Geotechnical Workshop at Griffith University Gold Coast Campus, 29 Sept – 3 Oct 2008

Part II
Preloading using Prefabricated Vertical Drains (PVDs)

2-1. Introduction
2-2. Design and analysis method
2-3. Installation
2-4. Quality control and practical considerations
2-5. Case studies

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Section 2-1
Introduction

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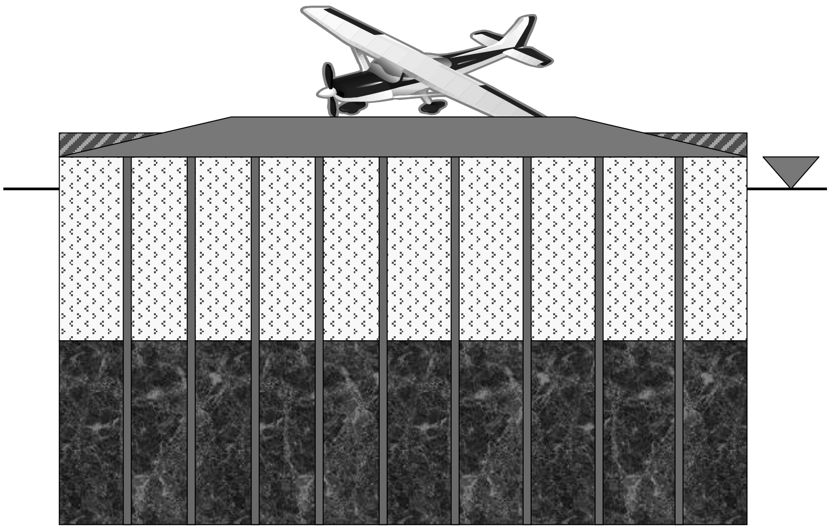
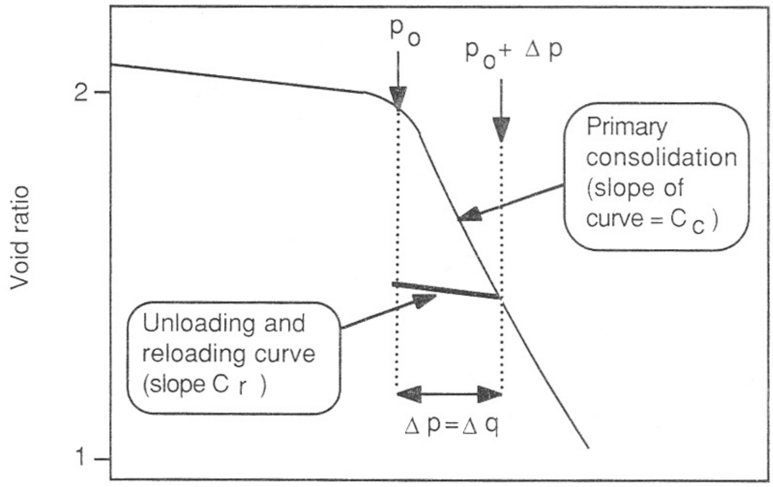
Offshore Land Reclamation Process



Purpose of Preloading

- To increase the bearing capacity and reduce the compressibility of weak ground by forcing loose cohesionless soils to densify or clayey, silty soil to consolidate.
- To reduce secondary compression under working load.
- Achieved by placing a temporary surcharge prior to the construction of the planned structure.
- The surcharge can be applied using fill weight or vacuum pressure.

Mechanism of preloading



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Preloading

- Advantages:
  - Inexpensive if large area is improved.
- Disadvantages:
  - Time consuming. Need a few months if not a few years. Therefore, prefabricated vertical drains (PVDs) are normally used to cut down the drainage path and thus accelerate the consolidation process.

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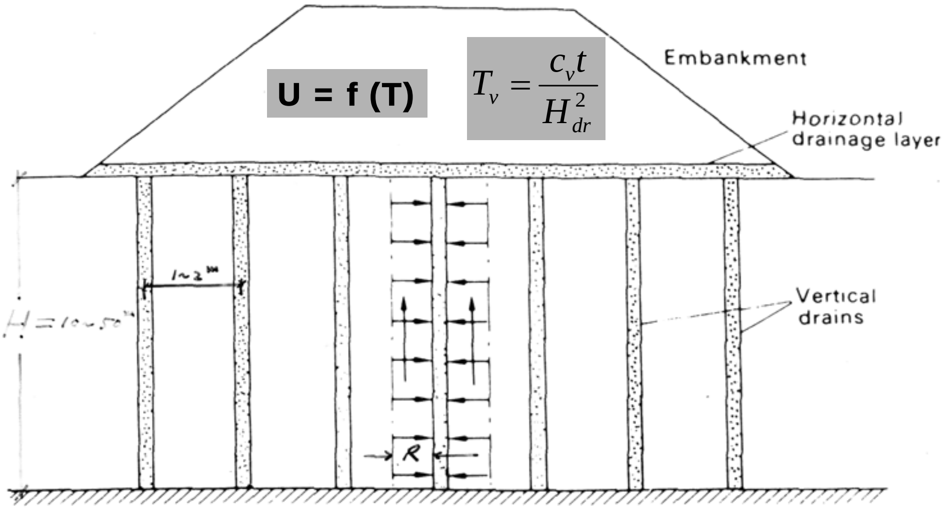
Why PVDs

U = f (T)

$$T_v = \frac{c_v t}{H_{dr}^2}$$



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Vertical drains

- Vertical drains are used to accelerate primary consolidation of cohesive soil by cutting down the drainage path.
- Types of vertical drains
  - *Cylindrical Sand Drains*: Boreholes filled with sand without or with a sock, 65 – 450 mm in diameter.
  - *Prefabricated Vertical Drains (PVDs)*: Band drains with a core and filter sleeve made of paper, PVC, PE (polyethylene), PP (polypylene), and PES (polyester).

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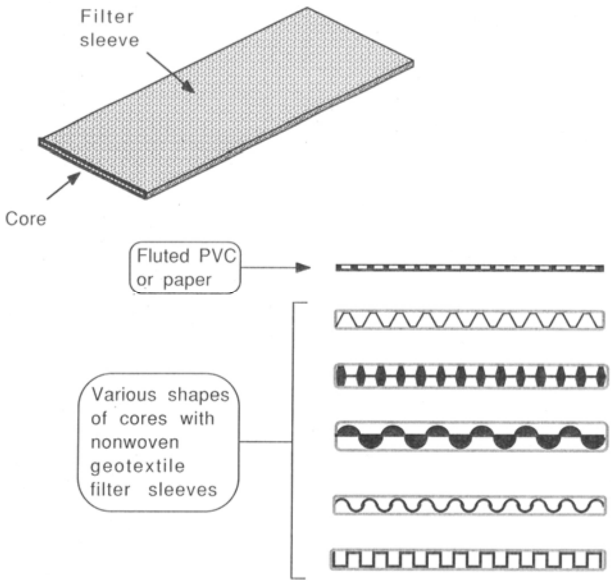

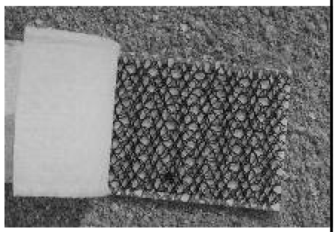

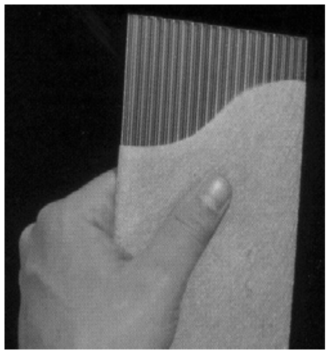

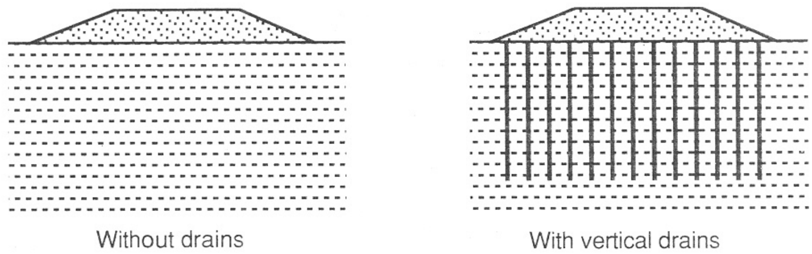

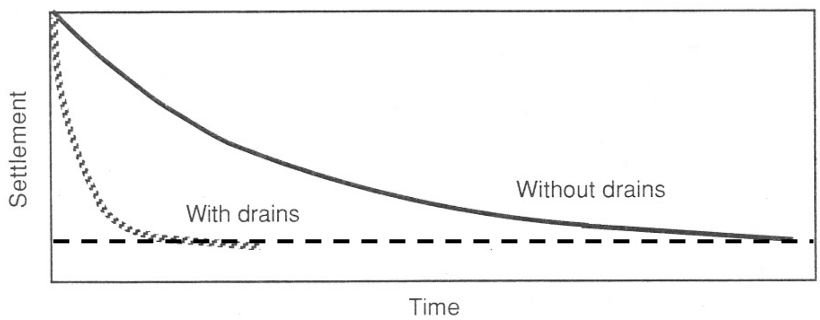

Vertical drains accelerate settlement, but do not reduce final settlement

Design problems

- Calculate the ultimate settlement and settlement at a given time;
- Estimate the rate of consolidation, i.e., how long will it take to achieve a given degree of consolidation;
- Design for vertical drains, e.g., the drain spacing required to achieve a given degree of consolidation.

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Section 2-2
Design and Analysis methods

2.2.1 Settlement calculation

2.2.2 Rate of settlement calculation

1). Terzaghi's (vertical) consolidation theory

2). Radial consolidation theory and vertical drain design

3). Combined vertical and radial flow

2.2.3 Design for vertical drain with smear zone

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2.2.1 Settlement Calculation

Ground settlement consists of 3 components:

$$\delta = \delta_i + \delta_c + \delta_s$$

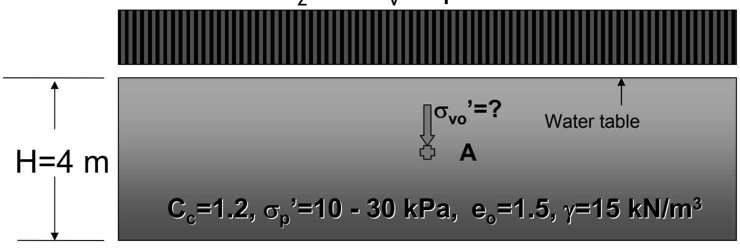
Initial settlement (δ_i), Consolidation settlement (δ_c), Secondary compression (δ_s)

For soft clay, consolidation settlement dominates.
With preloading, secondary compression is small.
The compressibility of sand fill is normally ignored.

Settlement calculation is assumed to be a 1D problem.
This is reasonable for reclamation problems.

Consolidation Settlement Calculation

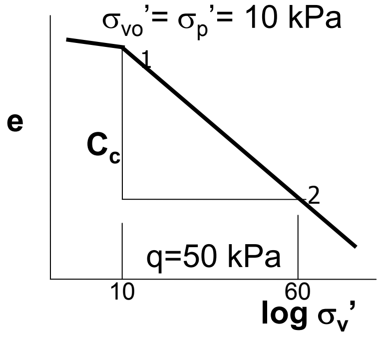
$\Delta\sigma_z = \Delta\sigma_v = q = 50$ kPa

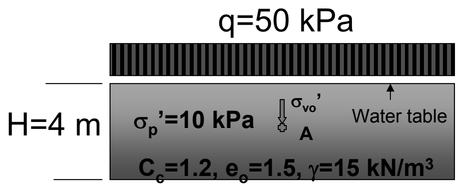


The effective overburden stress at the middle of the clay layer:

$$\sigma_{vo}' = 2*15 - 2*10 = 10 \text{ kPa.}$$

Settlement Calculation – σ_p'= 10 kPa





$$\Delta H = \frac{1.2*4}{1+1.5} \log \frac{10+50}{10}$$

$$= 1.48 \text{ m}$$

$C_c = \Delta e/[\log(\sigma_{vo}'+q)-\log \sigma_{vo}']$

$$\Delta H = \varepsilon_v H = \frac{\Delta e H}{1+e_o} = \frac{C_c H}{1+e_o}[\log(\sigma_{vo}'+q)-\log \sigma_{vo}'] \quad (1)$$

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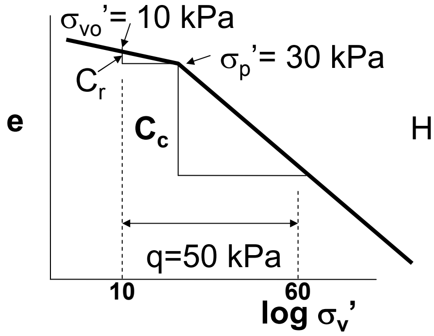
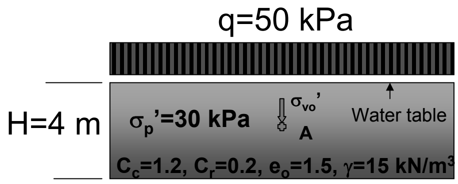
Settlement Calculation – σ_p'= 30 kPa





$$\Delta H = \frac{0.2*4}{1+1.5} \log \frac{30}{10} + \frac{1.2*4}{1+1.5} \log \frac{10+50}{30}$$

$$= 0.12 + 0.56 = 0.68 \text{ m}$$

$C_c = \Delta e/[\log(\sigma_{vo}'+q)-\log \sigma_p']$

$$\Delta H = \frac{C_r H}{1+e_o}[\log\sigma_p'-\log \sigma_{vo}'] + \frac{C_c H}{1+e_o}[\log(\sigma_{vo}'+q)-\log \sigma_p'] \quad (3)$$

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Settlement can be induced by ground water variations

Settlement due to NEL MRT construction in front of People's Park Complex

Fill surcharge can also be affected by the water table

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Remarks on ultimate consolidation settlement calculation

- It is important to determine σ_p' reliably, as σ_p' affects settlement calculation considerably.
- For multiple soft clay layers or a single soft clay layer but with large variations in σ_p' or C_c, subdivision is required for more accurate results.
- For thick clay layer, subdivision may also be required, as the effective stress changes is too large.
- Sometimes, we need to calculate the settlement at a given time or degree of consolidation, U_{avg}. You can either use $s(t) = U_{avg}S_{ult}$, or use $\sigma_v'(t)$.

