

LECTURE SERIES AND WORKSHOPS ON GEOTECHNICAL ENGINEERING IN PRACTICE

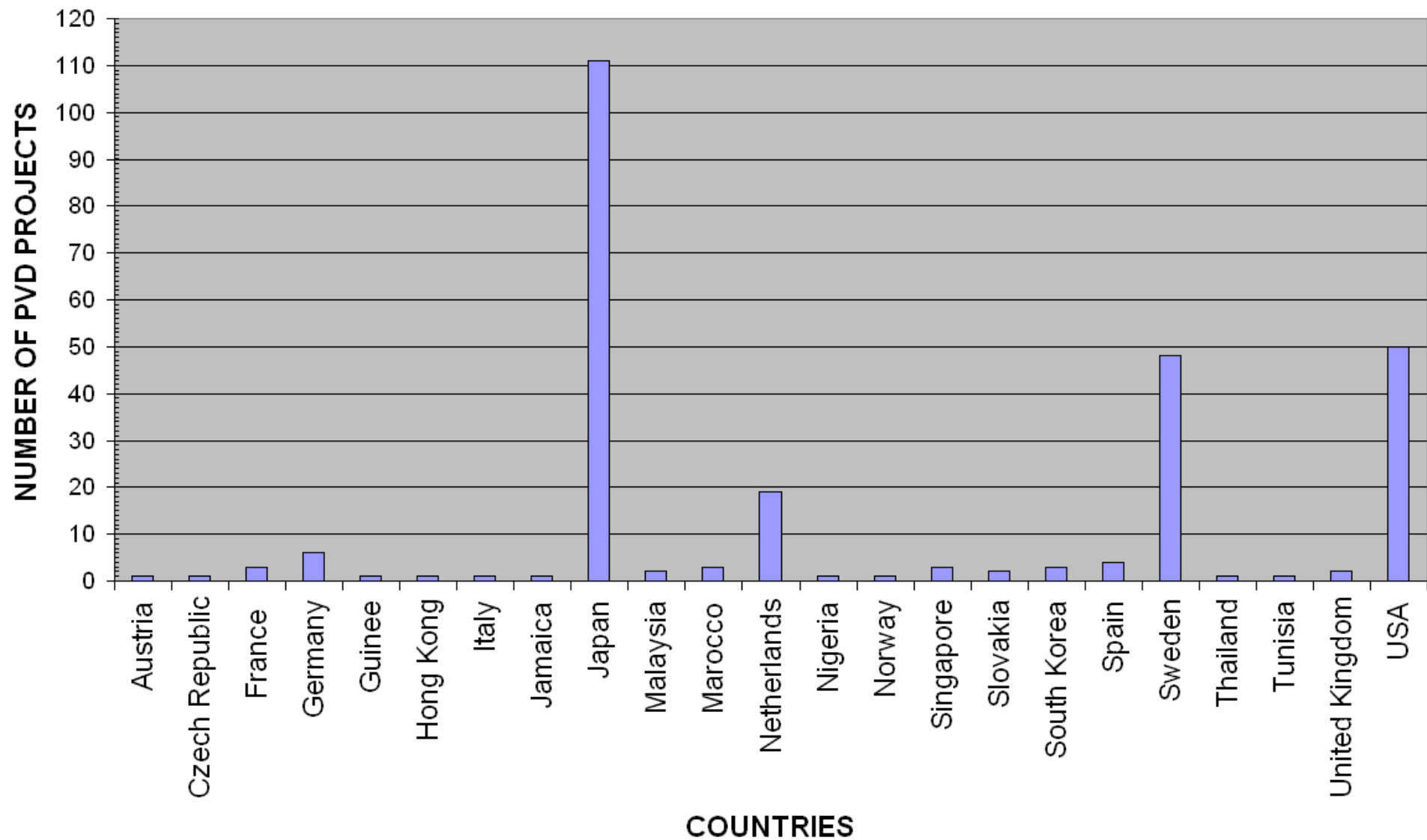
Vertical Drainage From Theory to Practice

