Behaviour of the Haarajoki test embankment constructed on soft clay improved by vertical drains

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Outline of presentation

- User defined soil model: S-CLAY1S model
- Haarajoki test embankment without vertical drains
- Modelling of vertical drains
- Haarajoki test embankment with vertical drains
- Conclusions
Structure of Natural Clays

Soil structure consists of:
- fabric (anisotropy)
- interparticle bonding (sensitivity)

Due to plastic straining gradual degradation of bonding (destructuration) and changes in fabric
Modelling Plastic Anisotropy

1. Standard elasto-plastic framework (kinematic or translational hardening laws)
   - Note: You cannot use invariants!

2. Multilaminate framework
Modelling Destructuration

Concept of an intrinsic yield surface proposed by Gens & Nova (1993)

S-CLAY1S Model

Intrinsic yield surface (Gens & Nova 1993)

\[
F = \frac{3}{2} \left[ \{\sigma_d - p'\alpha_d\}^T \{\sigma_d - p'\alpha_d\} \right] - \left[ M^2 - \frac{3}{2} \{\alpha_d\}^T \{\alpha_d\} \right] [p'_m - p'] p' = 0
\]

\[
p'_m = (1 + x)p'_m
\]
Definitions:

Deviatoric stress vector

\[ \sigma_d = \begin{bmatrix} \sigma'_x - p' \\ \sigma'_y - p' \\ \sigma'_z - p' \\ \sqrt{2}\tau_{xy} \\ \sqrt{2}\tau_{yz} \\ \sqrt{2}\tau_{zx} \end{bmatrix} \]

\[ p' = \frac{\sigma'_x + \sigma'_y + \sigma'_z}{3} \]

Deviatoric fabric tensor (in vector form)

\[ \alpha_d = \begin{bmatrix} \alpha_x - 1 \\ \alpha_y - 1 \\ \alpha_z - 1 \\ \sqrt{2}\alpha_{xy} \\ \sqrt{2}\alpha_{yz} \\ \sqrt{2}\alpha_{zx} \end{bmatrix} \]

\[ \frac{\alpha_x + \alpha_y + \alpha_z}{3} = 1 \]
Hardening laws:

1) Size of the intrinsic yield surface

\[ dp'_m = \frac{vp'_m d\varepsilon^p_v}{\lambda_i - \kappa} \]

2) Rotation of the yield surface

\[ d\alpha_d = \mu \left[ \left( \frac{3\eta}{4} - \alpha_d \right) d\varepsilon^p_v + \frac{\eta}{3} - \alpha_d \right] d\varepsilon^p_d \]

3) Degradation of bonding

\[ dx = -ax \left[ |d\varepsilon^p_v| + b(d\varepsilon^p_d) \right] \]
S-CLAY1 and MCC

By setting $x$ to zero and using an oedometric value (1D) for compressibility $\lambda$:
S-CLAY1 model (anisotropy only)

By setting, in addition, $\alpha$ and $\mu$ to zero:
MCC model (isotropy only)
## Additional State Variables and Soil Constants

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
<th>Method</th>
</tr>
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<tbody>
<tr>
<td>$\alpha_0$</td>
<td>Initial inclination of the yield curve</td>
<td>Estimated via $\phi'$</td>
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<tr>
<td>$\beta$</td>
<td>Proportion constant</td>
<td>Estimated via $\phi'$</td>
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<tr>
<td>$\mu$</td>
<td>Rate of rotation</td>
<td>$\approx (10...20)/\lambda_{K_0}$</td>
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<tr>
<td>$x_0$</td>
<td>Initial amount of bonding</td>
<td>$\approx S_t -1$</td>
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<td>$\lambda_i$</td>
<td>Slope of intrinsic compression line</td>
<td>Oedometer test on reconstituted soil</td>
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<tr>
<td>$b$</td>
<td>Proportion constant</td>
<td>For most clays 0.2-0.3</td>
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<tr>
<td>$a$</td>
<td>Rate of destructuration</td>
<td>Typically 8-11</td>
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</table>
The analysis of embankment on soft soil

During the construction of embankments (undrained behaviour)
- an increase in pore pressure.
- the effective stress remains low

After construction (drained behaviour)
- the excess pore pressures will dissipate
- The consolidation settlements will start due to drained behaviour.
- The consolidation takes a long time to complete because of the very low permeability,
- The effective stress will increase and soil can obtain the necessary shear strength to continue the consolidation.

\[ \sigma = \sigma' - \Delta u \]

\( t_c \): construction time
Vertical drains

• to reduce the length of the drainage paths
• to shorten consolidation time
• to increase shear strength
Haarajoki Test Embankment

- Finnish National Road Administration organised an international competition to predict the behaviour of the Haarajoki Test Embankment.

- The embankment which is used as a noise barrier was constructed in 1997 after all participants of the competition submitted the results.