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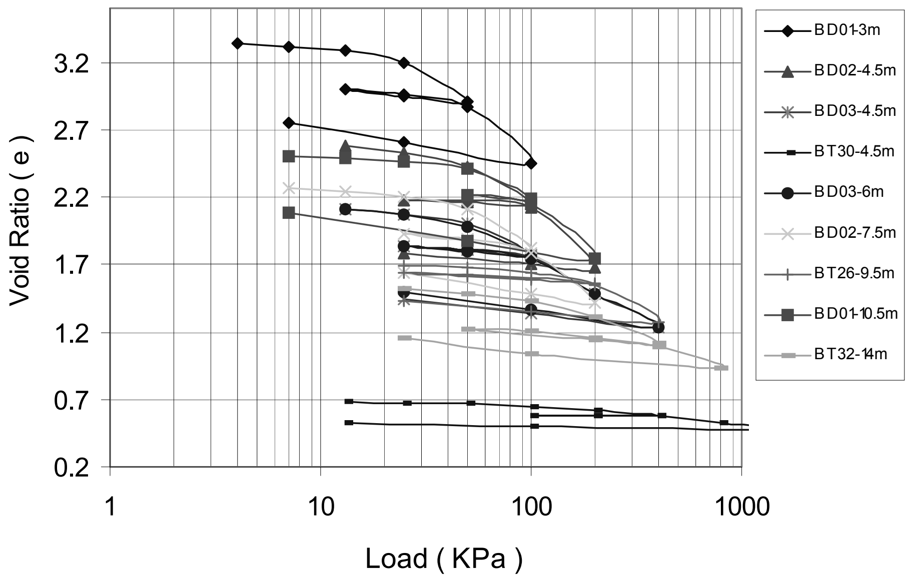
e ~ log σ_v' Curves for KFC at Different Depths



2.2.2 Rate of Consolidation

Terzaghi's 1D Consolidation Theory

$$\frac{\partial u_e}{\partial t} = c_v \frac{\partial^2 u_e}{\partial z^2} \qquad (1\text{-}1)$$

Excess pore water pressure (pwp) — u_e

Time — t

Vertical distance below the ground surface — z

$$c_v = \left(\frac{2.30\sigma_z' k}{\gamma_w}\right)\left(\frac{1+e}{C_c}\right)$$

Coefficient of consolidation — c_v

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What's For?

Excess pore water pressure (pwp)

$$u_e = \Delta\sigma_z f\left(T_v, \frac{z_{dr}}{H_{dr}}\right) = \sum \frac{2u_0}{M}(\sin M \frac{z}{H_d})e^{-M^2T_v} \quad (1.2)$$

Fig. 1.1

Change in vertical effective stress

$$M = \frac{\pi}{2}(2m+1)$$

For uniform pwp distribution

Vertical distance from point to nearest drainage boundary

Time factor: $T_v = \frac{c_v t}{H_{dr}^2}$

Drainage path:
- H_{dr} = H, single drainage
- H_{dr} = H / 2, double drainage

Solution

$$u_e = \Delta\sigma_z f\left(T_v, \frac{z_{dr}}{H_{dr}}\right)$$

$$T_v = \frac{c_v t}{H_{dr}^2}$$

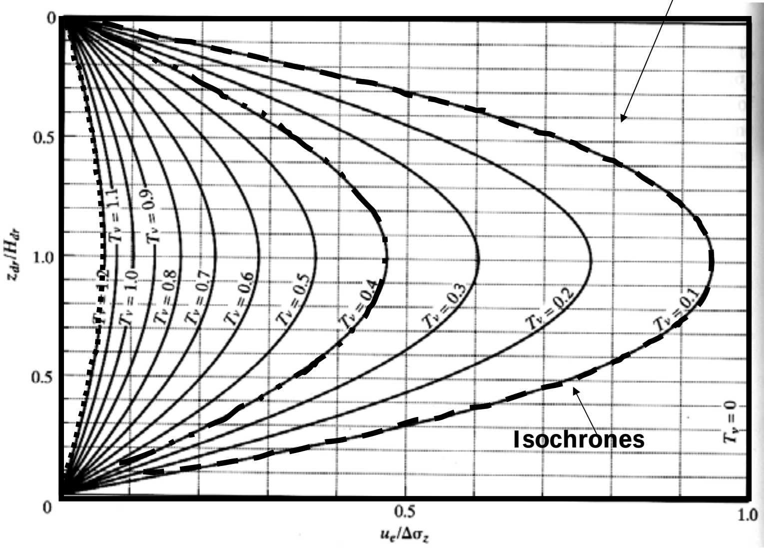

Fig 1.1 $u_e/\Delta\sigma_z$ for various values with double drainage.

(For the single drainage case, use only the upper half of this diagram.)

Average Degree of Consolidation

Consolidation settlement

$$\overline{U_v} = \frac{S_c(t)}{(S_c)_{ult}} 100\% = \frac{(4T_v/\pi)^{0.5}}{[1+(4T_v/\pi)^{2.8}]^{0.179}} \quad (1.3)$$

Average degree of consolidation

Ultimate consolidation settlement

$$T_v = \frac{c_v t}{H_{dr}^2}$$

$$S_c(t) = \overline{U}(S_c)_{ult} \quad (1.3a)$$

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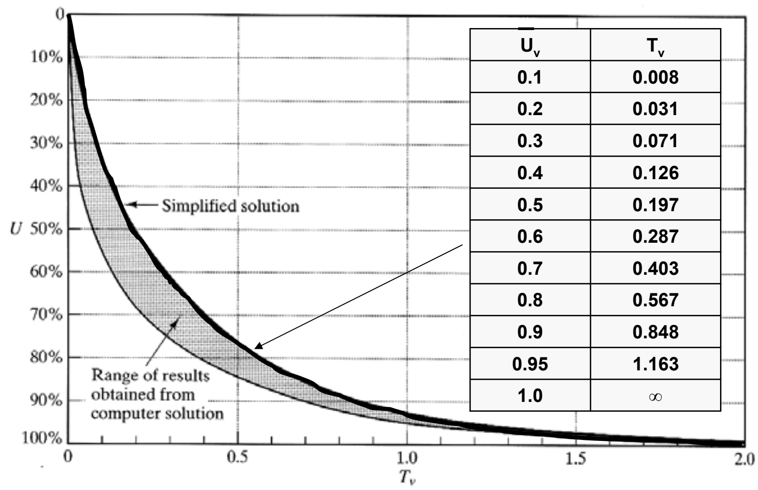
$\overline{U}_v$ versus T_v Relationship



$\overline{U}_v$	T_v
0.1	0.008
0.2	0.031
0.3	0.071
0.4	0.126
0.5	0.197
0.6	0.287
0.7	0.403
0.8	0.567
0.9	0.848
0.95	1.163
1.0	∞

Figure 12.10 The solid line is the U vs. T_v function for the simplified analysis of one-dimensional consolidation. The shaded area represents the range of values obtained from the more precise computer solution described earlier.

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Radial Consolidation Theory

$$c_h\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\frac{\partial u}{\partial t} \qquad c_h=\frac{k_h}{m_v\gamma_w} \qquad (1.4)$$

c_h is the coefficient of consolidation in the horizontal direction.

For combined vertical and radial flow

$$c_v\,\frac{\partial^2 u}{\partial z^2}+c_h\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\frac{\partial u}{\partial t} \qquad (1.5)$$

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Consolidation due to combined vertical and radial flow

Carillo's equation

In practice, the excess pore pressure ratios u_e/u_o are computed separately based on vertical flow (Eq. 1.1) and radial flow (Eq. 1.4) alone, and then combined using Carillo's equation:

$$(1-U_{vh})=(1-U_z)\times(1-U_h)$$ **at a point (1.6)**

$$(1-\overline{U}_{vh})=(1-\overline{U}_v)\times(1-\overline{U}_h)$$ **average value (1.7)**

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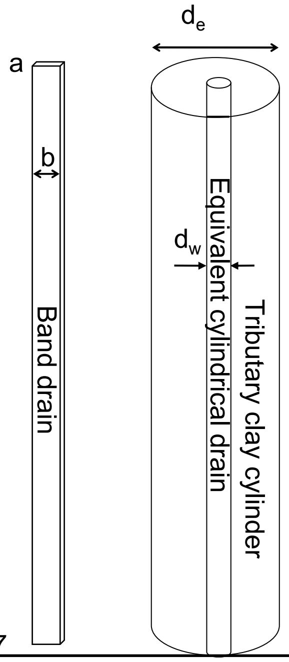
Barron's theory for pure radial drainage



$$d_w = 2(a+b)/\pi$$ Eq. (1.8)

$$\overline{U}_h = 1 - \exp\left[\frac{-8T_h}{F(n)}\right]$$

$$F(n) = \frac{n^2}{(n^2-1)}\ln(n) - \frac{(3n^2-1)}{4n^2} \approx \ln(n) - 0.75$$ Eq. (1.9)

$$T_h = \frac{c_h t}{d_e^2} \qquad n = \frac{d_e}{d_w}$$ Eq. (1.10)

Fig 1.2 Barron's equal-strain solution for a unit cell

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An Alternative Formula

(Terzaghi, Peck & Mesri (1996))

$$\overline{U}_h = 1 - \exp\left[\frac{-2T_r}{F(n)}\right]$$

$$F(n) \approx \ln(n) - 0.75$$

Eq. (1.11)

$$T_r = \frac{c_h t}{r_e^2} \qquad r_e = \frac{d_e}{2}$$

Equations 1.9 and 1.11 are <u>identical.</u>

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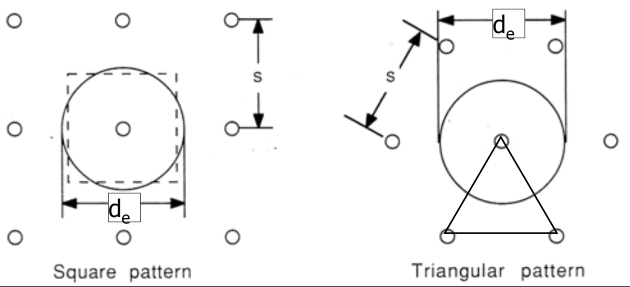
Calculation of d_e

The diameter of the equivalent clay cylinder tributary to a vertical drain is based on equivalent cross-sectional area. If the vertical drains are installed in a square grid pattern, the equivalent drainage diameter is obtained as follows:

$$\textit{Square grid:} \quad spacing^2 = \pi \frac{d_e^2}{4}, \text{ hence } d_e = 1.128 \times spacing \qquad (1.12)$$

If the vertical drains are spaced in a triangular grid pattern, the equivalent drainage diameter is:

$$\textit{Triangular grid:} \quad spacing^2 \times \sin 60° = \pi \frac{d_e^2}{4}, \text{ hence } d_e = 1.05 \times spacing \qquad (1.13)$$



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Equal-strain consolidation with no smear

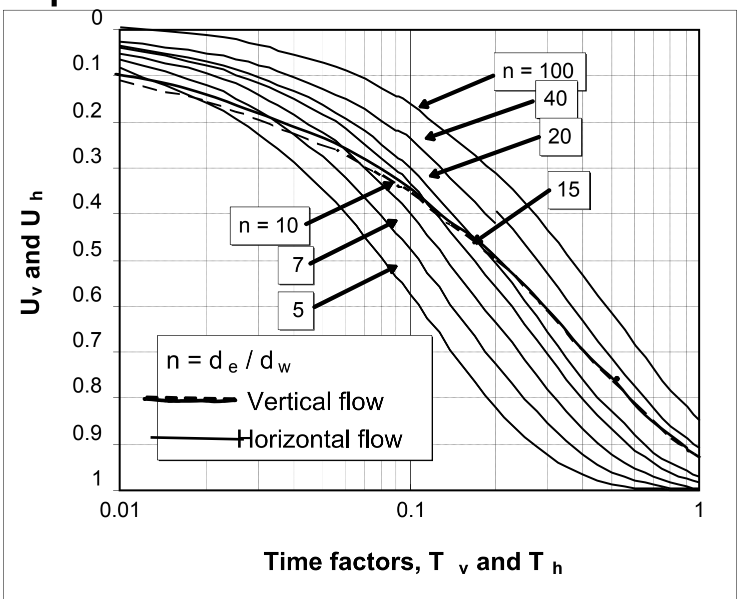

Fig 1.3 Solutions to Eq. (1.3) & Eq. (1.9)

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Design charts for combined vertical and radial

$$\left(1-\overline{U}_{vh}\right)=\left(1-\overline{U}_v\right)\times\left(1-\overline{U}_h\right) \qquad \overline{U}_h=1-\exp\left[\frac{-8T_h}{F(n)}\right]$$

$$\overline{U}_{vh}=1-\left(1-\overline{U}_v\right)\times\exp\left[-8T_h/F(n)\right]$$

$$=1-\left(1-\overline{U}_v\right)\times\exp\left[-8\frac{T_h}{T_v}T_v\Big/F(n)\right]$$

$$=1-\left(1-\overline{U}_v\right)\times\exp\left[-\lambda T_v\right] \qquad \textbf{(1.14)}$$

where $\lambda=\dfrac{8}{F(n)}\dfrac{T_h}{T_v}=\dfrac{8}{F(n)}\dfrac{c_h/d_e{}^2}{c_v/H_d{}^2}$ **(1.14a)**

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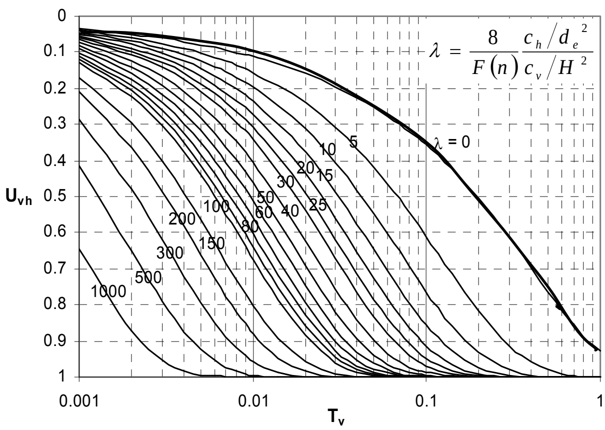

Fig 1.4 Solution to combined drainage

After Bo et al. (2003)

$T_v=c_v t/H^2$

$F(n)\approx\ln(n)-0.75$

$n=d_e/d_w$

c_v = coefficient of consolidation (vertical flow)

c_h = coefficient of consolidation (horizontal flow)

H = Maximum length of vertical drainage path

(Note: λ = 0 if no horizontal drainage)

d_e = 1.13s for sqaure grid

1.05s for triangular grid

s = drain spacing

d_w = drain diameter

Note: λ = 0 for cases with no vertical drains or horizontal drainage layer.

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Example -1

EXAMPLE:

Given: Saturated clay layer 8 m thick, lower boundary impermeable, 70 mm diameter prefabricated vertical drains at 2 m centres in a square pattern, c_v = 2.0 m^2/year , c_h = 3.0 m^2/year.

To Find: Time required for 90% consolidation of the clay layer as a result of an extensive fill.

$$\bar{U}_h = 1 - \exp\left[\frac{-2T_r}{F(n)}\right]$$

$$F(n) \approx \ln(n) - 0.75$$

$$\lambda = \frac{8}{F(n)}\frac{T_h}{T_v} = \frac{8}{F(n)}\frac{c_h/d_e^2}{c_v/H_d^2}$$

Solution: $d_e = 1.13 \times 2$ m = 2.26 m

n = 2.26 m / 0.07 m = 32.3

$F(n) \approx \ln(32.3) - 0.75 = 2.73$

$\lambda = (8 / 2.73) \times (3 / 2.26^2) / (2 / 8^2) = 55$ (using the above λ eqn.)

Enter chart with λ = 55 and U_{vh} = 90%,

$n = d_e / d_w$

get T_v = 0.038

then, required $t = T_v H^2 / c_v$ = 1.2 years.