Sven Hansbo & K. Rainer Massarsch
Sweden

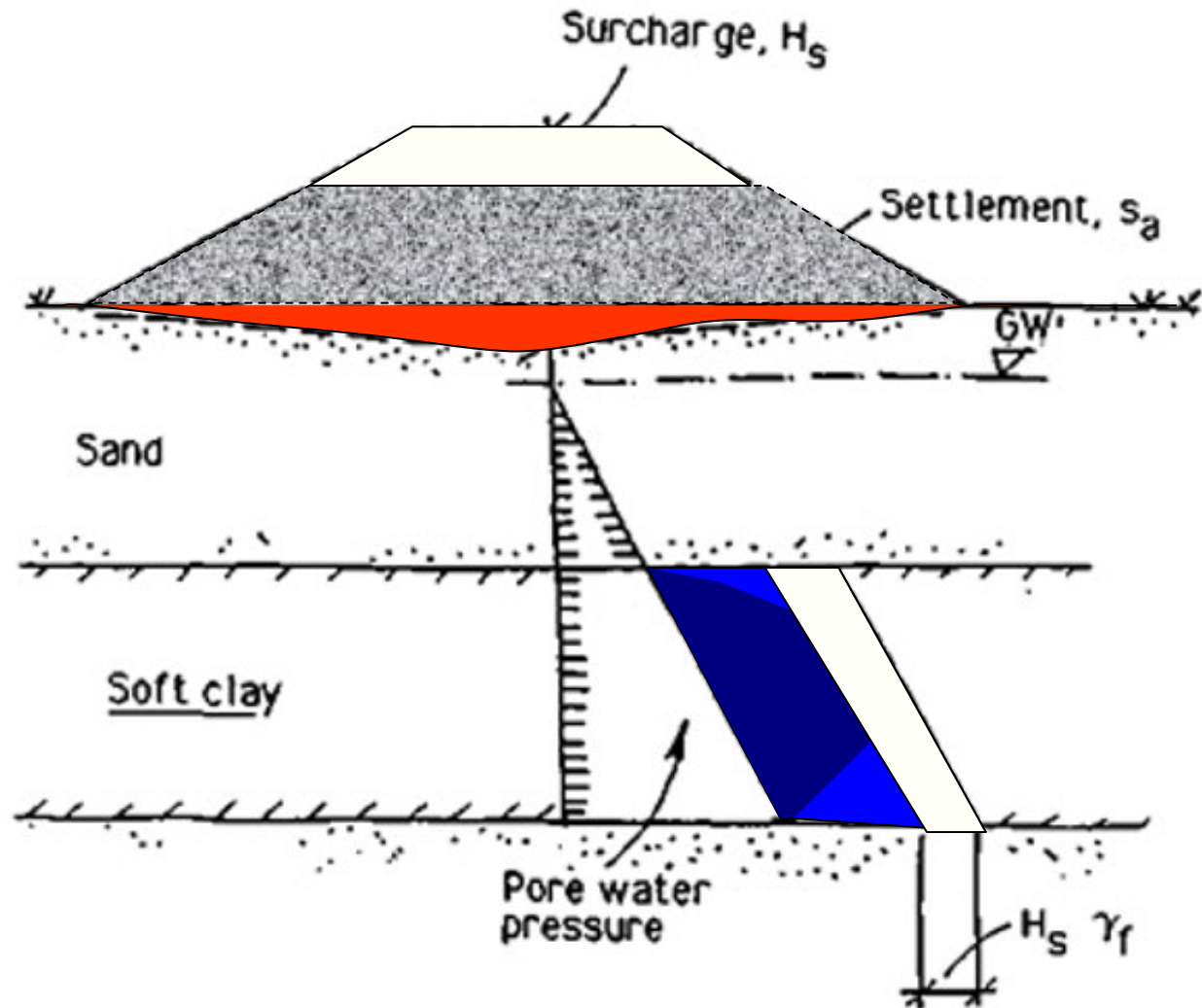
Use of PVD Method

- In **Japan**, the total length of installed band drains is in excess of 37 million m. The average length of installed drains on these projects is in excess of 20 m.
- During the same period, almost 50 band drain projects have been carried out in **Sweden**, with a total drain length of 11 million m.
- In the **North America**, more than 50 projects have been reported, followed by the Netherlands with about 20 projects.

