \quad \text{and} \quad b_s = \frac{\pi r_s^2}{2S} \tag{5}$$

If both the smear and well resistance are ignored, then the simplified ratio of plane strain to axisymmetric horizontal permeability, k_h is represented by:

$$\frac{k_{hp}}{k_h} = \frac{0.67}{[\ln(n)-0.75]} \tag{6}$$

If the effect of well resistance is ignored, and only the effect of smear is taken into consideration, the smear zone permeability k'_{hp} is given by (Indraratna and Redana, 1997):

$$\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\frac{k_{hp}}{k_h}\left[\ln\left(\frac{n}{s}\right)+\left(\frac{k_h}{k'_h}\right)\ln(s)-0.75\right]-\alpha} \tag{7}$$

In the above, r_s = radius of smear, r_w = radius of drain, $s = r_s/r_w$, and $n = R/r_w$, S = spacing of the drains, B = width of unit cell in plane strain (B=R), k_h and k'_h = coefficient of horizontal permeability outside and inside the smeared zone, respectively. The above parameters are defined diagrammatically in Fig. 1.

In the case of Prefabricated Vertical Drains (PVD), the equivalent drain diameter is given by:

$$d = \frac{a+b}{2} \tag{8}$$

where, a = the PVD width and b = the PVD thickness.

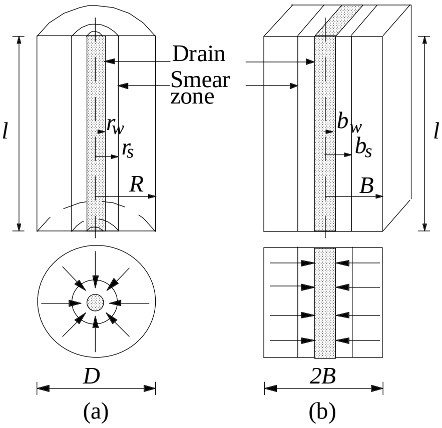

Figure 1. Conversion of an (a) axisymmetric unit cell into (b) plane strain (Indraratna and Redana, 1997).

3 SECOND BANGKOK INTERNATIONAL AIRPORT TEST FIELD

The three test embankments (TS1, TS2 and TS3) were located at the site of the Second Bangkok International Airport about 30 km east of Bangkok. The sub-soil properties including the Cam-clay parameters and the in-situ stress profile under the embankments are shown in Fig. 2. The upper sub-soil layer consists of a thin weathered clay crust (1.5 m deep) overlying a 12 m thick layer of soft Bangkok clay. A stiff clay layer underlies the soft clay and it extends to a depth of 20-24 m below the ground surface. During the wet seasons the area often gets flooded, and the soil generally retains a very high moisture content.

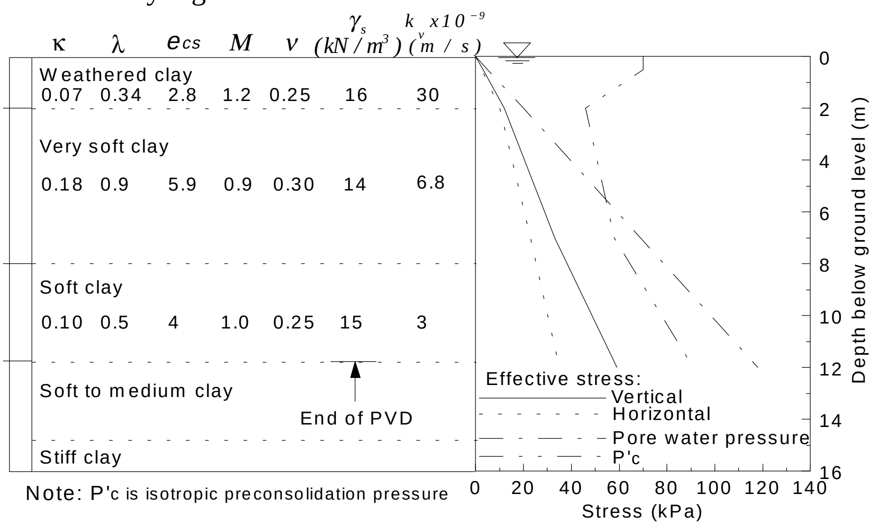

Figure 2. Sub-soil profile, Cam-clay parameters and stress condition used in numerical analysis, Second Bangkok International Airport, Thailand (after Asian Institute of Technology, 1995).