Example -1 (Cont'd)

In Example 1, if H = 20 m,

$\lambda = (8/2.73)x(3/2.26^2)/(2/20^2) = 344$

$T_v = 0.006$, then $t = T_v H^2/c_v = 0.006x20^2/2 = 1.2$ yr

Therefore, radial drainage controls when the clay layer is thick.

An alternative method is to calculate U_v and U_h using the equations (1.3), (1.8 – 1.10) or the chart, Fig. 1.3. However, the solution has to be done by trial and error. For example, you can assume t =1 yr, then calculate U_v and U_h and then U_{vh}. If U_{vh} is less than 90%, increase t and calculate again. This is illustrated using the next example.

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Example -2

As in Example 1, given c_v = 2.0 m^2/yr, c_h = 3.0 m^2/yr, H = 8 m, PVD 104 x 5 mm at a spacing of 2 m in square pattern. Calculate the degree of consolidation achieved in 1 yr.

$T_v = c_v t/H_{dr}^2 = 2x1/4^2 = 0.125$
$U_v = \text{sqrt}(4T_v/\pi) = \text{sqrt}(4x0.125/3.14) = 0.4$

$d_w = 2(a + b)/\pi = 2(0.104+0.005)/3.14 = 0.07$ m
$d_e = 1.13s = 1.13x\ 2 = 2.26$ m, $n = 2.256/0.07 = 32.3$
$F(n) = \ln(n) - 0.75 = \ln(32.3) - 0.75 = 2.73$
$T_h = c_h t/d_e^2 = 3x1/2.26^2 = 0.587$
$U_h = 1 - \exp[-8T_h/F(n)] = 1 - \exp[-8x0.587/2.73] = 0.82$
$U_{vh} = U_v + U_h - U_v U_h = 0.4 + 0.82 - 0.4x0.82 = 0.89$ or 89%

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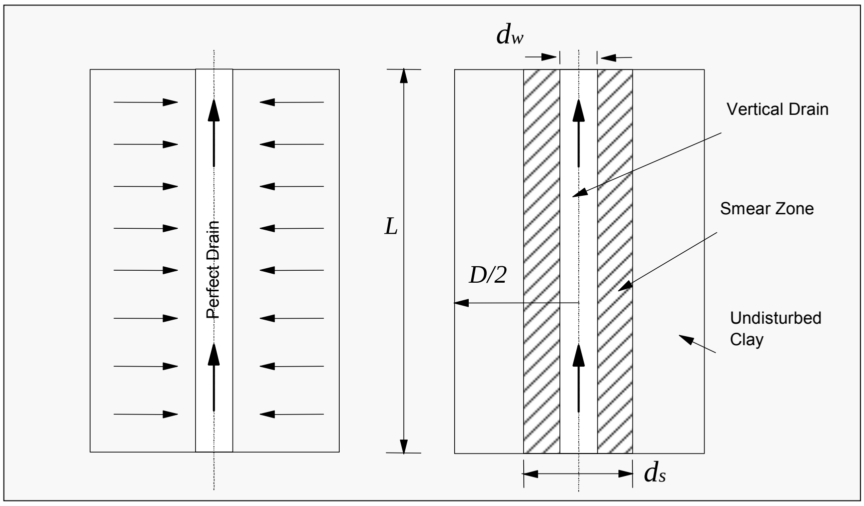
Design for Vertical Drain with Smear Zone



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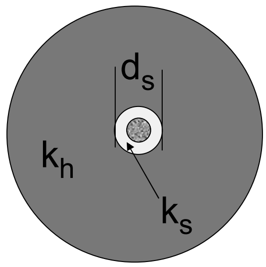
Smear effect

An annulus of smeared clay around the drain. Within this annulus of diameter d_s, the remolded soil has a coefficient of permeability k_s which is lower than the k_h of the undisturbed clay.

$$F_s(n) = \ln\left(\frac{n}{s}\right) - 0.75 + \left(\frac{k_h}{k_s}\right)\ln(s)$$

(Eq. 1.15)



where s = smear zone ratio d_s/d_w

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The new boundary condition between the undisturbed zone and the smeared annulus affects the above solution for $\overline{U}_h$ by changing the factor F(n) which becomes:

$$F_s(n) \approx \ln\left(\frac{n}{s}\right) - 0.75 + \left(\frac{k_h}{k_s}\right)\ln(s) \qquad (1.16)$$

or, equivalently, $$F_s(n) \approx \ln(n) - 0.75 + \ln(s)\left(\frac{k_h}{k_s} - 1\right) \qquad (1.17)$$

so that Eq. 2.16 becomes $$\overline{U}_h = 1 - \exp\left[\frac{-8T_h}{F_s(n)}\right] \qquad (1.18)$$

where s = smear zone ratio d_s/d_w (1.18a)

The two additional parameters, s and k_h/k_s, are difficult to estimate.

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Example -3

As in Example 1, given c_v = 2.0 m^2/yr, c_h = 3.0 m^2/yr, H = 8 m, PVD 104 x 5 mm at a spacing of 2 m in square pattern. Calculate the degree of consolidation achieved in 1 yr. Assume that permeability in the smear zone is 1/2 of the undisturbed clay and the smear zone diameter is 2 time of the drain diameter.

$T_v = c_v t/H_{dr}^2 = 2x1/4^2 = 0.125$,
U_v = sqrt ($4T_v/\pi$) = sqrt (4x0.125/3.14) = 0.4

$d_w = 2(a + b)/\pi$ = 2(0.104+0.005)/3.14 = 0.07 m
d_e = 1.128s = 1.128x 2 = 2.256 m, n = 2.256/0.07 = 32.3
$S = d_s/d_w = 2$, $k_h/k_v = 2$

$F(n) = \ln\left(\frac{n}{s}\right) - 0.75 + \left(\frac{k_h}{k_s}\right)\ln(s)$ = ln(32.3/2) – 0.75 + 2ln2 = 3.4

$T_h = c_h t/d_e^2 = 3x1/2.256^2 = 0.589$
U_h = 1 – exp[$-8T_h/F(n)$] = 1 – exp[-8x0.589/3.4] = 0.75
$U_{vh} = U_v + U_h - U_v U_h$ = 0.4 + 0.75 – 0.4x0.75 = 0.85 or 85%

	A	B	C	D	E	F	G	H	I	J
3	**time**	1.000	year	**SquareGrid?**		**d_e**	**n**	**T_v**	**T_h**	**F(n)**
4	**c_v**	2	m^2/yr		Square	2.256	32.229	0.125	0.58944	3.416
5	**H_d**	4	m		(y or n)					
6	**c_h**	3	m^2/yr						**Carillo**	
7	**Spacing**	2.000	m				**U_v**	**U_h**	**U_{vh}**	
8	**d_w**	0.07	m				0.3985	0.7485	0.849	
9	**s**	2	(smear diameter ratio)							
10	**k_h/k_s**	2	(smear k ratio)							
11										
12		**Formulas:**		d_e = Spacing*IF(E4="y",1.128,1.05)						
13				$n = d_e/d_w$						
14				$T_v = c_v$*time/H_d^2						
15				$T_h = c_h$*time/d_e^2						
16				F(n) = LN(n)-0.75+LN(s)*(k_h/k_s-1)						
17				U_v = SQRT(4*T_v/PI())/(1+(4*T_v/PI())^2.8)^0.179						
18				U_h = 1-EXP(-8*T_h/F(n))						
19				$U_{vh} = U_v+U_h-U_v*U_h$						

Example -4

EXAMPLE: A wide sand fill is to be placed on a layer of soft clay. The clay is 8 m thick and underlain by impermeable hard stratum. The coefficients of consolidation of the clay are c_v = 2.0 m^2/year and c_h = 3.0 m^2/year. Assume that the placement of fill to its final thickness will occur gradually over 6 months. To accelerate consolidation of the clay layer, it is proposed to install vertical prefabricated band drains (of cross-section 100 mm × 5 mm) all the way to the bottom of the clay layer. The drains will be installed when the fill reaches half its final thickness. Determine the horizontal spacing of the vertical drains (installed in a triangular-grid layout) so as to achieve 90% of primary consolidation in 1.5 years after the commencement of fill placement. Assume that the installation of the band drains results in an annulus of smeared zone in which the coefficient of permeability (k_s) for radial flow is 1/3 that of the undisturbed clay. The smear zone ratio is s = 2. Assume negligible well resistance.

Note: $$\overline{U}_v \approx \frac{\sqrt{(4T_v/\pi)}}{\left[1+(4T_v/\pi)^{2.8}\right]^{0.179}} \quad \text{for } 0 \le \overline{U}_v \le 100\%$$

$$\overline{U}_h = 1-\exp(-8T_h/F_s(n))$$

where $$F_s(n) = \ln\left(\frac{n}{s}\right) - 0.75 + \left(\frac{k_h}{k_s}\right)\times\ln(s)$$

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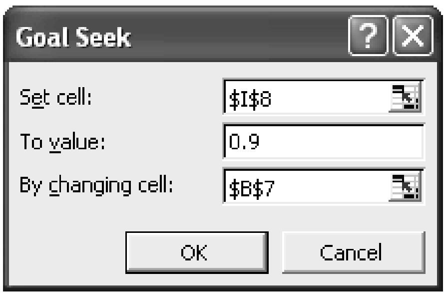
Available consolidation time = 18 months – 0.5*6 months = 1.25 years

	A	B	C	D	E	F	G	H	I	J
3	**time**	1.250	year	**SquareGrid?**		**d_e**	**n**	**T_v**	**T_h**	**F(n)**
4	**c_v**	2	m^2/yr	Triangular		1.9211	27.444	0.0391	1.01611	3.94844
5	**H_d**	8	m		(y or n)					
6	**c_h**	3	m^2/yr						**Carillo**	
7	**Spacing**	1.830	m				**U_v**	**U_h**	**U_{vh}**	
8	**d_w**	0.07	m				0.223	0.8724	0.901	
9	**s**	2	(smear diameter ratio)							
10	**k_h/k_s**	3	(smear k ratio)							



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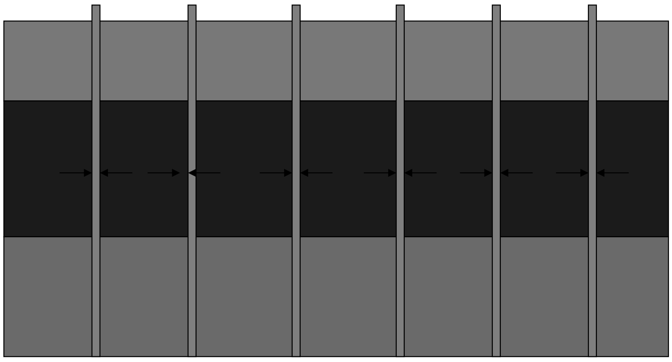
For Layered soils



The rate of consolidation is controlled by the soil layer with a smaller c_h

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Summary

(1) Vertical drainage:

For $U_v < 0.60$, $T_v = (\pi/4)U_v^2$

For $U_v > 0.60$, $T_v = -0.933\log(1-U_v) - 0.085$

Or $$\overline{U}_v \approx \frac{\sqrt{(4T_v/\pi)}}{\left[1+(4T_v/\pi)^{2.8}\right]^{0.179}} \quad \text{for } 0 \le \overline{U}_v \le 100\%$$

(2) Radial drainage:

Where: $F(n) \approx \ln(n) - 0.75$ For case of no smear and no well resistance

$$F(n) = \ln\left(\frac{n}{s}\right) - 0.75 + \left(\frac{k_h}{k_s}\right)\ln(s)$$ For smear case

$n = d_e/d_w$ $s = d_s/d_w$ $d_w = 2(a + b)/\pi$

d_e = 1.128 x spacing For square grid

d_e = 1.05 x spacing For triangular grid

d_s = outer diameter of the smear zone annulus

(3) Carillo's Equation:

$(1 - U_{vh}) = (1 - U_v) \times (1 - U_h)$

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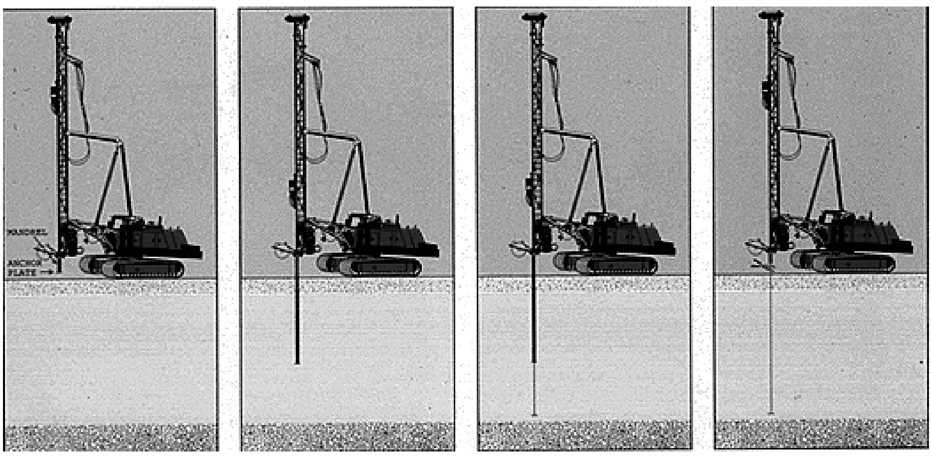
Section 2-3
Installation

Installation of PVD



Installation equipment　　Driving mandrel　　Extracting mandrel　　Cutting drain

After “www.americanwick.com/graphics/stickrigb.gif”

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Types of vertical drain installation rigs

	Type of base machine	Weight of base (t)	Penetration Power (t)	Height of Rig (m)	Max penetration depth (m)	Mechanism of Penetration	Max production / day (m/14 hrs)
Cofra	O & K Excavator	70 - 110	20 - 30	36 - 55.5	50.5	Hydraulic motor	30,000
Econ	O & K Excavator	70 - 120	20 - 30	36 - 56.1	51.2	- do -	27,000
Yuyang	Crane	77 - 100	25 - 30	52.5 - 55.8	52.8	Hydraulic motor	10,000
						sprocket & chain	
Chosuk	Daewoo - Solar	45	25	54.6	51	Hydraulic motor	12,000
	450 - Excavator					multi pulley system	
Dae Yang	Daewoo - Solar	33 - 55	20 - 34	42 -56	51	Hydraulic motor	13,500
	450 - Excavator					Push in roller	
						and clamp	
B + B	Excavator	-	-	31 – 47	29 - 45	Hydraulic	19,200
						sprocket & chain	
B + B	Excavator	-	-	43 – 50	41 - 48	Vibro push in	8,600