- Laboratory investigations have been carried out by Road Administration and the laboratory of Soil Mechanics and Foundation Engineering at the Helsinki University of Technology

- The embankment was monitored with settlement plates, piezometers, inclinometers, extensometers and total stress gauges.
Longitudinal section:

- 3m above Dry crust
- 20m below Dry crust
- 2-3m Silt
- 2-3m Till
- 100m distance between 35840 and 35880
- Vertical drains c/c 1m
Construction Schedule

Total construction time: 14 days
each layer: 2 days construction
1 day for consolidation.

Instrumentation installation
Measured settlements (1997-2002)

Without PVD

With PVD

PVD: Prefabricated vertical drain
Initial conditions

\[ \sigma'_v : \text{Pre-consolidation pressure} \]

\[ \sigma'_v0 : \text{Initial vertical stress} \]
# Soil Parameters

## Initial State Parameters

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<tr>
<th>Name</th>
<th>depth [m]</th>
<th>$e_0$</th>
<th>POP $[kN/m^2]$</th>
<th>$\alpha$</th>
<th>$x_0$</th>
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<td>3b</td>
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<td>18-22.2m</td>
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## Additional Soil Constants

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<tr>
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<th>$\beta$</th>
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<th>$\lambda_i$</th>
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<th>$b$</th>
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## Conventional Soil Constants

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<tr>
<th>Name</th>
<th>$\gamma$ $[kN/m^3]$</th>
<th>$\kappa$</th>
<th>$\nu'$</th>
<th>$\lambda$</th>
<th>$M$</th>
<th>$k_x$ $[m/d]$</th>
<th>$k_y$ $[m/d]$</th>
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<td>1.33</td>
<td>1.15</td>
<td>1.56E-04</td>
<td>1.30E-04</td>
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<td>14</td>
<td>0.033</td>
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<td>1.33</td>
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<td>1.56E-04</td>
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<td>2.59E-04</td>
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<td>17</td>
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<td>0.10</td>
<td>1.50</td>
<td>8.00E-03</td>
<td>8.00E-03</td>
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</table>
Plaxis simulations

- Two dimensional (2D) finite element code PLAXIS V8.
- The problem was analysed in a plane-strain conditions.
- The geometry of the test embankment was assumed symmetric.
- The real construction schedule has been simulated in calculation.
- The initial stresses were calculated by using $K_0 = (1 - \sin\phi') \cdot OCR^{\sin\phi'}$ (Mayne & Kulhawy 1982)
Soil permeability

• The changes of soil permeability during consolidation were taken into account according to the relationship (Taylor 1948)

\[
\log\left( \frac{k}{k_0} \right) = \frac{\Delta e}{c_k}
\]

\(\Delta e\): change in void ratio,
\(k\): the soil permeability in the calculation step
\(k_0\): the initial input value of the permeability
\(c_k\): the permeability change index

• It was assumed that \(c_k=0.5e_0\) in the analyses (Tavenas et al. 1983).
The analysis of area without vertical drains

(Cross-section 35840)
Time-settlement curve

Center Line (35840)

Time (days)

Settlement (mm)

- Observed
- MCC
- S-CLAY1
- S-CLAY1S
Pore pressures

Cross Section: 35840

Active Pore pressure (kPa)

- Observered
- S-CLAY1S
- S-CLAY1
- MCC

Time (days)

15 m
10 m
7 m
Surface settlements

Cross section (35840)

Settlement (mm)

Time (days)

-500
-400
-300
-200
-100
0
100
200
300
400
500

5.9.97
24.9.02
S-CLAY1S 5.9.97
S-CLAY1S 24.9.02
The analysis of improved area by vertical drains

(Cross-section 35880)
Drain properties in Haarajoki Test Embankment

- Vertical drains are 15m long
- They are installed in a square grid with 1m spacing
- Equivalent radius of drains is 0.034 m
- The radius of axisymmetric unit cell = 0.565 m
- Discharge capacity, $q_w = 157 \text{ m}^3/\text{year}$
Background theory of vertical drains

- **Barron (1948)** developed the solution of the horizontal consolidation under ideal conditions using an axisymmetric unit cell model.

\[
U_h = 1 - \exp\left(-\frac{8T_n}{\mu}\right)
\]

- There are two important factors in the analysis of vertical drains.
  - smear effect
  - well resistance.

Radial drainage of a vertical drain
Smear effect

- Because of the installation of the drains soil around the drain (smear zone) is disturbed.
  - The smear zone depends on
    - the method of drain installation
    - the size and shape of mandrel
    - the soil structure
  - Problems for the analysis:
    - What is the diameter of the smear zone ($d_s$)?
    - What is the permeability of soil in the smear zone ($k_s$)?

- Recent investigations on a laboratory scale
  - $d_s/d_m = 4-5$ (Indraratna and Redana, 1998)
  - $k_h/k_s = 2$ (Bergado et al. 1993)

- $5 < k_h/k_s < 20$ for field full-scale test (Bergado et al. 1993)
Well resistance

- The limited discharge capacity of drains can cause a serious delay in the consolidation process.