Three test embankments TS1, TS2 and TS3 which were 40 m x 40 m in plan with side slopes of 3:1 were constructed and stabilised with Prefabricated Vertical Drains (PVD). The drains were installed in a square pattern to a depth of 12 m. Three types of PVD were used, namely, Flodrain (FD4-EX), Castle Board (CS1) and Mebra (MD-7007) drains. The Flodrains (100 mm x 4 mm) were installed at 1.5 m spacing beneath test embankment TS1. Castle Board drains (94 mm x 3 mm) were installed at a spacing of 1.2 m beneath embankment TS2. Mebra drains (100 mm x 3 mm) were installed at 1 m spacing at embankment TS3. The drains were installed using a mandrel (125 x 45 mm) which was continuously pushed into the soil using a static weight, in order to reduce smear as much as possible.

The embankment was constructed in four stages. The rate of loading and the construction history of all three embankments are shown later in Fig. 4. The Stage 1 loading was equivalent to a vertical stress of 18 kPa, followed by Stage 2 loading (45 kPa), Stage 3 (54 kPa) and Stage 4 (75 kPa). In order to maintain stability of embankment TS1, a berm of 5 m width and 1.5 m height was also added, once the surcharge increased from 45 kPa to 54 kPa. For embankments TS2 and TS3, a berm of 7 m width was added when the surcharge load exceeded 54 kPa. The excess pore water pressures and settlements were measured for more than one year. The behaviour of the soft clay foundation was monitored using settlement plates, slope inclinometers and an array of piezometers, consisting of open standpipes, pneumatic and closed hydraulic piezometers.

The settlement behaviour of clay under loading was analysed using a finite element method, extended from the original CRISP (Britto and Gunn, 1987), incorporating the modified Cam-clay model (Roscoe and Burland, 1968). Further analysis was conducted based on several subroutines developed by the authors. Fig. 3b shows the finite element discretization of the embankment, which is composed of linear strain triangular (LST) elements having three pore pressure nodes. Due to symmetry, it was sufficient to consider one half of the embankment. The measured vertical permeability coefficients of the undisturbed soil (k_v) are given in Fig. 2, and their equivalent plane strain values were converted using Eqs. (6) and (7) based on the authors' model. Inside the smear zone, the horizontal permeability was taken to be equal to the vertical permeability and outside the smear zone the horizontal permeability was taken to be 1.75-2 times the vertical permeability (Bergado et al., 1991; Indraratna and Redana 1998a). The equivalent vertical band drain radius and the radius of mandrel (axisymmetric) were converted using Eq. 8, which gives r_w = 0.03 m and r_m=0.06 m, respectively. The radius of the smear zone was taken to be 0.3 m, which is about 5 times the radius of the mandrel based on large scale consolidation studies. The technique of estimating the extent of smear zone is described by Indraratna and Redana, (1998a). The equivalent plane strain width of drain and smear zone were determined using Eq. 5. In the analysis, the Cam-clay parameter λ was taken to be close to κ values in the first two stages of loading, where the pre-consolidation pressure of the soil is not exceeded. The clay layer is characterised by drained conditions at the upper boundary only, due to the presence of a stiff clay layer below 12 m depth.

The results of the plane strain analysis together with the measured settlements are shown in Fig. 4 for all embankments. The analysis based on perfect drain conditions (no smear, rapid pore pressure dissipation) overpredicts the measured settlement, but the inclusion of smear significantly improves the predictions. Particularly for TS1 and TS3, even a better match is obtained when the smear permeability (k'_{hp}) is slightly increased by a factor of 2-3. This suggests that in the field, the smear zone permeability is probably slightly greater than the values measured in the laboratory using the large-scale consolidation apparatus (Indraratna and Redana, 1998a). The predicted and measured excess pore water pressures along the centreline of the embankments at a depth of 8 m below the ground surface are compared in Fig. 5. The trend of pore water pressure increase is well predicted during Stage 1 and Stage 2 loading, but after Stage 3 loading, the predicted pore pressure is significantly greater than the measurements. Again, a partial smear condition gives a better prediction, during stage 4, where the smear permeability is increased slightly. As expected, the perfect drain predictions underestimate the actual pore water pressures. In the finite element model, the sharp increase in pore water pressure corresponding to peak loading stage is not always reflected by the measured values. This is because in practice, the load application is more gradual and some-times not uniform, hence a sudden increase in pore pressure (as modelled in the numerical analysis) is generally less marked.