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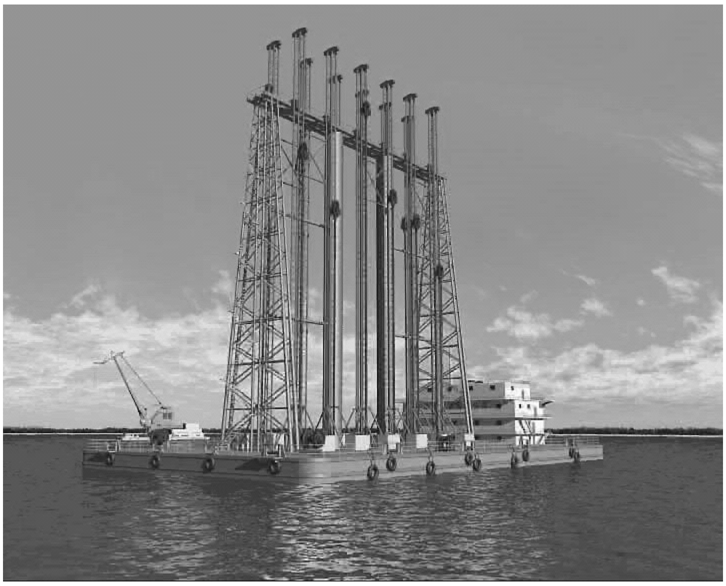
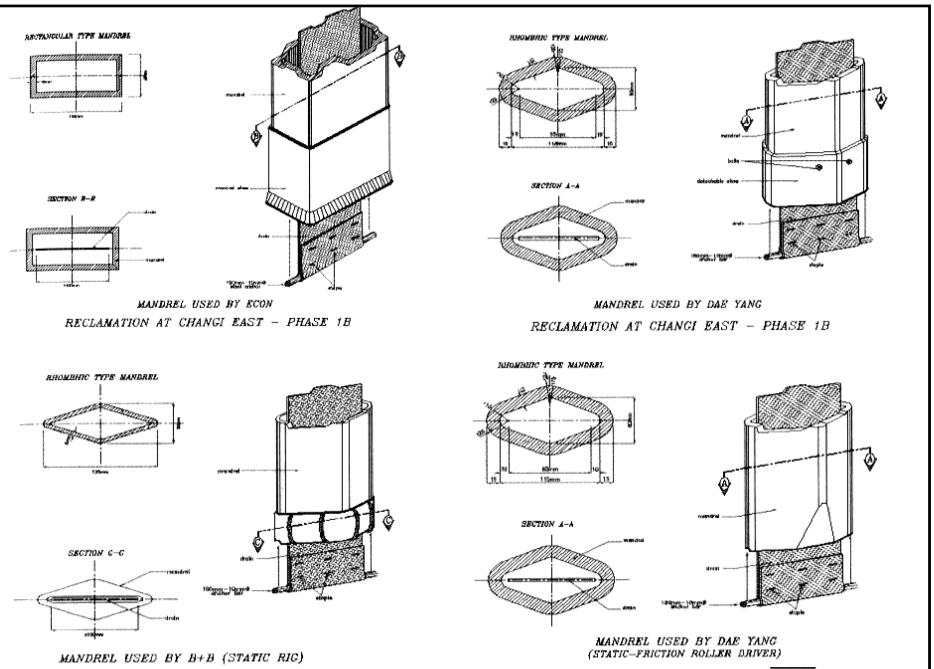
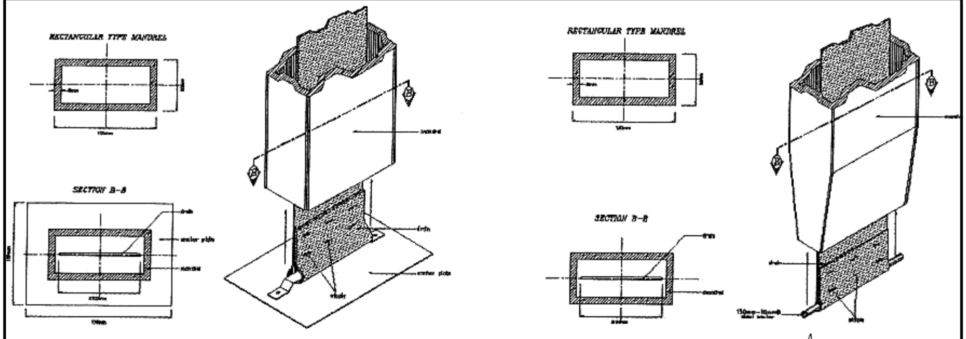
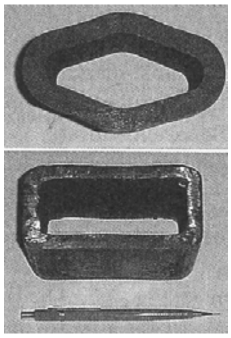


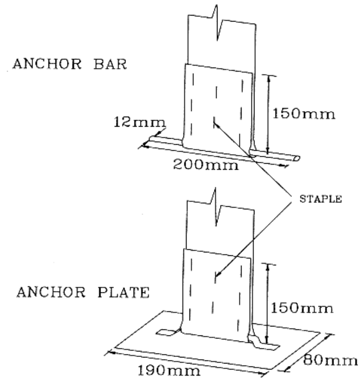


Types of mandrels



Types of Mandrels and Anchors





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The mandrel can bend!

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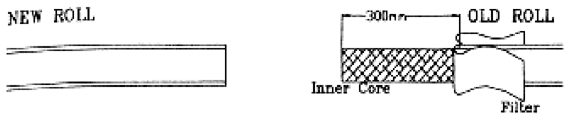
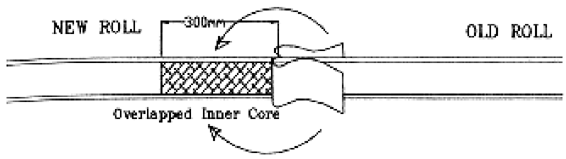
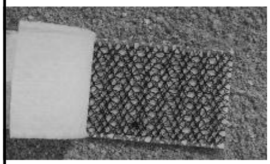
Splicing

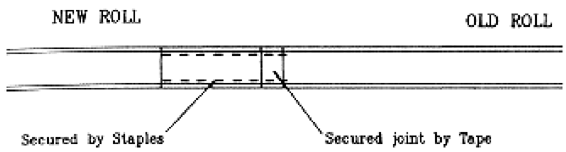


* Insert the opened core into the new drain and make sure smooth overlap:





* Fold back the filter, secure the opening by tape and staple as shown:



Colband drain

After Bo et al. (2003)

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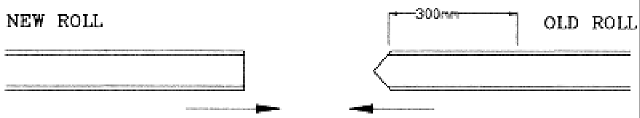
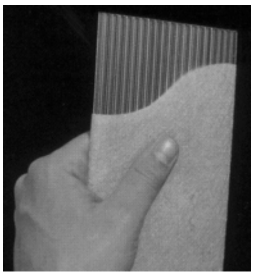
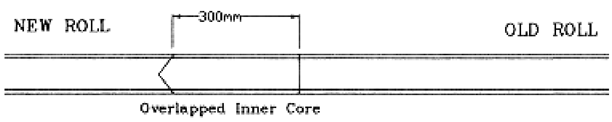
Splicing

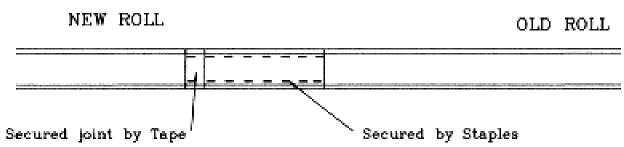




* Overlapped length should be 300mm



* Secure the opening by tape and staple as shown:



Mebra drain

After Bo et al. (2003)

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Installation of Vertical Drains

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Mud flow is observed during drain installation in ultra soft soil.

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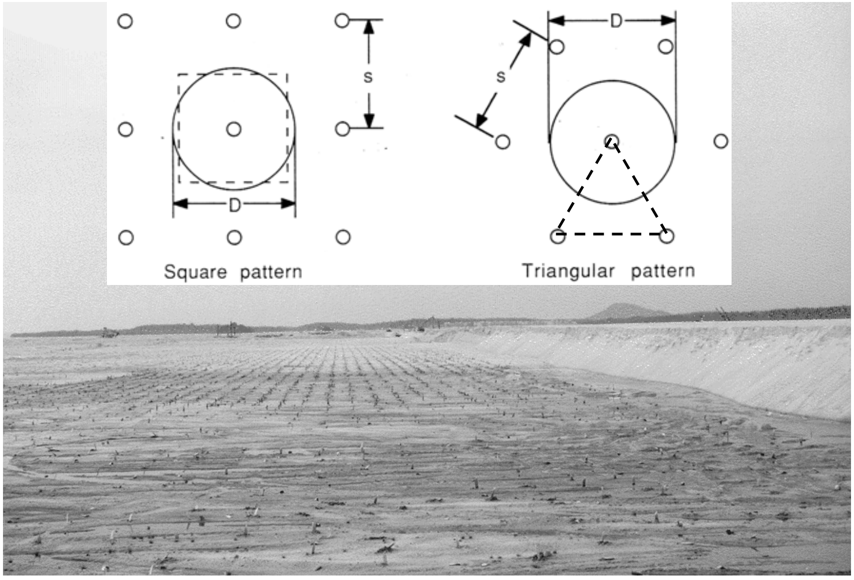

Area Installed with Vertical Drains

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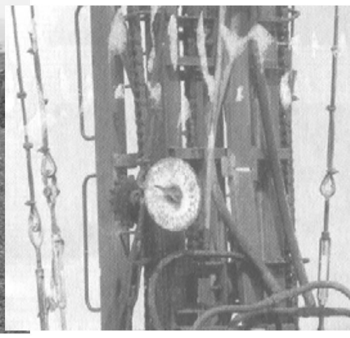
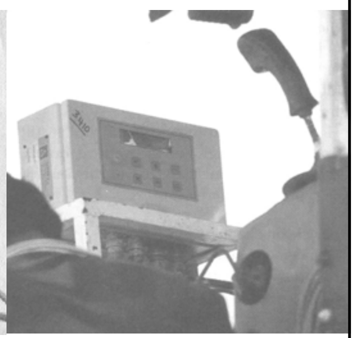
Counters to record Installation Depth





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Preloading using fill or vacuum pressure

Section 2-4

Quality Control and Practical Considerations

2.4.1 Terminology

2.4.2 Properties of PVD

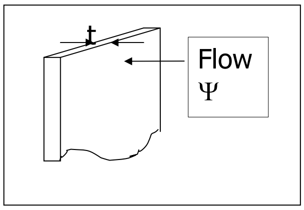
2.4.3 Factors control the PVD selection

2.4.4 Quality control tests

2.4.5 Practical Considerations

2.4.1 Terminology -1

- Permittivity of filter, $\Psi = k_n / t$, 1/s (permeability of the filter/ thickness)



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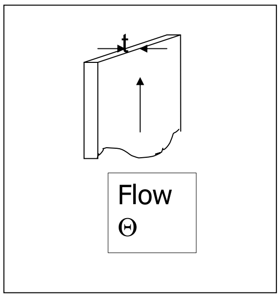
Terminology -2

- Transmissivity of filter , $\Theta = k_d\ t$, m^2/s (In-plane permeability x thickness)



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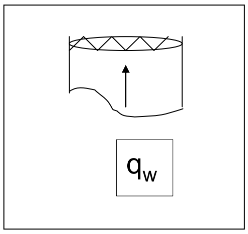
Terminology -3

- Discharge capacity of drain, $q_w = Q / i$, m^3/s



Discharge velocity

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Terminology -4

- Discharge factor

$$D = \frac{q_w}{k_h l_m^{\ 2}}$$

The efficiency of PVD in discharging water is controlled not only by the discharge capacity, but also by the permeability of the soil and the length of the drain.

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Terminology -5

- Apparent opening size (AOS)

AOS is usually defined as the size that is larger than 95% or 90% of the fabric pores, denoted as O_{95} or O_{90}.

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Terminology -6

- D_{85} of soils

D_{85} is used to measure the particle size of soil. It is defined as the size that is larger than 85% of soil particles as measured from the particle size distribution curve.

Sometimes, D_{90}, D_{50}, or D_{15} may be used.

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2.4.2 Properties of PVD

With pure radial flow, the average degree of consolidation, U_h, reached at a certain depth, z, can be calculated by:

$$U_h = 1 - \exp\left(-\frac{8c_h t}{\mu D^2}\right)$$

Coefficient of consolidation of soil in the redial direction

$$\mu = \ln\left(\frac{D}{d_s}\right) + \frac{k_h}{k_s}\ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} + \pi z(2l_m - z)\frac{k_h}{q_w}$$

Permeability of soil

Diameter of smeared zone

Well resistance

Discharge capacity of PVD

Permeability of smeared soil

$$d_w = 2(a+b)/\pi$$

Equivalent drain diameter

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2.4.3 Factors Affecting the Performance of PVD

Discharge capacity

Tensile strength

Filter

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Discharge Capacity

- Normally the well resistance is ignored in design. Therefore, we must ensure the discharge capacity of the drain is sufficient so that well resistance is negligible.
- How large is sufficient?

Discharge Capacity (Cont'd)

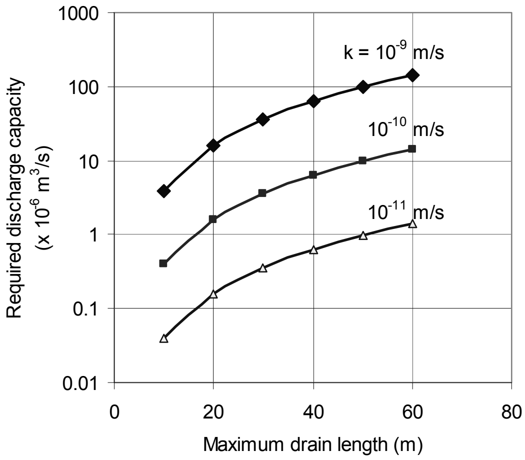
- The condition for well resistance to be insignificant is:

$$D = \frac{q_w}{k_h l_m^2} \geq 7.85$$

$$q_{req} \geq 7.85 F_s k_h l_m^2$$

F_s = factor of safety, F_s = 4~6

Discharge Capacity (Cont'd)



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Discharge Capacity (Cont'd)

If we take F_s = 5, k_s = 10^{-10} m/s, and l_m = 25 m,
then q_w = 2.45 x 10^{-6} m^3/s, or 82 m^3/yr.
If l_m = 50 m instead of 25 m,
then q_w = 9.81 x 10^{-6} m^3/s, or 327 m^3/yr.

If k_s = 10^{-9} m/s instead of 10^{-10} m/s,
then q_w = 98.1 x 10^{-6} m^3/s, or 3,270 m^3/yr.

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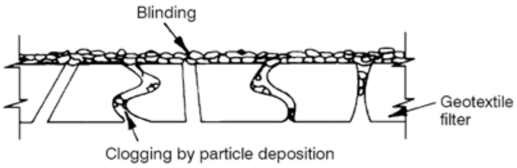
Tensile Strength

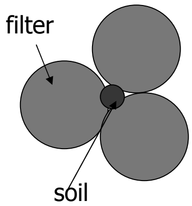
- To sustain the tensile load applied to PVD during installation.
- Commonly, it is required the drain to have a tensile strength of larger than 1 kN at a tensile strain of 10% at either dry or wet conditions.
- Sometimes, permanent necking can occur. This will reduce the discharge capacity.