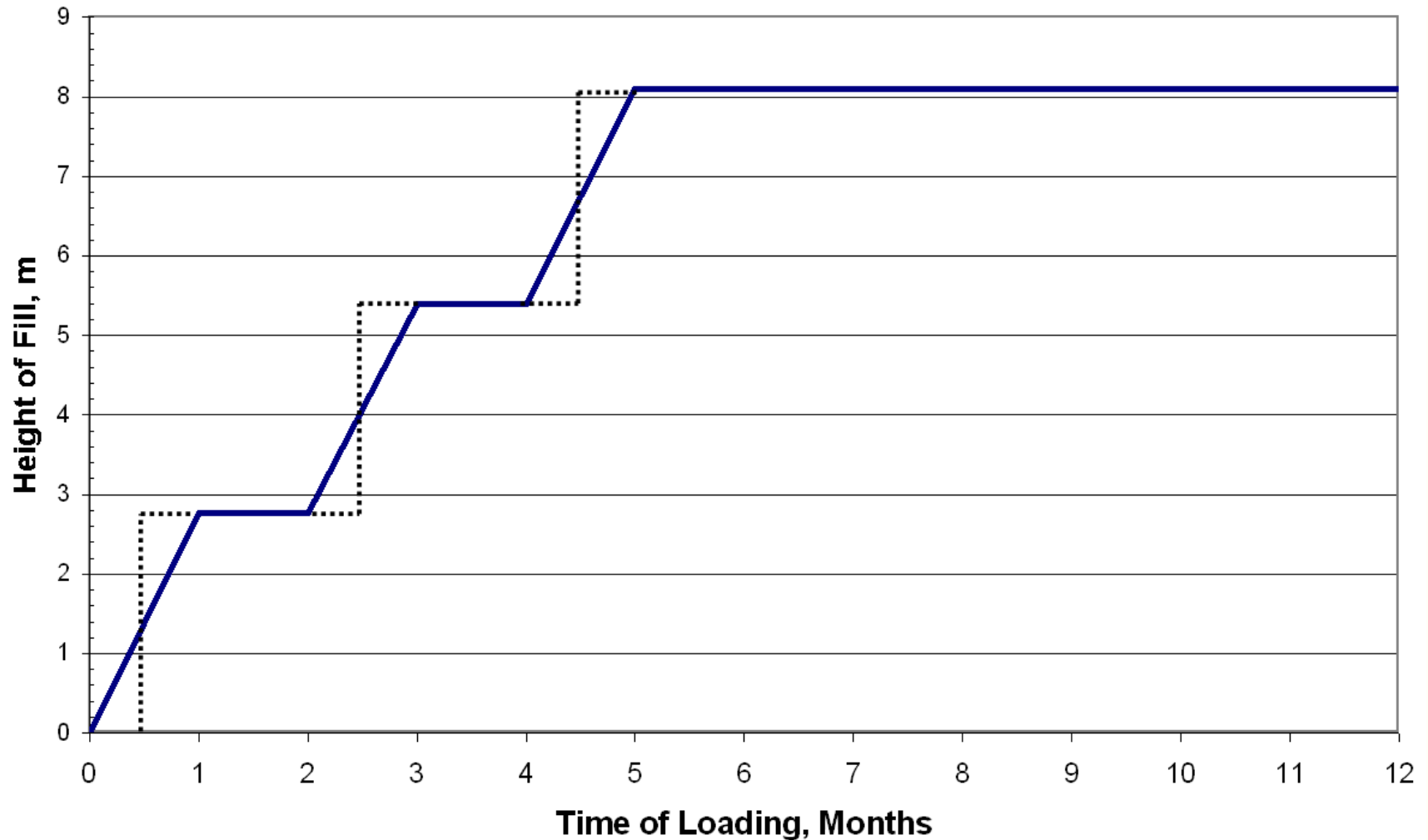
Survey of International PVD Projects



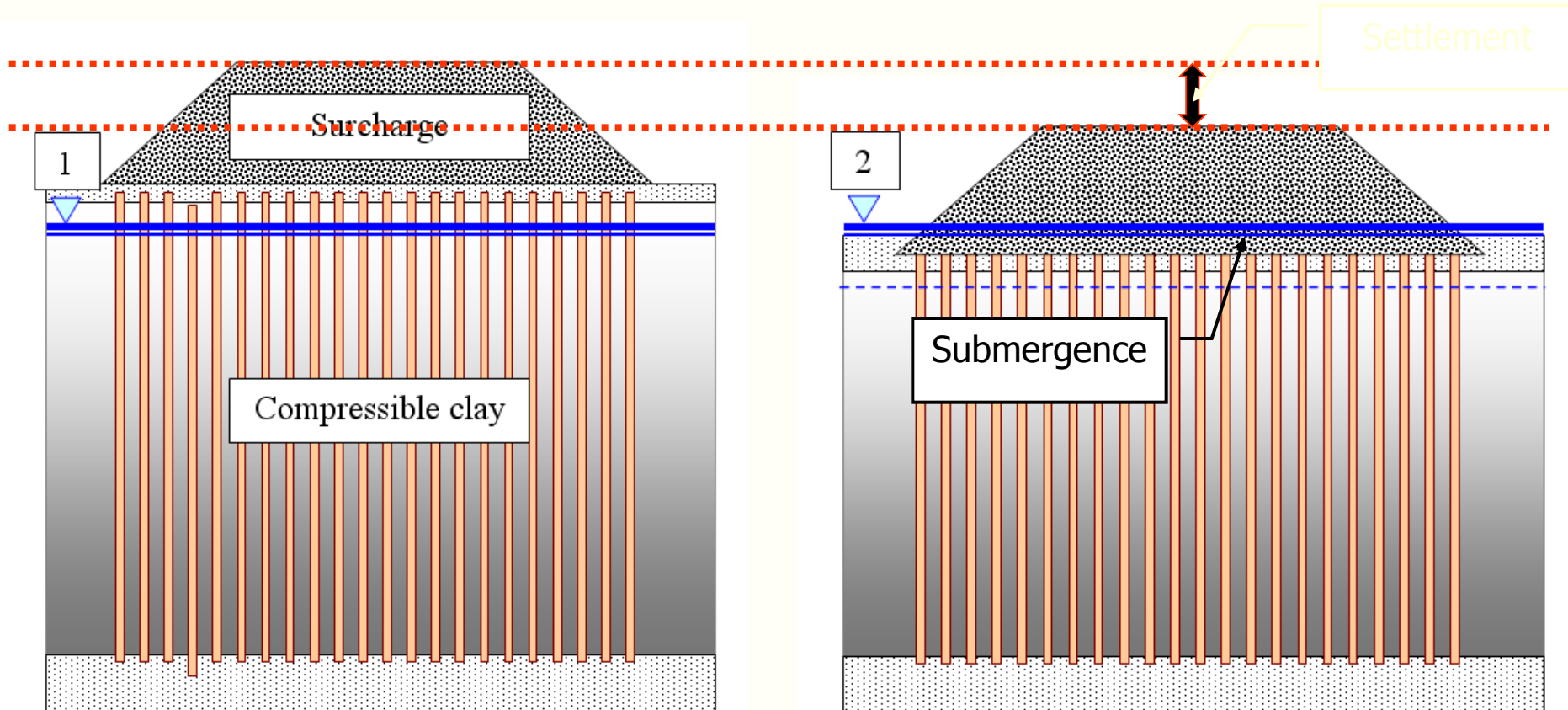
Preloading



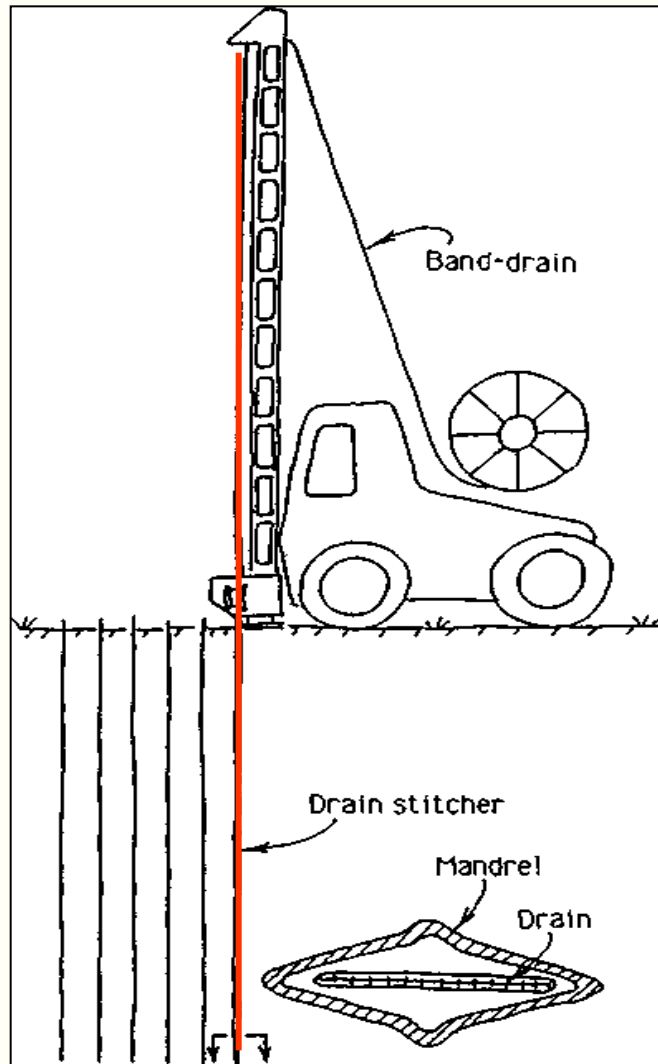
Surcharging Method – Step-loading



Drainage and Effect of Submergence



Vertical Drainage by PVD



















Band Drain (PVD) Installation



Drain Installation in Uppsala, Sweden



Execution of special geotechnical works - Vertical drainage

Exécution des travaux géotechniques spéciaux - Drains
verticaux

Ausführung von besonderen geotechnischen Arbeiten
(Spezialbau) - Vertikaldrainierung

This draft European Standard is submitted to CEN members for formal vote. It has been drawn up by the Technical Committee CEN/TC 235.

If this draft becomes a European Standard, CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration.

This draft European Standard was established by CEN in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the Management Centre has the same status as the official versions.

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EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

Management Centre: rue de Stassart, 35 B-1050 Brussels

European Standard

Vertical Drainage: prEN 15237

Foreword

1 Scope

2 Normative reference

3 Terms and definitions

4 Information needed for the execution of the work

5 Geotechnical investigations

6 Materials and products

7 Considerations related to design

8 Execution

9 Supervision and monitoring

10 Records

11 Special requirements

Scope

This European Standard establishes general principles for the execution, testing, supervision and monitoring of vertical drain projects.

The Standard includes the application of prefabricated vertical drains and sand drains and deals with requirements to be placed on design, drain material, installation methods and loading (static, vacuum, groundwater lowering).

This Standard applies to the improvement of low-permeability, highly compressible soils by vertical drainage and preloading.

Vertical drainage is used both on land and in marine constructions

- ❑ (pre-)consolidation and reduction of post-construction settlements;
- ❑ speeding up the consolidation process by decreasing the path lengths for pore water dissipation;
- ❑ increase of stability (by increasing effective stresses in the soil);
- ❑ groundwater lowering;
- ❑ mitigation of liquefaction effects.