- Modern drains have a high enough discharge capacity ($q_w > 150 \text{ m}^3/\text{year}$).

- The effect of well resistance can be ignored in the design.
Hansbo (1981) modified the equation of Barron (1948) to include the effect of smear and well resistance

\[ U_h = 1 - \exp\left(-\frac{8T_n}{\mu}\right) \]

\[ \mu = \ln(n) - \frac{3}{4} \quad (\text{perfect drain}) \]

\[ \mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k}{k_s}\right)\ln(s) - \frac{3}{4} \quad (\text{smear effect only}) \]

\[ \mu = \ln\left(\frac{n}{s}\right) + \left(\frac{k}{k_s}\right)\ln(s) - \frac{3}{4} + \pi(2lz-z^2)\frac{k_h}{q_w} \quad (\text{smear effect and well resistance}) \]

\[ n = \frac{R}{r_w} \quad \text{and} \quad s = \frac{r_s}{r_w} \]
Finite element analysis of vertical drains

2-D FE analysis of embankments is conducted on the plane strain conditions whereas vertical drains are axisymmetric.

- The Vertical drain system may be converted into equivalent plane strain model by manipulating the drain spacing \((B)\) and/or soil permeability \((k_h)\).

- Hird et al. (1992) developed an equivalent plane strain analysis considering a unit cell of the vertical drain based on Hansbo's theory.
The degree of consolidation in plane strain condition can be expressed as follows:

\[ U_{hp} = 1 - \frac{\bar{u}}{u_0} = 1 - \exp(-8 \frac{T_{hp}}{\mu_p}) \]

\[ U_{hax} = U_{hpl} \]

\[ \frac{T_{hpl}}{\mu_{pl}} = \frac{T_{hax}}{\mu_{ax}} \quad \text{or} \quad \frac{c_{hpl}}{B^2 \mu_{pl}} = \frac{c_{hax}}{R^2 \mu_{ax}} \]

\( u \): the pore pressure at time \( t \),
\( u_0 \): the initial pore pressure
\( T_{hp} \): the time factor in plane strain
\( \mu_{pl} \): the parameter that includes the effect of smear and well resistance

Average degree of consolidation for both axisymmetric and equivalent plane strain conditions are made equal at each time step and at a given stress level.
Geometry matching \((k_{ax} = k_{pl})\)

\[
\frac{B}{R} = \left\{ \frac{3}{2} \left[ \ln(n) + \left( \frac{k_h}{k_s} \right) \ln(s) - \left( \frac{3}{4} \right) \right] \right\}^{\frac{1}{2}}
\]

Permeability matching \((B = R)\)

\[
k_{pl} = \frac{2k_{ax}}{3 \left[ \ln\left( \frac{n}{s} \right) + \left( \frac{k_{ax}}{k_s} \right) \ln(s) - \frac{3}{4} \right]}
\]

Combined matching

\[
k_{pl} = \frac{2B^2 k_{ax}}{3R^2 \left[ \ln\left( \frac{n}{s} \right) + \left( \frac{k_{ax}}{k_s} \right) \ln(s) - \frac{3}{4} \right]}
\]
## Matching procedures for plane strain model

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<th>Permeability matching</th>
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<td>k_{pl}/k_{ax}</td>
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Unit cell analysis

(a) Axisymmetric Radial Flow

(b) Plain strain
Unit Cell Analysis

Perfect drain

Smear effect
\( \frac{k_h}{k_s} = 20 \quad \frac{d_s}{d_m} = 5 \)
**Unit Cell Analysis (smear effect)**

### k/kₚ effect ($d_s/d_m = 5$)

- $k = k_h$ horizontal permeability in the undisturbed area

### $d_s/d_m$ effect ($k/k_s = 10$)

- $d_m$ mandrel diameter
The simulation of Haarajoki Test embankment

Time – Settlement curves

- Observed
- MCC
- S-CLAY1
- S-CLAY1S
Comparison of the settlement behaviour of embankment

PVD: Prefabricated vertical drain
Excess pore pressures
Pore pressure distribution

After construction
6\textsuperscript{th} layer
Vertical displacements

After construction
6th layer
Lateral displacements

After construction
6th layer
Conclusions

- S-CLAY1S simulations of Haarajoki test embankment on natural soil are in good agreement with observed settlements.

- It is important to account for anisotropy, bonding and degradation of bonds in boundary value problems where loading is dominant.

- Vertical drains can be modelled simply and successfully in plane strain simulations using a proper matching procedure.

- Smear effect must be taken into account in the analysis.
Future Work

- full 3D embankment simulations
- enhance S-CLAY1S model to consider creep effects
Thank you for your attention!

For information email to 
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or 
harald.krenn@strath.ac.uk

You can also visit the AMGISS website for further info:

http://www.civil.gla.ac.uk/amgiss