Filter

- Soil retention ability
  - $O_{95} \leq (2 \sim 3)\ D_{85}$ (D_{85}=0.01~0.03mm)
  - and
  - $O_{50} \leq (10 \text{ to } 12)\ D_{50}$ (D_{50}=0.001~0.002mm)
- Permeability
  - $k_f \geq 10\ k_s$ (normally not a problem)
- Clogging resistance
  - $n \geq 30\%$
  - $O_{95} \geq 3\ D_{15}$
  - $O_{15} \geq (2 \text{ to } 3)\ D_{10}$



Filter design criteria



Terzaghi and Peck (1948) suggested:

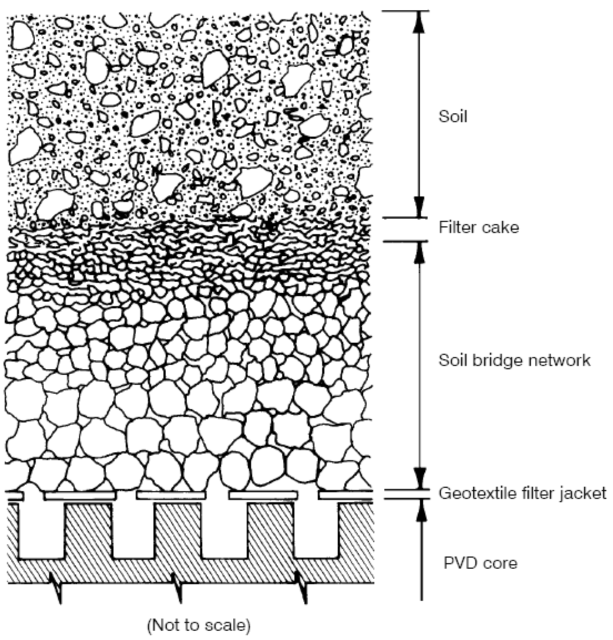
For retention:

$D_{15}(F) < 4D_{85}(S)$

For permeability:

$D_{15}(F) > 4D_{15}(S)$

How filter works



2.4.4 Quality Control Tests

Discharge Capacity

Tensile Strength

Permeability and AOS of Filter

Need for Quality Control Tests

- Several million meters of PVDs can be used in one project and the supplies are over a few months. We have to know the consistence of the product. The only way to check is to conduct quality control tests.
- It is NOT possible to make direct comparison among q_w given by suppliers unless the drains are tested in the same way.

Discharge Capacity

- Normally measured under straight and deformed conditions.

- Types of Tests
  - Straight drain tester
  - Buckled drain tester
  - Kinked drain tester

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Discharge Capacity Tests

- ASTM 4716 is NOT the standard for this test, although it is often used.

- The drain to be tested MUST be embedded into soil.

- MUST have independent water head measurement.

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Straight Drain Tester

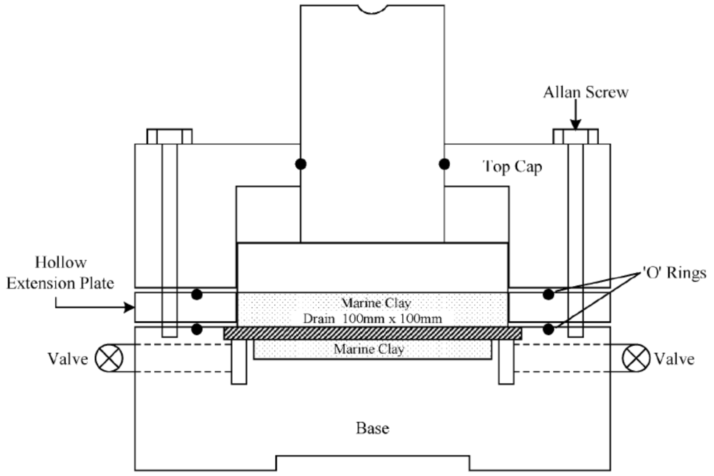
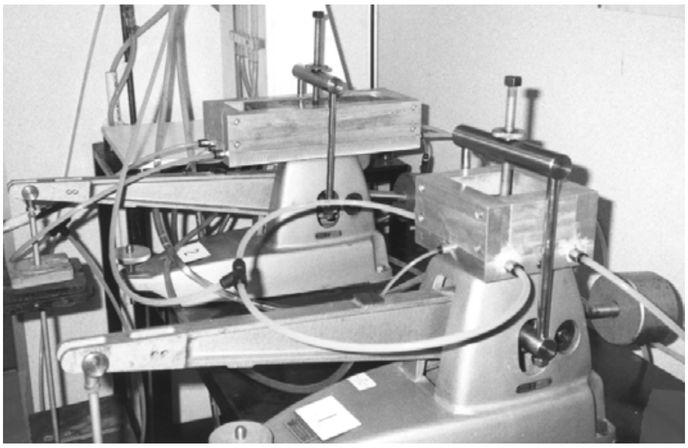
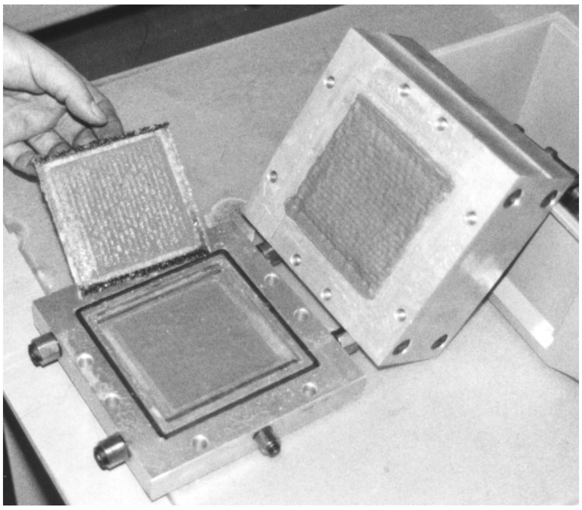


Fig. 4. A cross-section of the straight drain tester.

Straight Drain Tester (Cont'd)



Straight Drain Tester (Cont'd)



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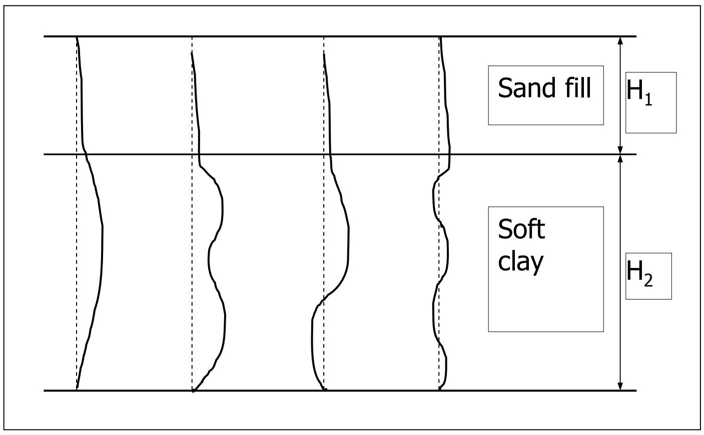
Buckling of Drains



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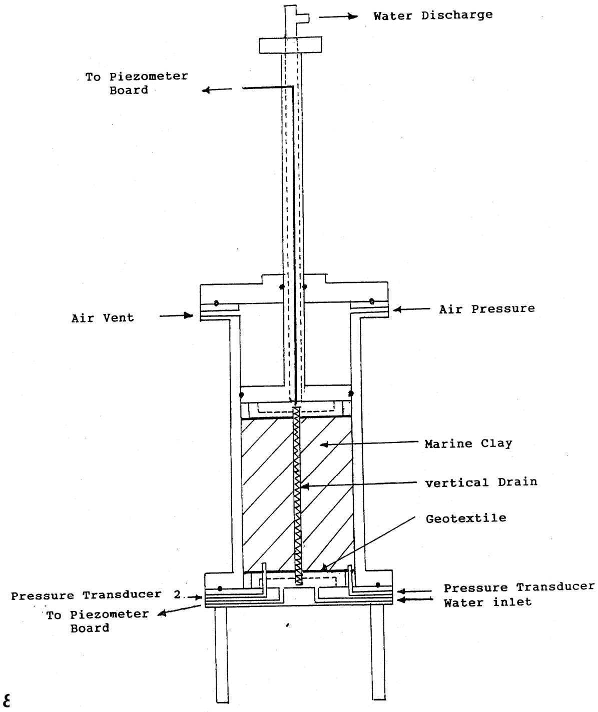

Buckled Drain Tester

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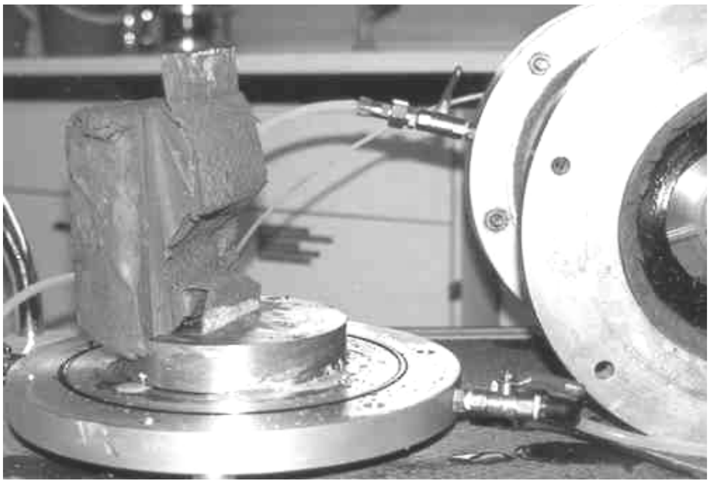
Buckled Drain Tester (Cont'd)



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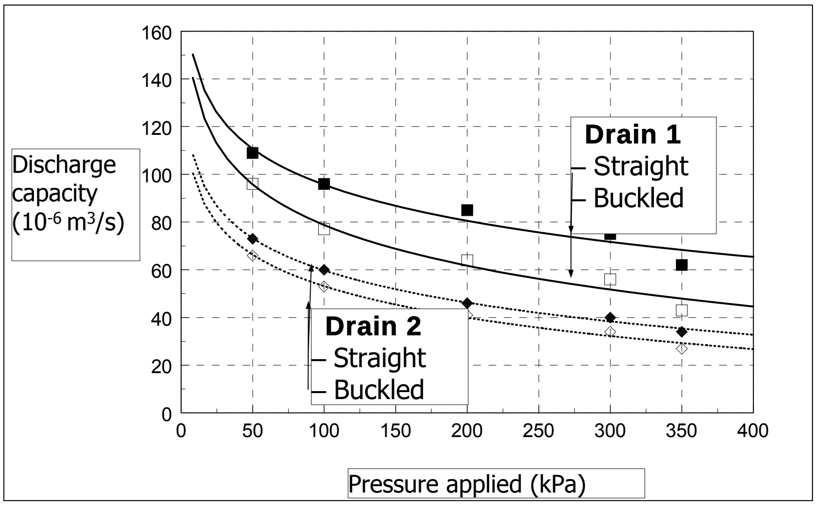
Example - Discharge Capacity



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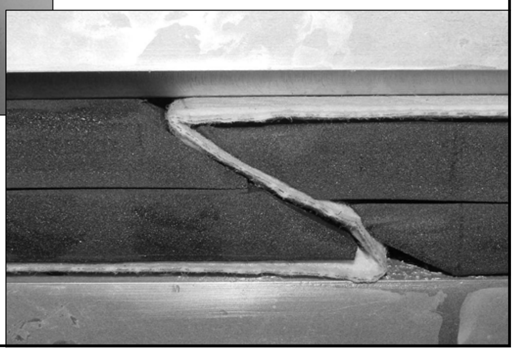
Kinked Drain Tester



Discussion on q_w Measurement

- The measurement of q_w is affected by hydraulic gradient *i*. The greater the *i* the smaller the q_w. It appears *i* =0.5 is the most suitable value for q_w measurement.
- q_w is a measure of the in-plane flow capacity of the drain. The measurement is not affected by the smear effect. For the same reason, it is NOT affected by the size of the soil media used to embed the drain.

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Discussion on q_w Measurement (Cont'd)

- In practice, the drain wells work together. It is the overall effect which matters. Therefore, the drains should be tested under typical, not the most severe conditions.
- The q_w reduces with time, but so does the rate of consolidation.

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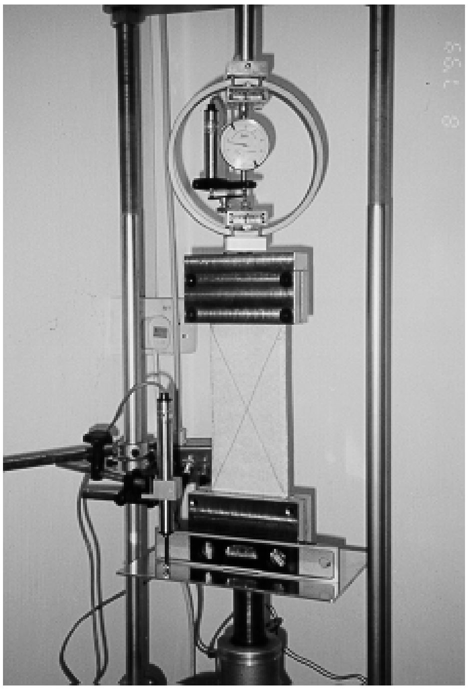

Tensile Strength Test

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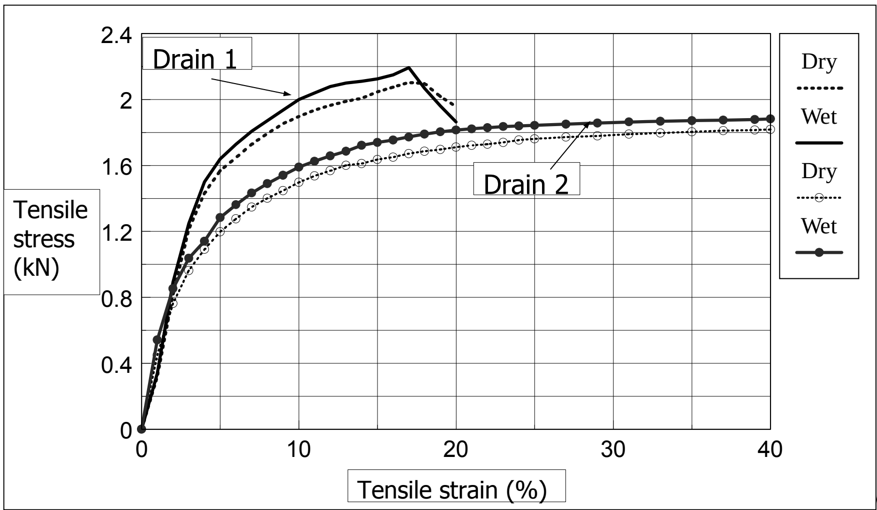
Example -Tensile Strength Test



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Permeability of Filter

$$k = \frac{Q\delta}{A\Delta h t} R_t$$

As specified by ASTM D4491-96, the permeability is taken as the value corresponding to a 50 mm water head.