Execution of vertical drainage

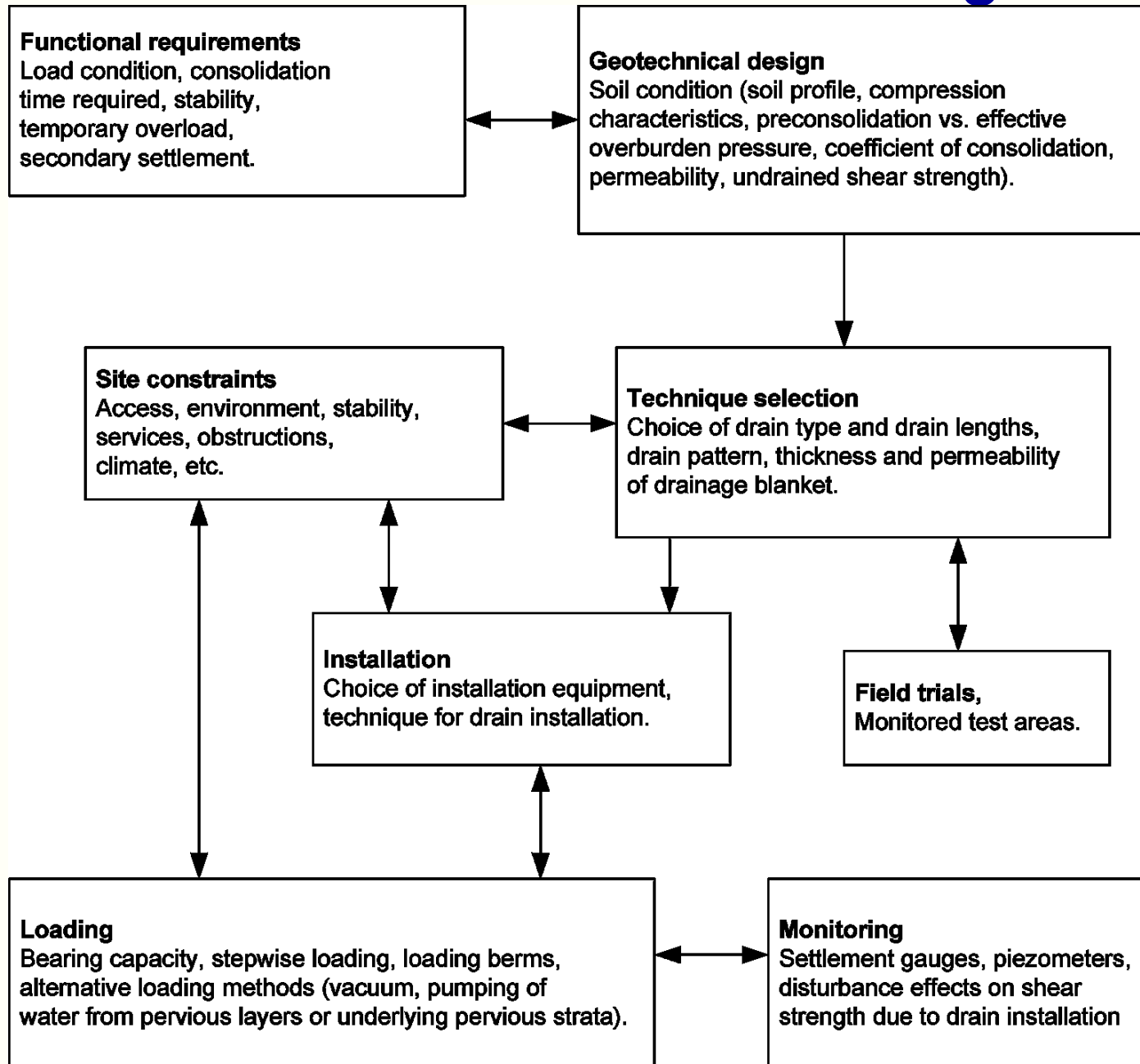
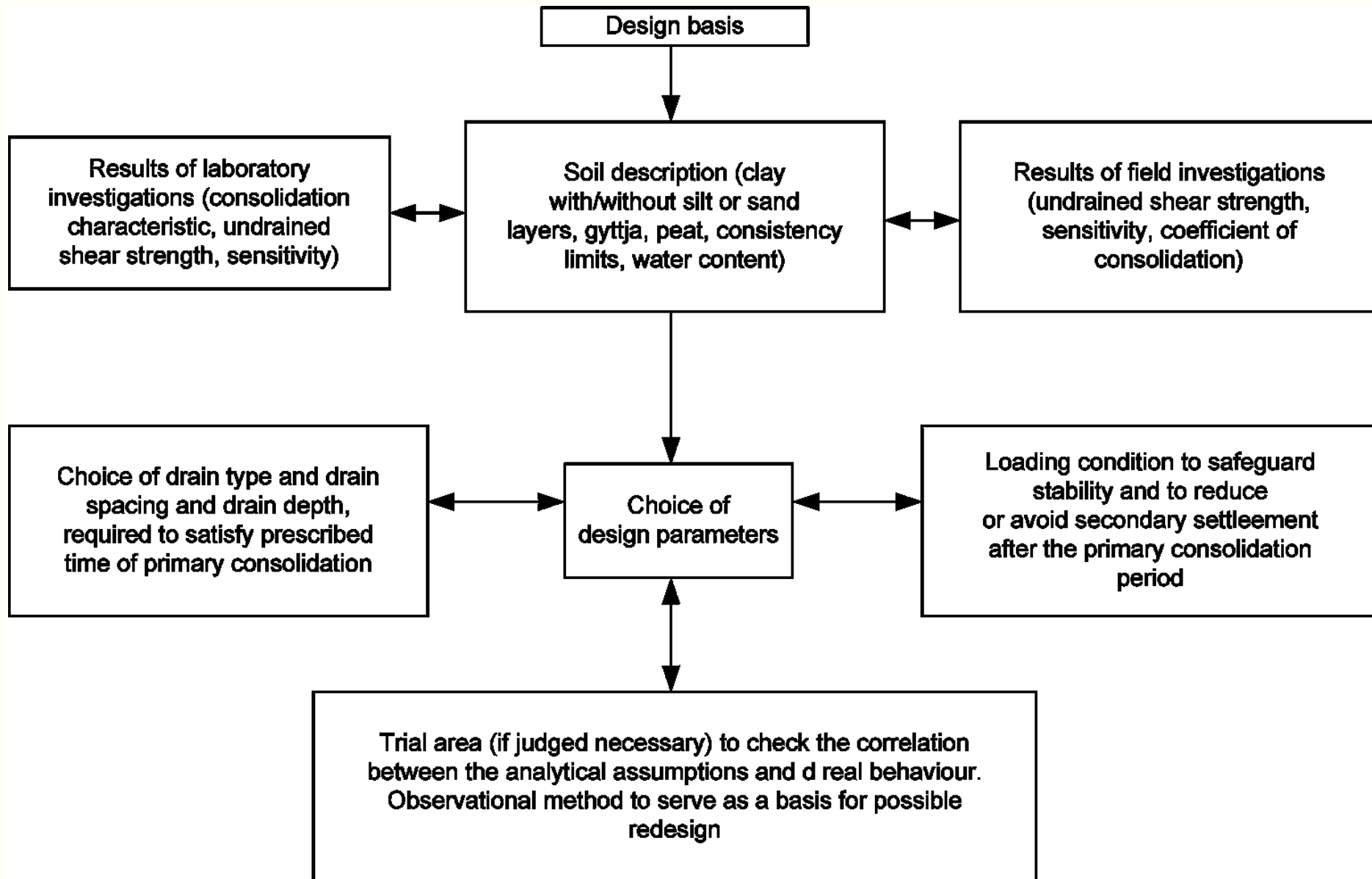
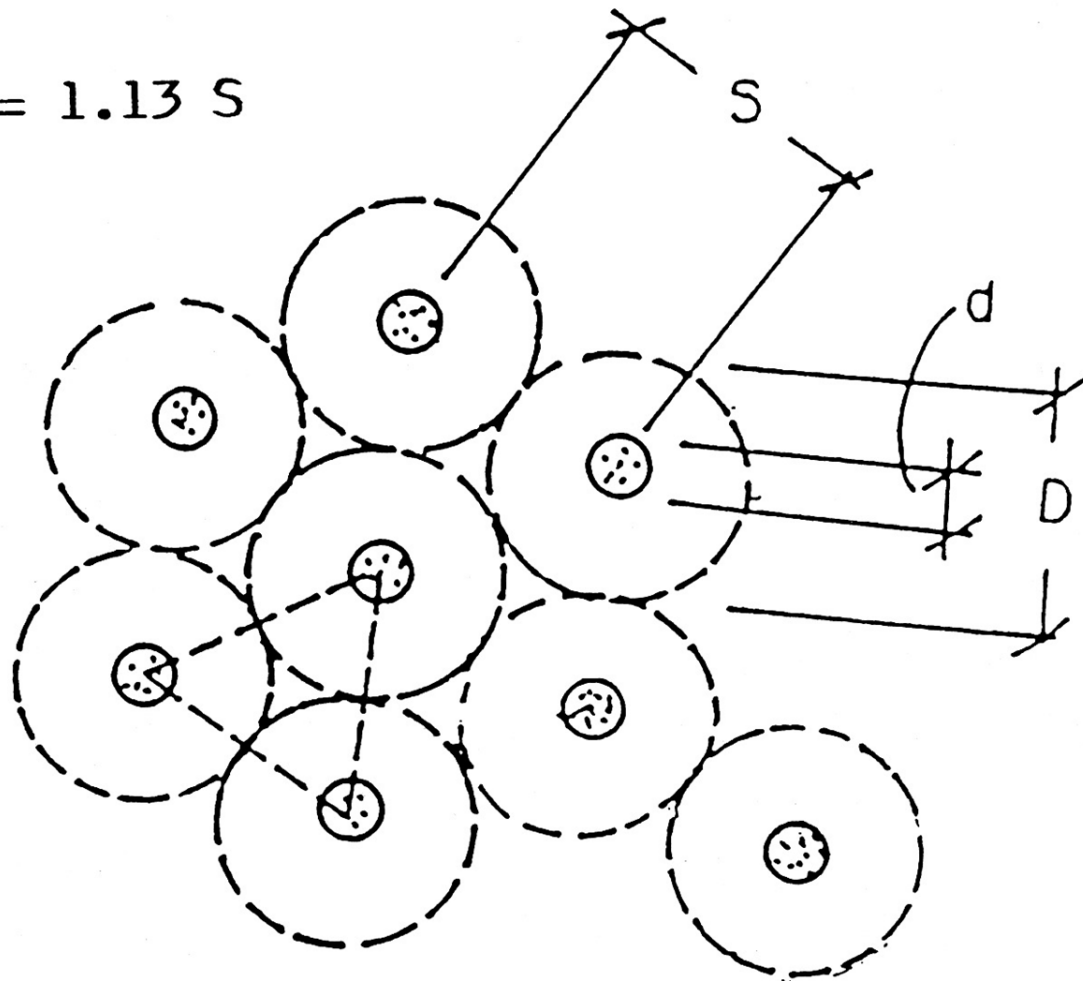