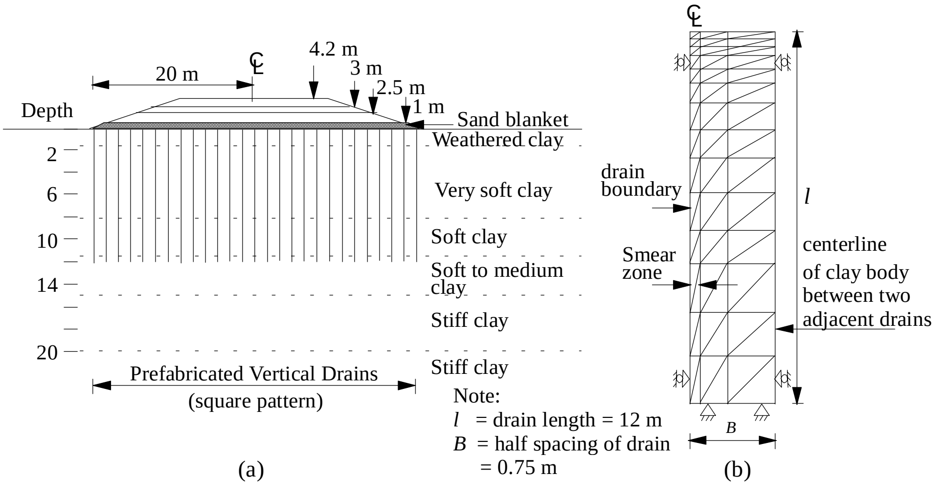

Figure 3. (a) Typical embankment with vertical drains and (b) Finite element mesh in the vicinity of drain (Indraratna and Redana, 1998).

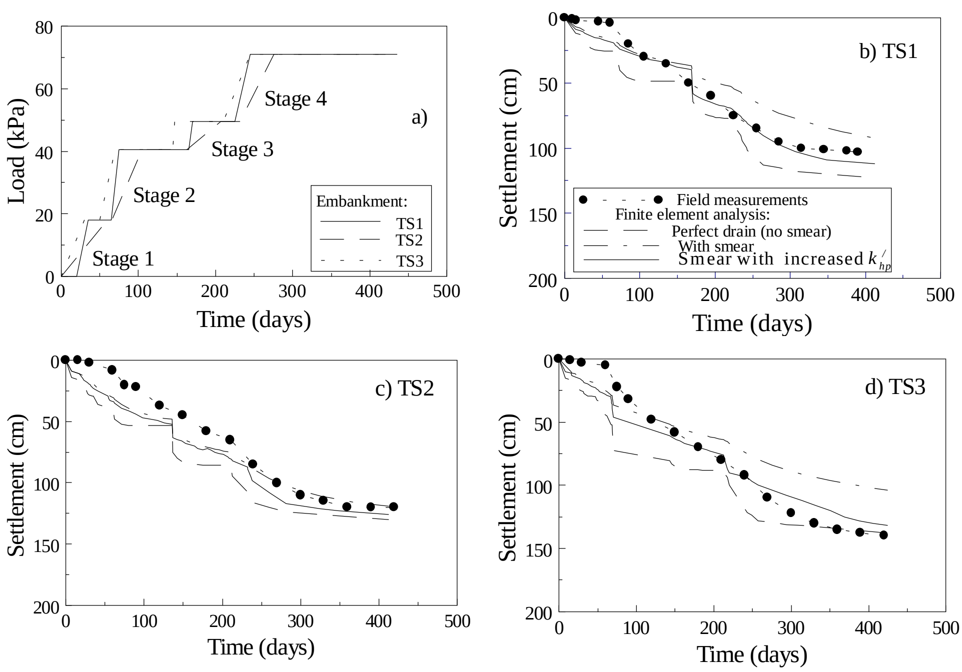

Figure 4. a) Construction loading history and (b, c and d) Surface settlement at the centre-line for embankments (b) TS1, (c) TS2 and (d) TS3, respectively, SBIA (Indraratna and Redana, 1998b).

4 CONCLUSIONS

In this study, the performance of three test embankments stabilised with vertical drains has been investigated using a plane strain finite element analysis. It has been demonstrated that the explicit modelling of the effect of smear in the vicinity of the drains improves both the settlement and excess pore water pressure predictions significantly. The modelling of perfect drain condition overestimates the settlements, because the perfect drains represent location of zero or very small undissi-

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