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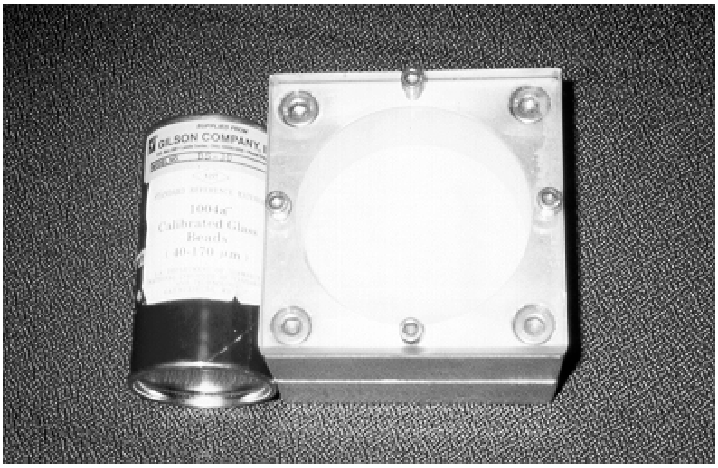
AOS of Filter



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AOS of Filter (Cont'd)

B = 100 P / T (4.14)

Where: B = beads passing through specimen, %
P = mass of glass beads in the pan, g, and
T = total mass of glass beads used, g.

Table 4.2 Cumulative size distribution percent of spheres with diameters less than the size indicated

Weight (%)	2.7	10.9	19.2	24.7	28.9	33.3	36.8	38.4	40.1	43.6
Diameter (μm)	40	44	48	52	56	60	64	68	72	76

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Quality Control of PVD


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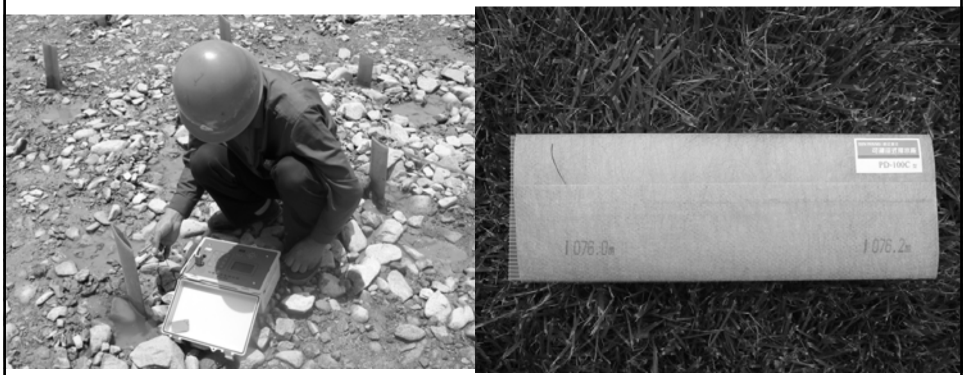
Quality Control of PVD (Cont'd)



Devices to measure the length of the drain

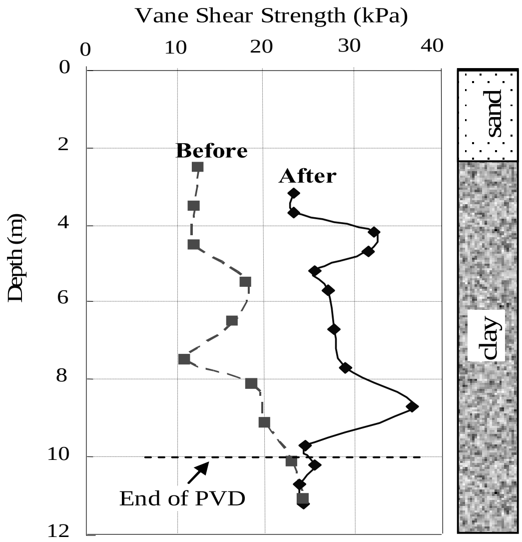
2.4.5 Practical Considerations

- Select drain with discharge capacity matches the need:

$$q_{req} \geq 7.85 F_s k_h l_m^2$$

- Select the filter with AOS matches the soil to be improved. For Singapore marine clay: $O_{95} \leq (4\text{-}7.5)D_{85}$. D_{85} =0.01–0.02mm. By experience, thicker filters are better. The mass/area > 90 g/m^2.
- If the drain spacing is smaller, the smear effect will be more significant.
- Complete penetration of drain into soft clay may not be necessary.

Comparison of vane shear strength before and after soil improvement



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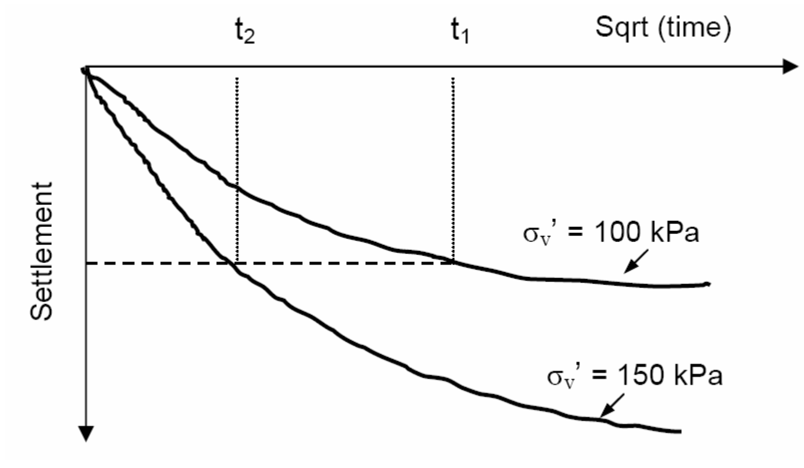
Reduction of consolidation time through application of higher surcharge



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Summary-1

- The selection of PVD is controlled not only by the drain itself, but also by the length of the drain and the permeability of the soil. Therefore, one cannot borrow specifications without considering the site conditions and the nature of the project.

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Summary-2

- The discharge capacity q_w must be adequately large so the well effect can be ignored.
- The drains also must have sufficient tensile strength. The AOS of the filter should match the grain size of the soil. The permeability of the filter should also have sufficiently large.

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Summary-3

- Quality control tests need to be conducted during a project to (a) compare & verify the properties of different PVD, (b) to ensure the design assumptions are met, and (c) to ensure the quality consistency.
- Some simple testing devices and methods for quality control tests for PVD have been introduced.

Concluding Remark

- **For the successful implementation of a drain project the design must take into consideration many factors such as the site and soil conditions, the client's requirements, the quality control of the drains, the method of installation, the experience of the contractor and the evaluation and interpretation of the soil instrumentation, laboratory and in-situ test data. A holistic approach to drain design has therefore to be adopted and experience plays an essential role in achieving the desired results.**

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Section 2.5 Case Studies

Changi International Airport

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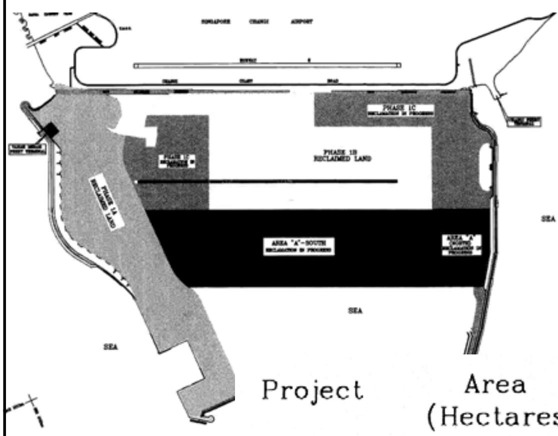
Changi East Land Reclamation Project



Area = 2000 ha

Sand = 272 M m^3

PVD = 140 Mm

Project	Area (Hectares)	Volume of Sand (Mm3)	Length of Vertical Drain (Mm)
PHASE 1A	501	65	–
PHASE 1B	520	75	28
PHASE 1C	451	68	49
AREA "A" – NORTH	90.7	12	13.3
AREA "A" – SOUTH	450	52	50.4

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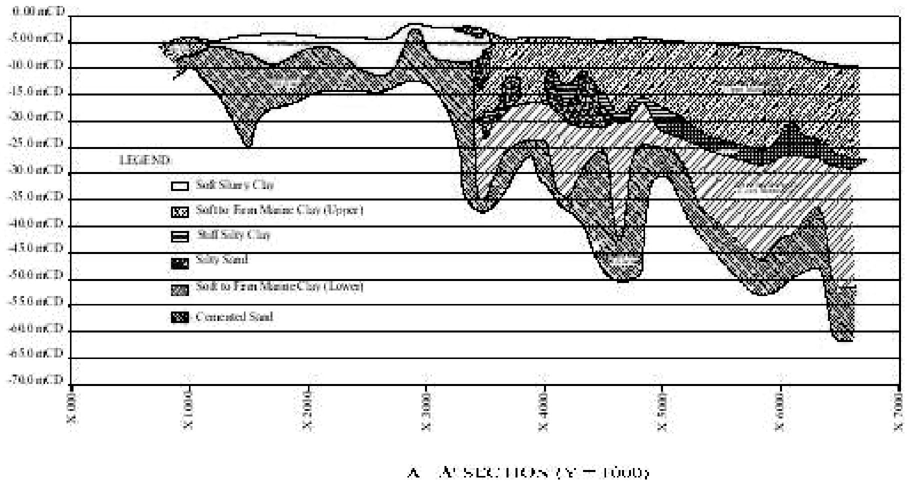

Soil profile along Section A-A’

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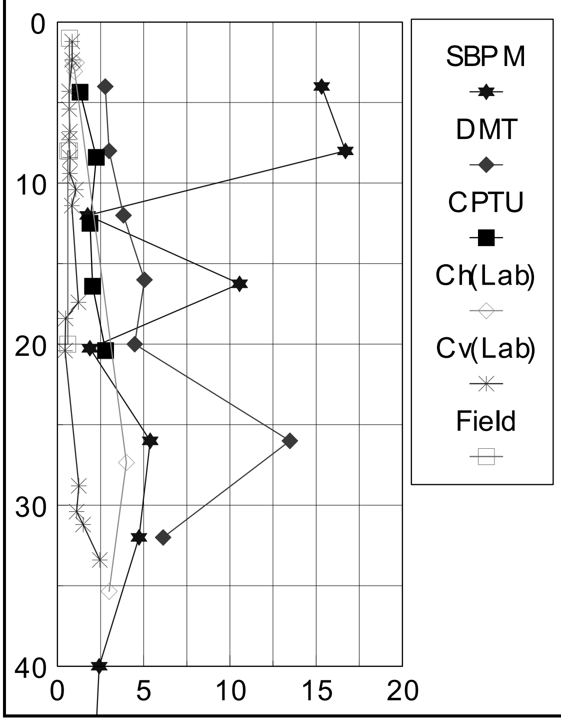

Comparison of different c_v measurements for Singapore marine clay

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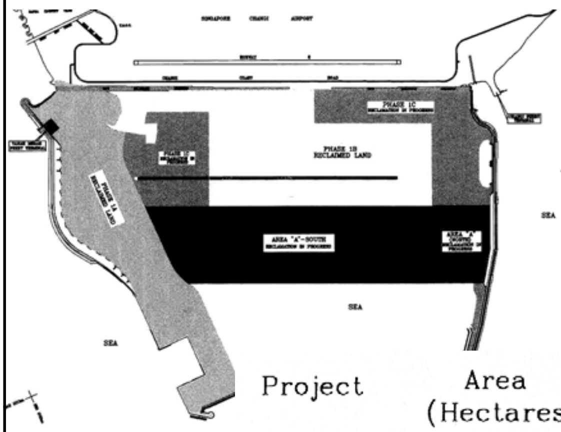
Changi East Land Reclamation Project



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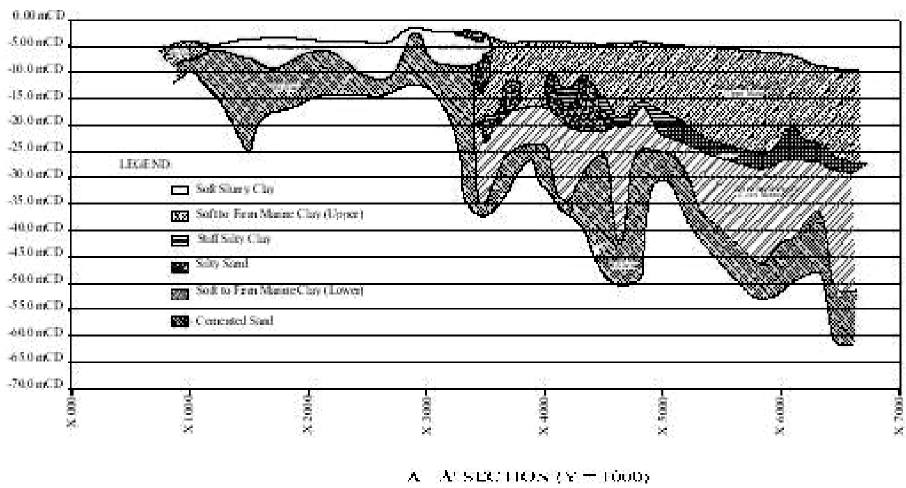

Soil profile along Section A-A’

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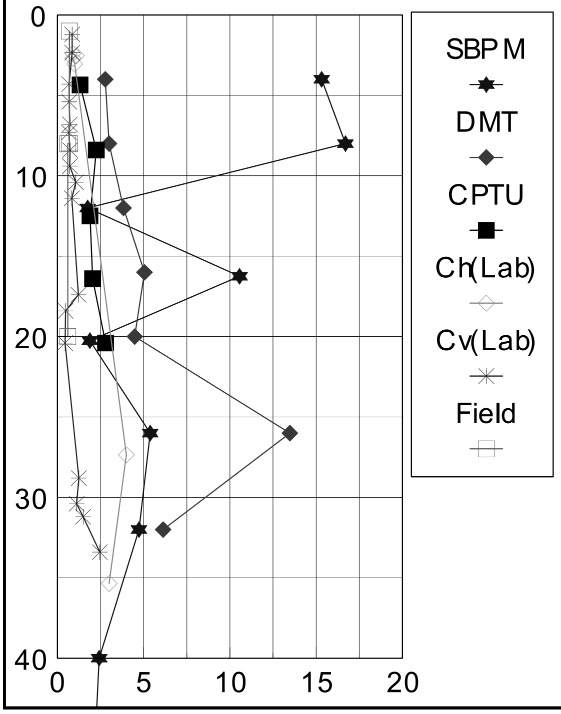

Comparison of different c_v measurements for Singapore marine clay

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Depositing sand

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Installation of Vertical Drains

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Area Installed with Vertical Drains

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Instrument Clusters

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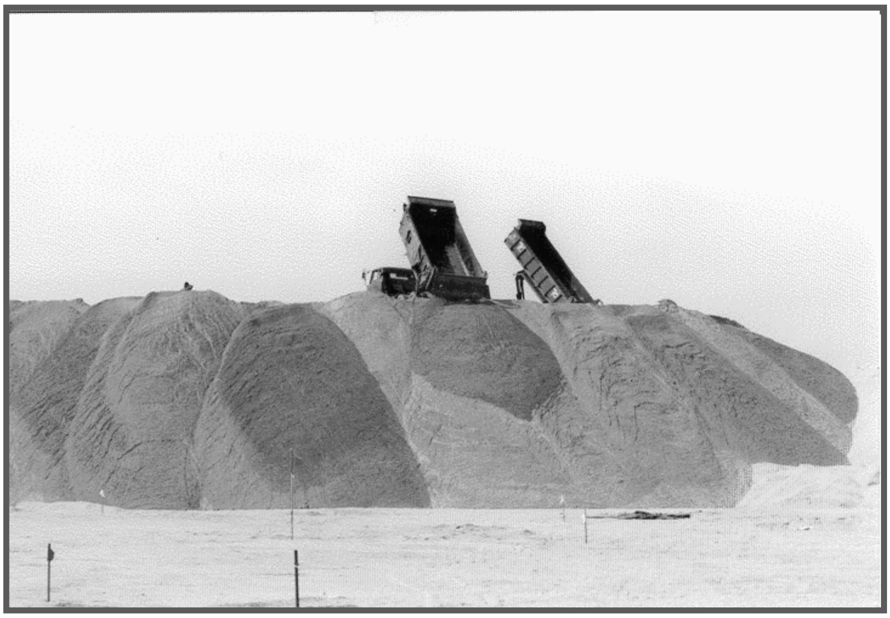