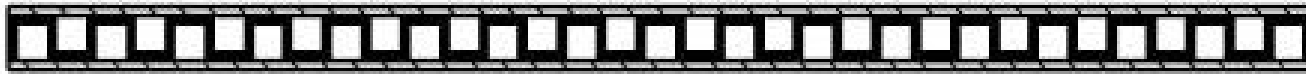
Chart of Design Process



Vertical Drains

$$D = 1.13 S$$





a) Channel-shaped core with glued filter



b) Channel-shaped core with wrapped filter

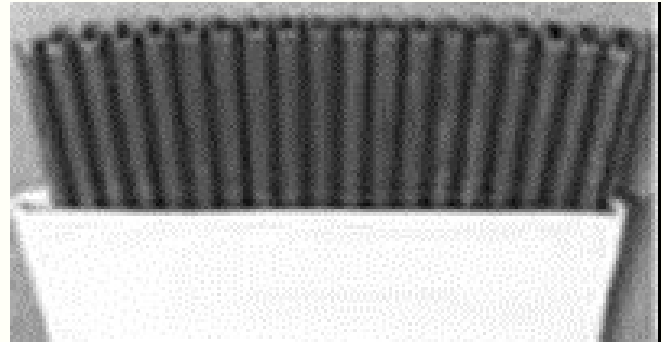
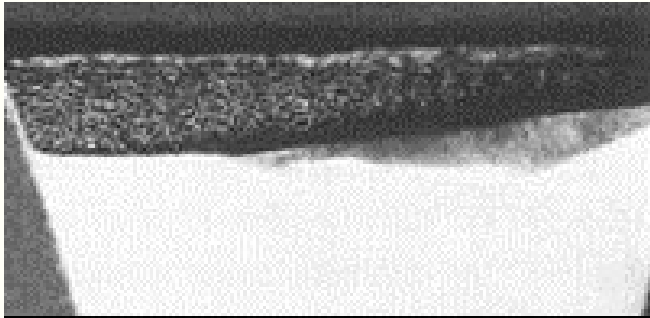
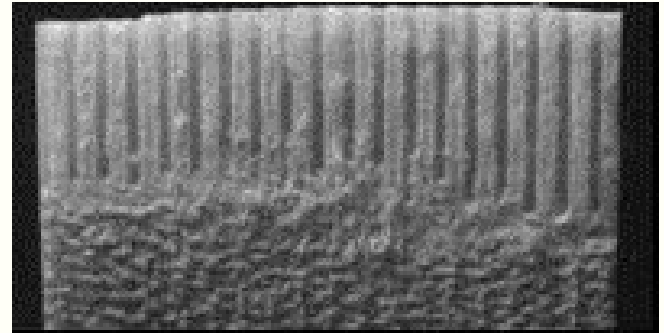
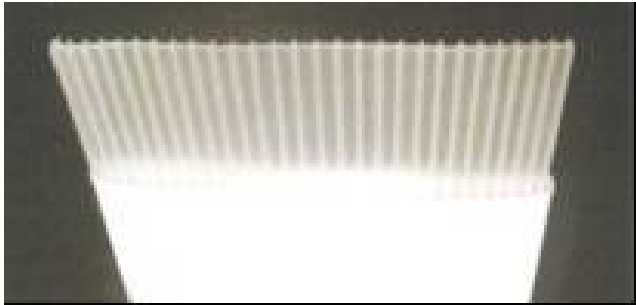