Application of Surcharge

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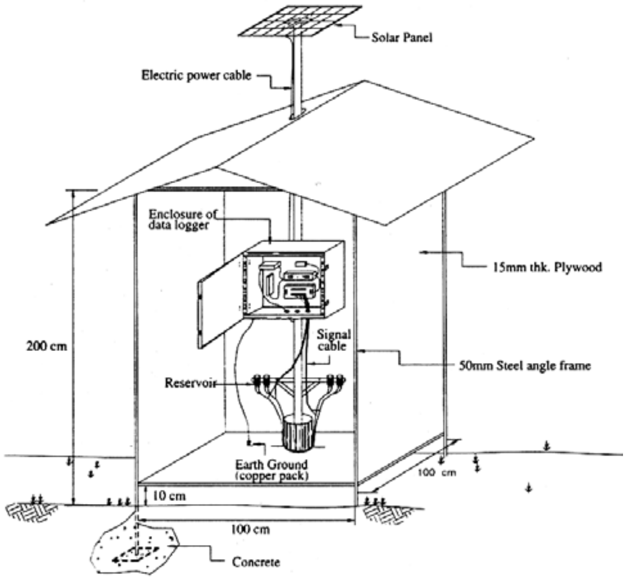

Data collection hut

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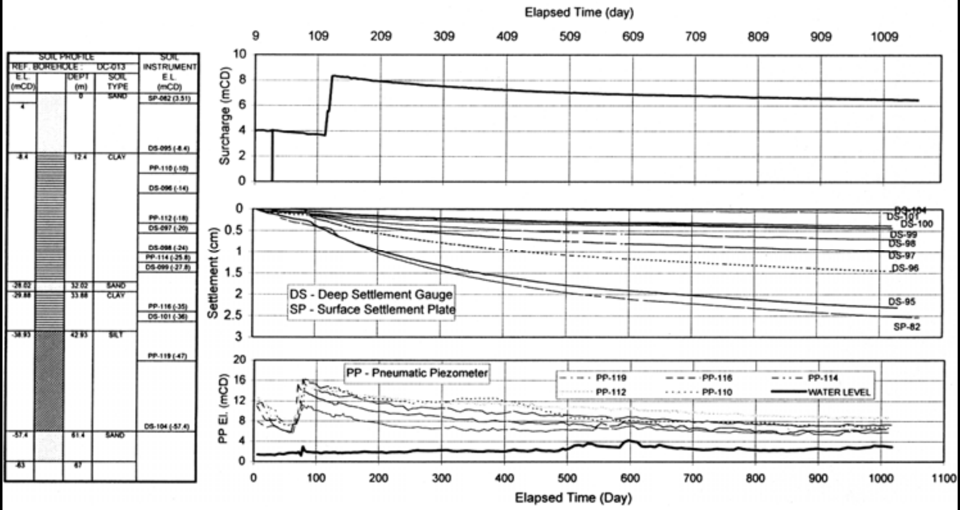

Settlement and pore water pressure monitoring

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Dynamic Compaction

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Soil Improvement for the slurry pond

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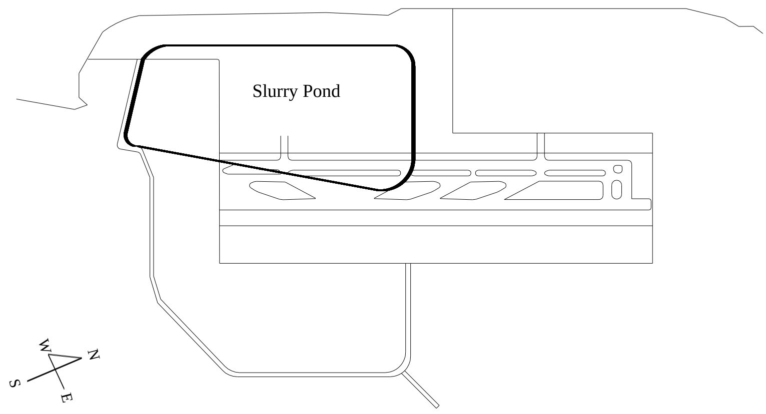
Location of the slurry pond



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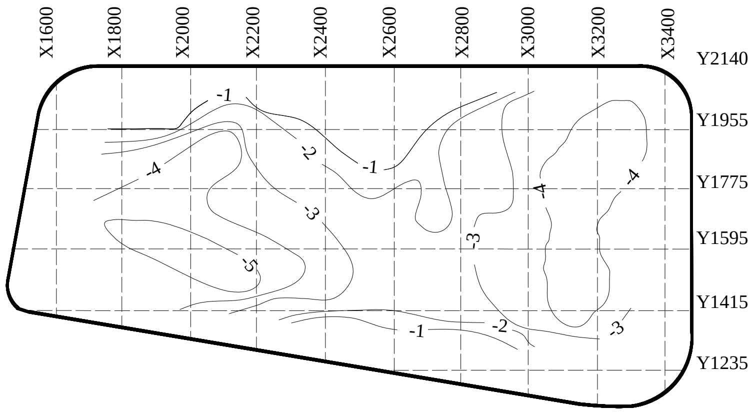
Water depth contour



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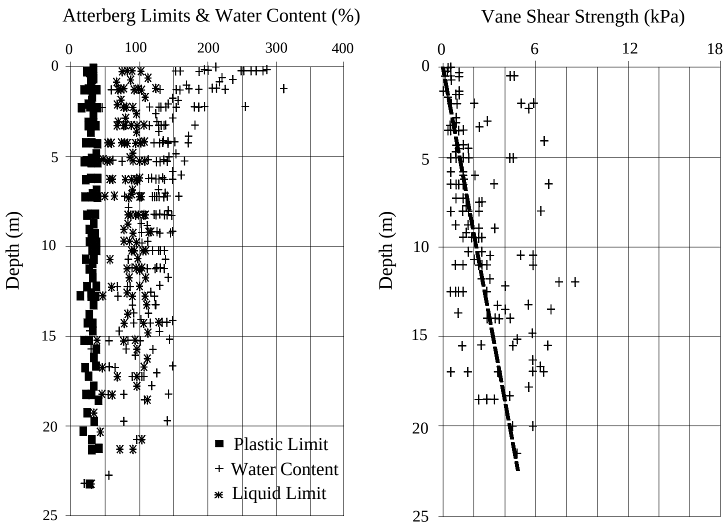
Properties of slurry



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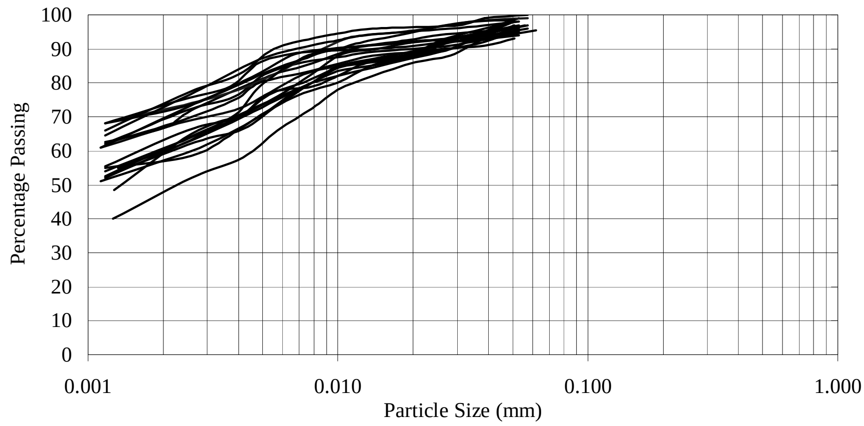
Grain size distribution of slurry soil



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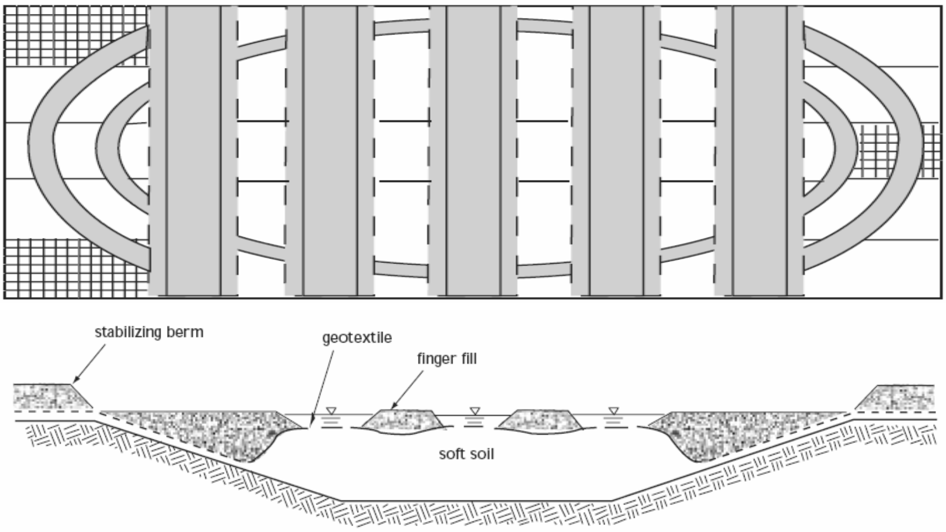
Broms' Method



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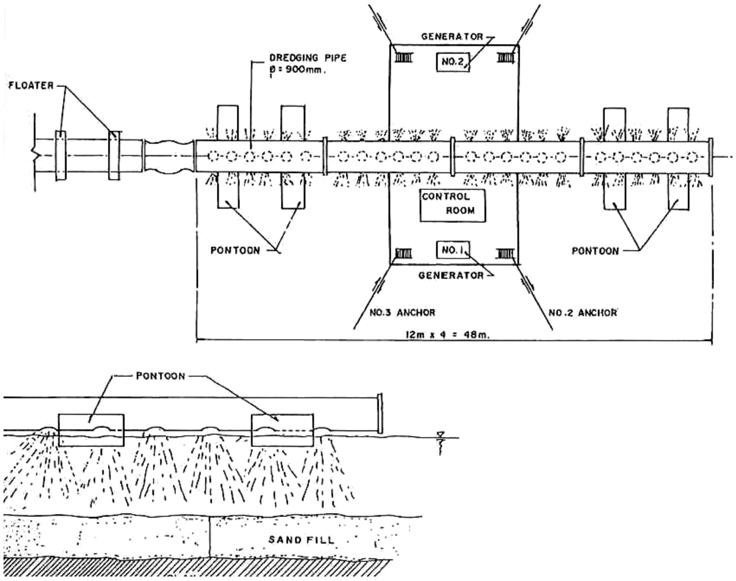
Sand spreader



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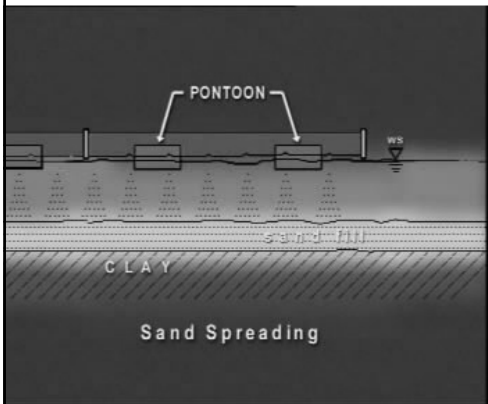
Sand Spreading



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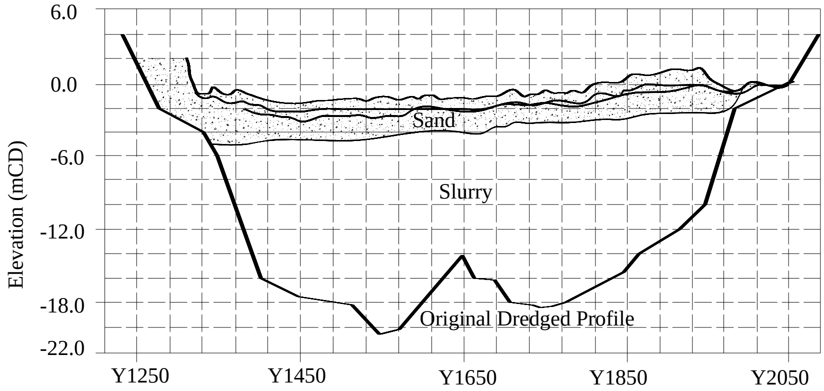
Spreading of sand cap



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Slurry burst

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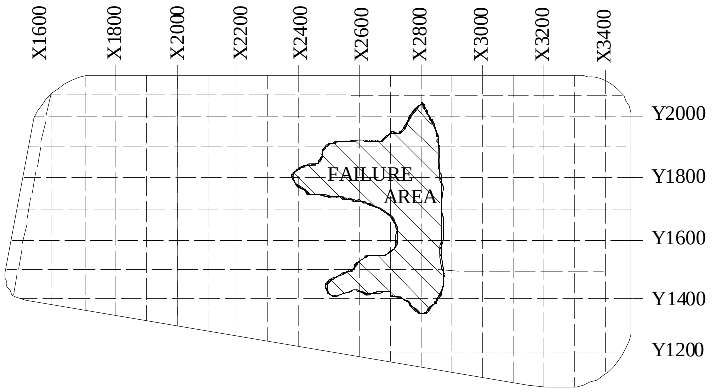
Failed area



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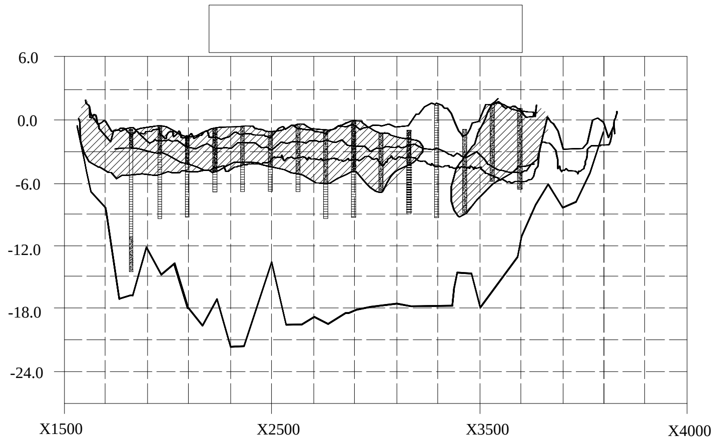
Cross-section along Chainage Y1700 after failure