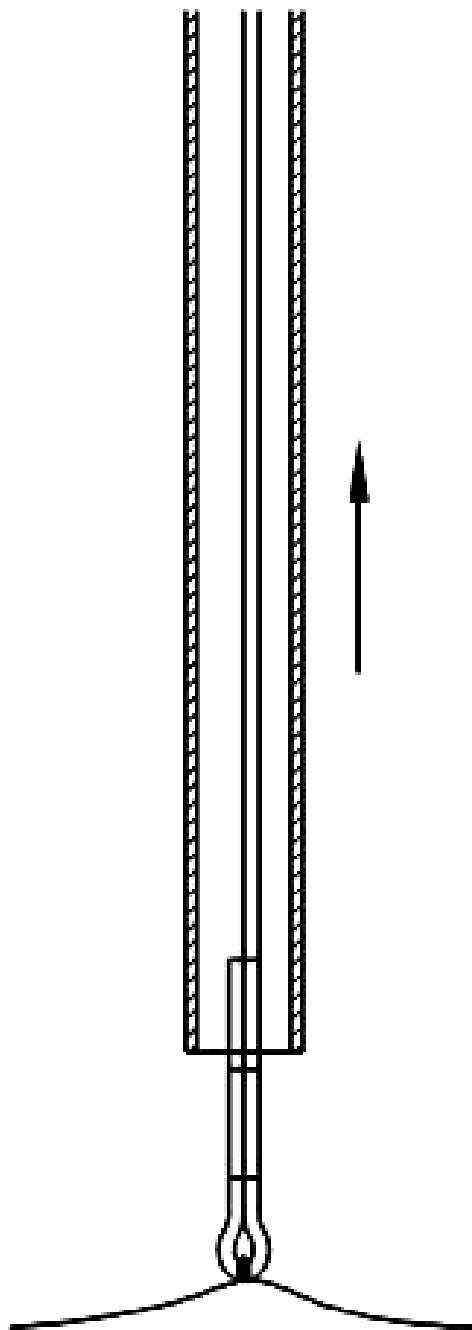
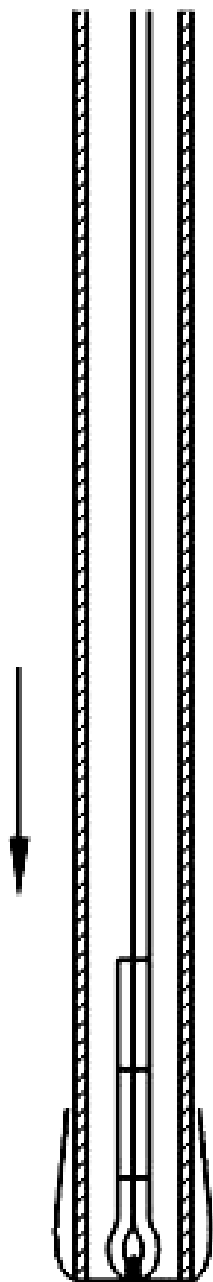
c) Geo-mat with edge-sealed filter