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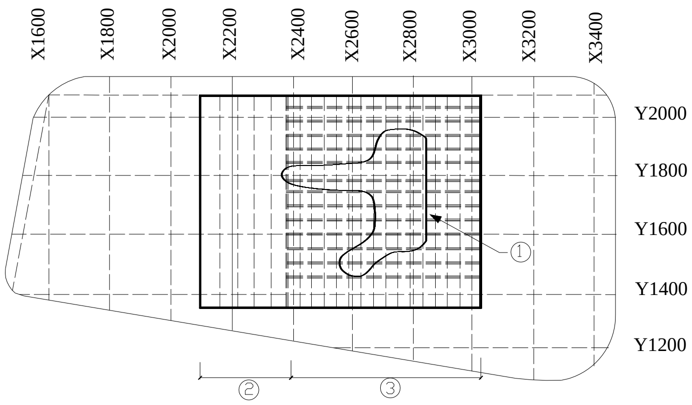
Area covered by geotextile sheet



700 x 300 m + 700 x 600 m = 630,000 m²

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Properties of geotextile

- **Woven geotextile HS150/150 with tensile strength of 150 kN/m in both warp and weft directions.**
- **Two layers of geofabrics with tensile strength of 100 kN/m in the warp direction and 50 kN/m in the weft direction.**

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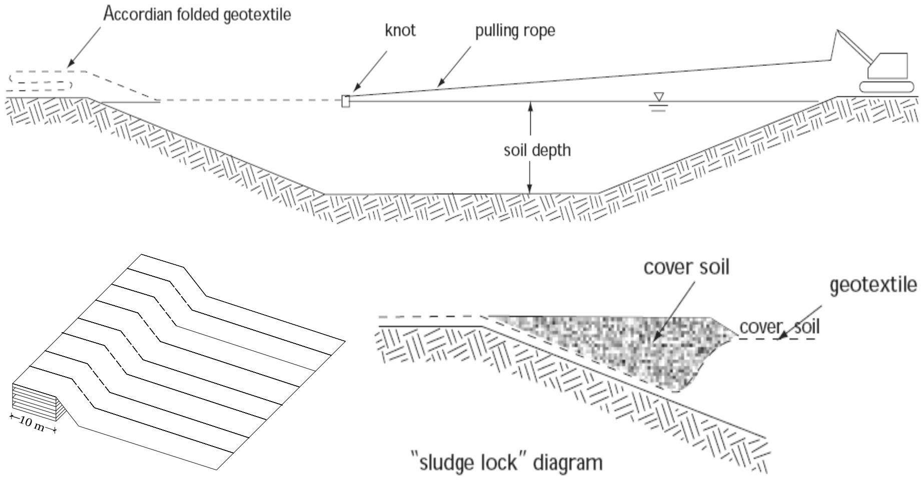
Placement of geotextile



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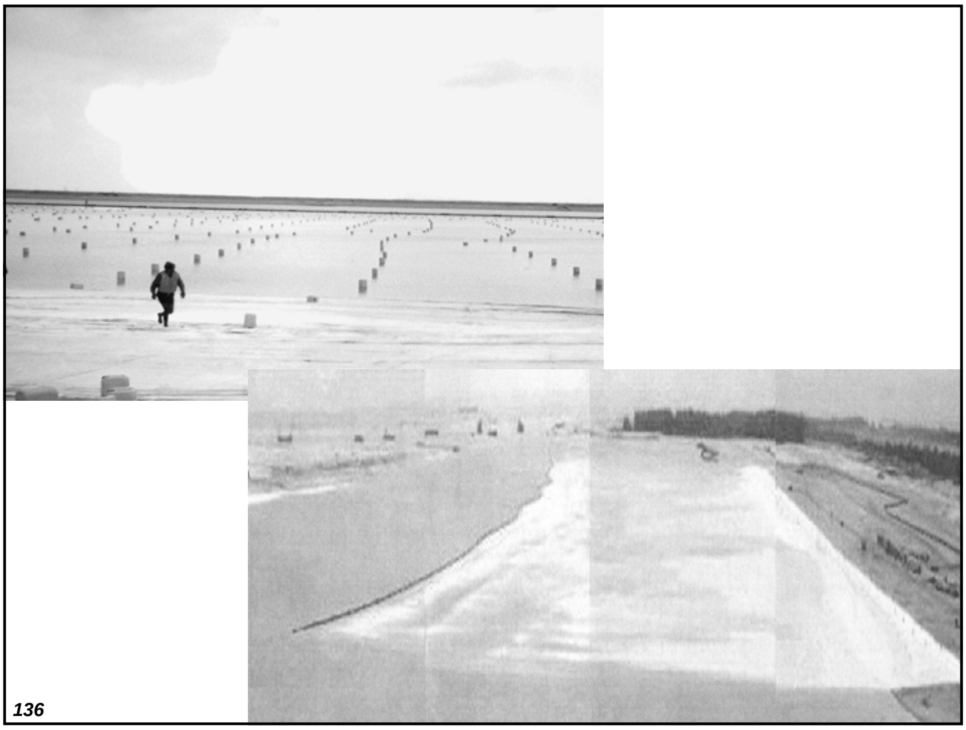

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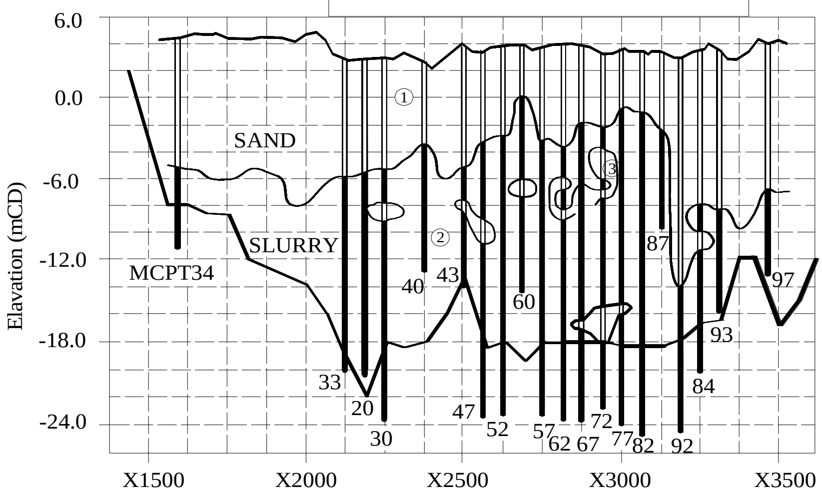
Soil profile after sand placement



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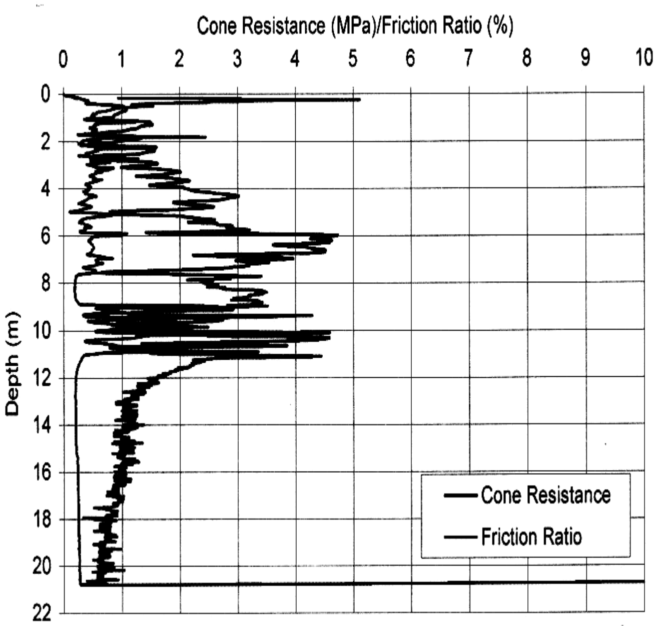

Mini-CPT results

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Reclamation of the slurry pond

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Soil improvement using PVDs

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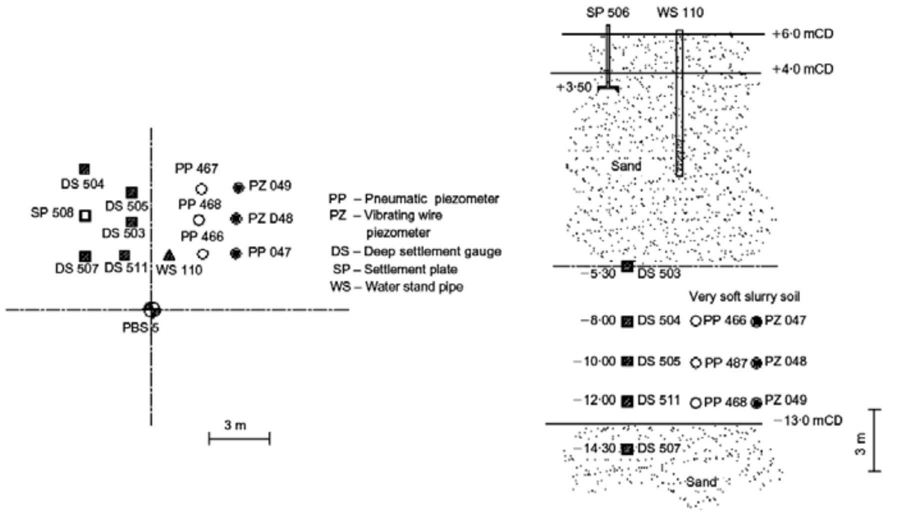
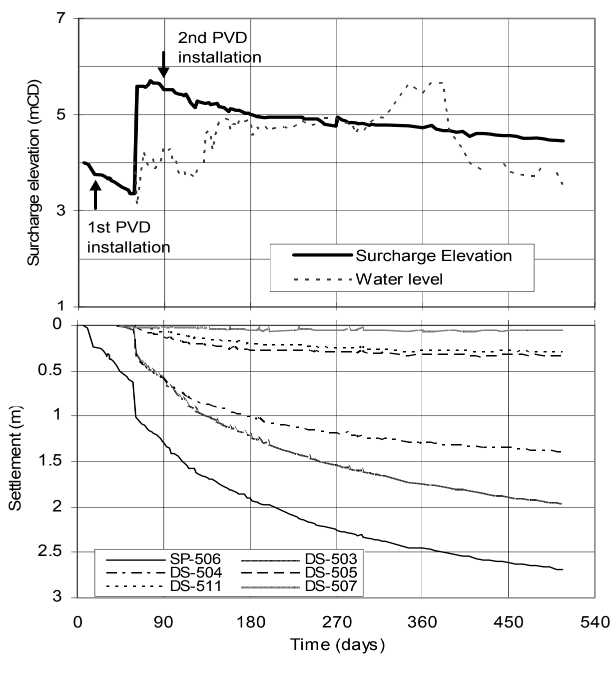
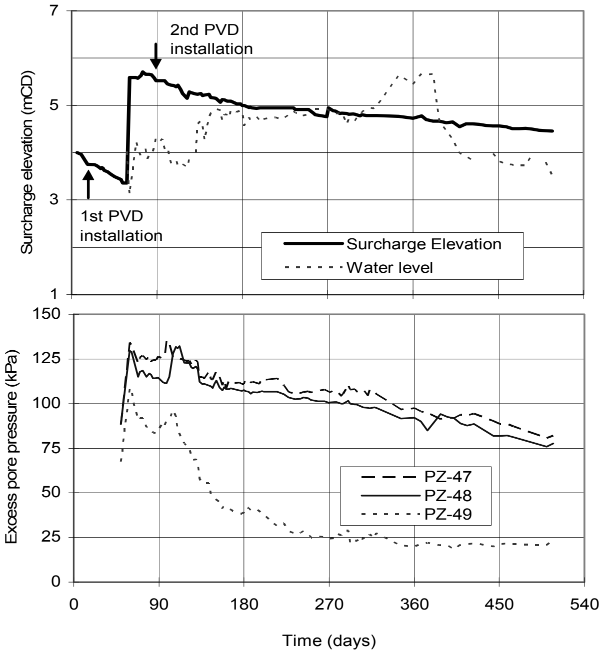
Instrumentations





Settlement under surcharge

Excess pore pressures under surcharge



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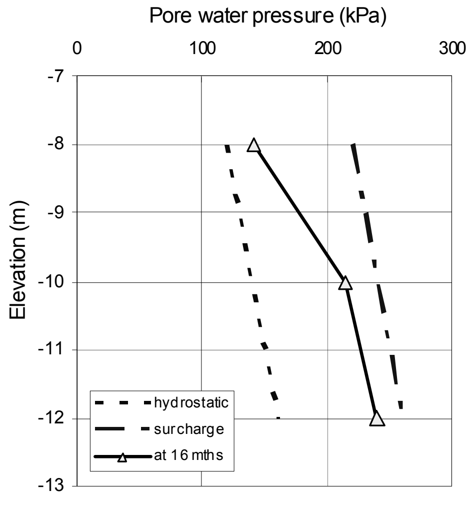

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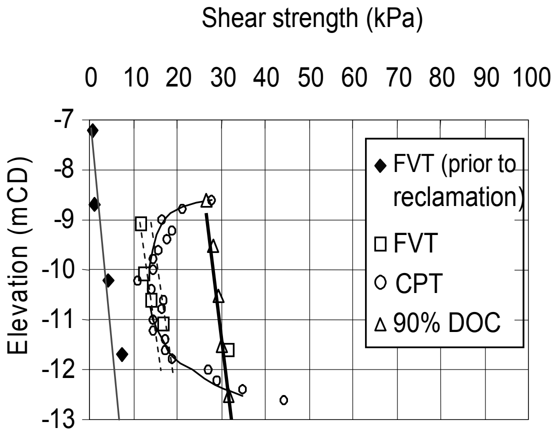
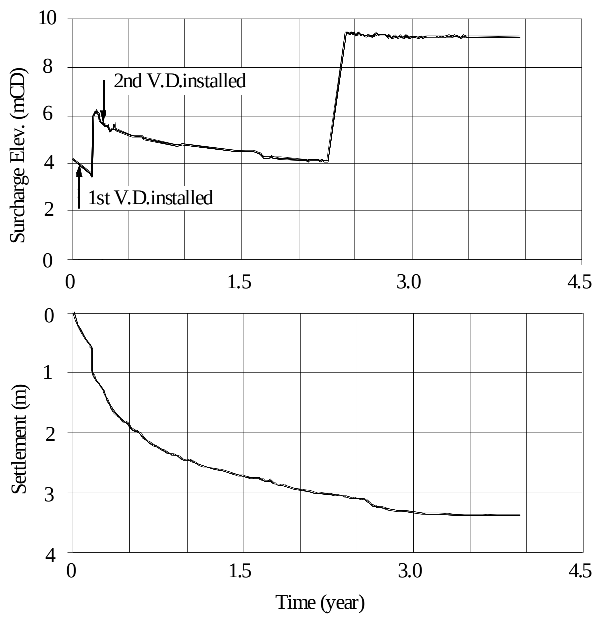
Shear strength profile in 14 mths



$U_{vh} = 0.45, U_v = 0.1, U_h = 0.4$ from $(1 - U_{vh}) = (1 - U_v)(1 - U_h)$



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Summary

- For reclamation over ultra soft, high water content slurry, a direct placement of fill can be difficult. Geotextile may have to be use to cap the slurry before sand fill can be placed.
- The sand placement sequence should be designed so favourable arching stress is generated in the geotextile.
- The slurry still need to be improved after the sand placement.

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