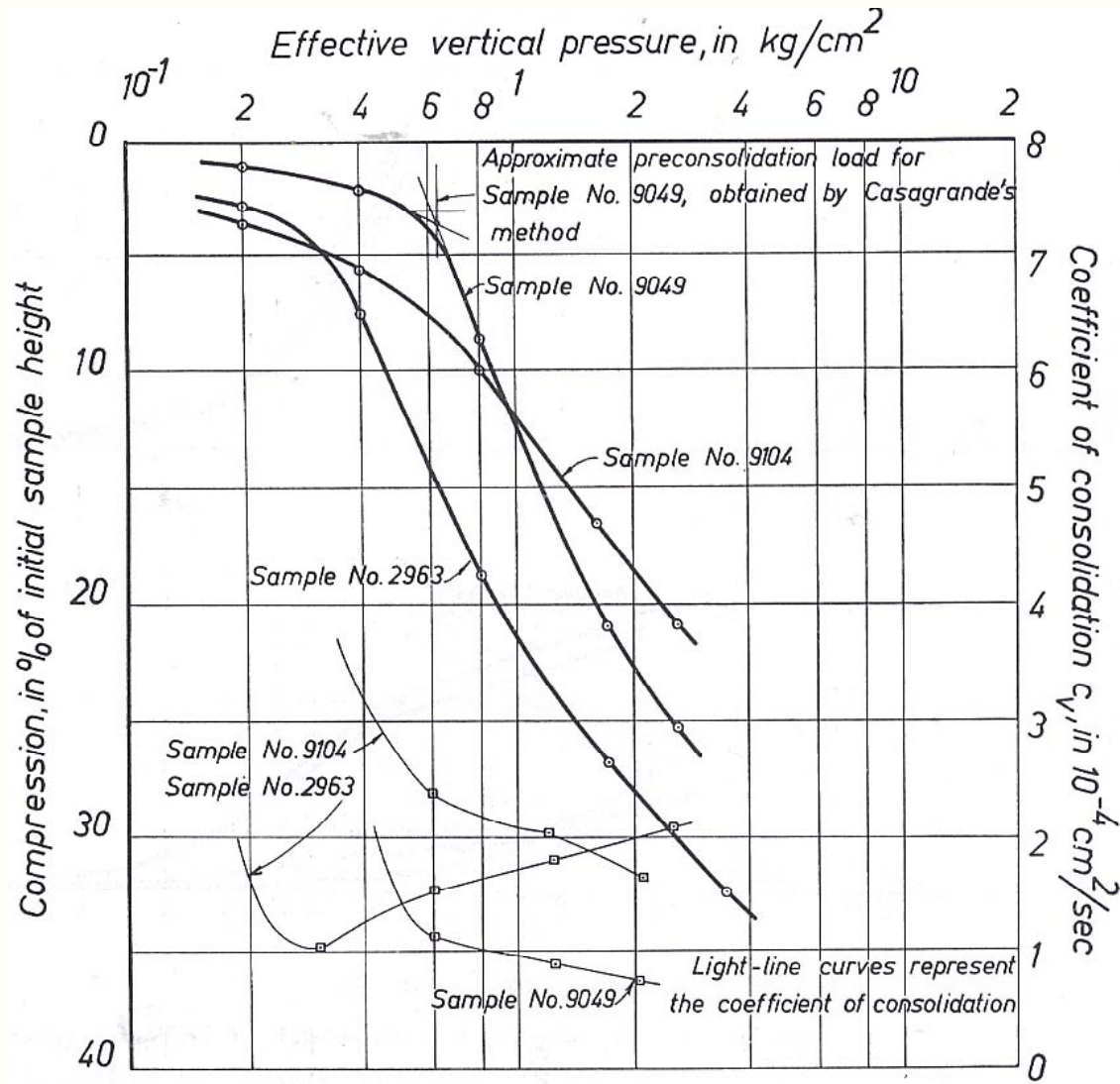
d) Cusp-shaped core with wrapped filter

Drain Types





Oedometer tests

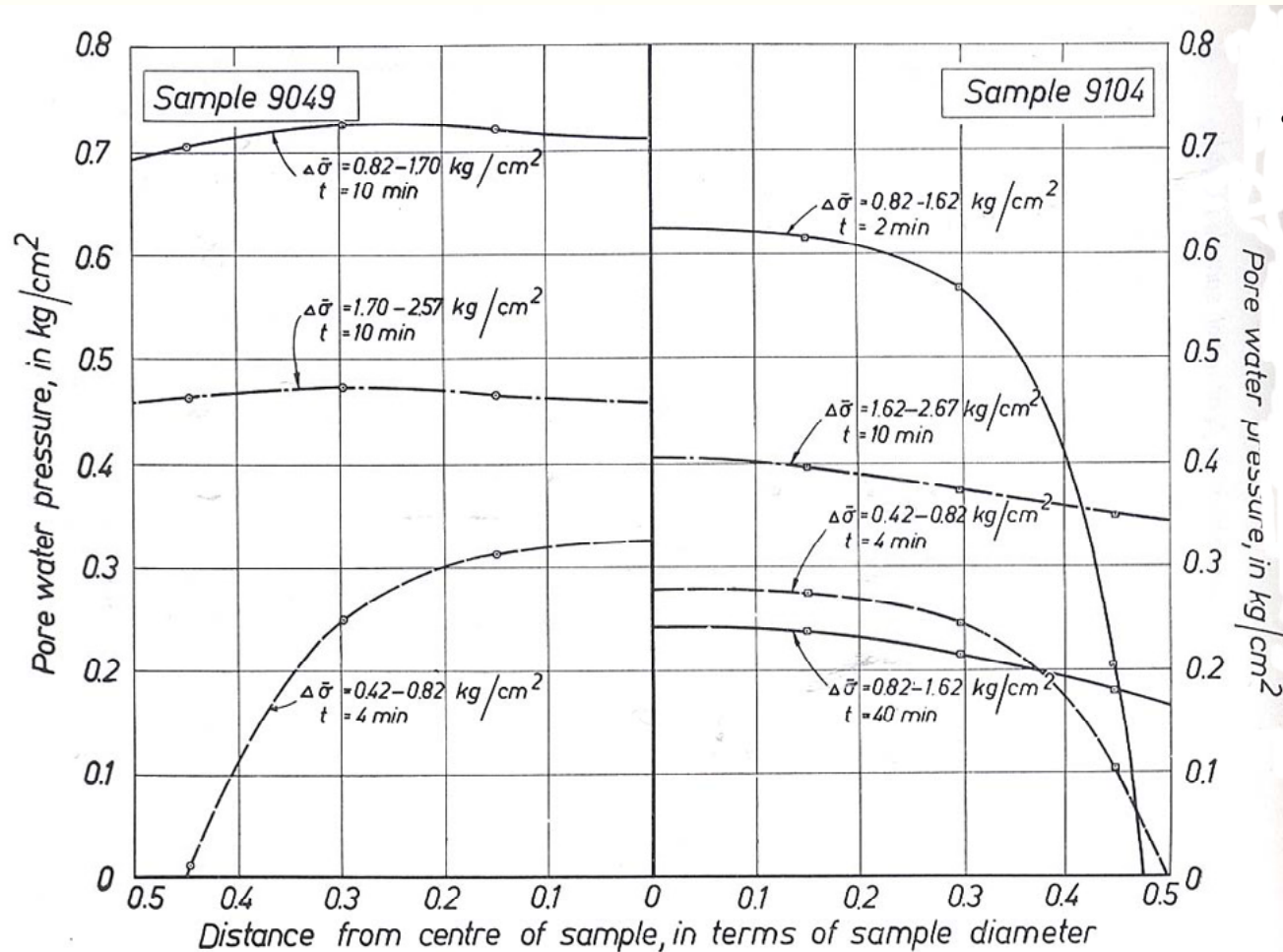


Consolidation characteristics of samples used in pore water pressure investigations. Sample 9104 disturbed (no sign of preconsolidation pressure).

Factors Affecting Drains

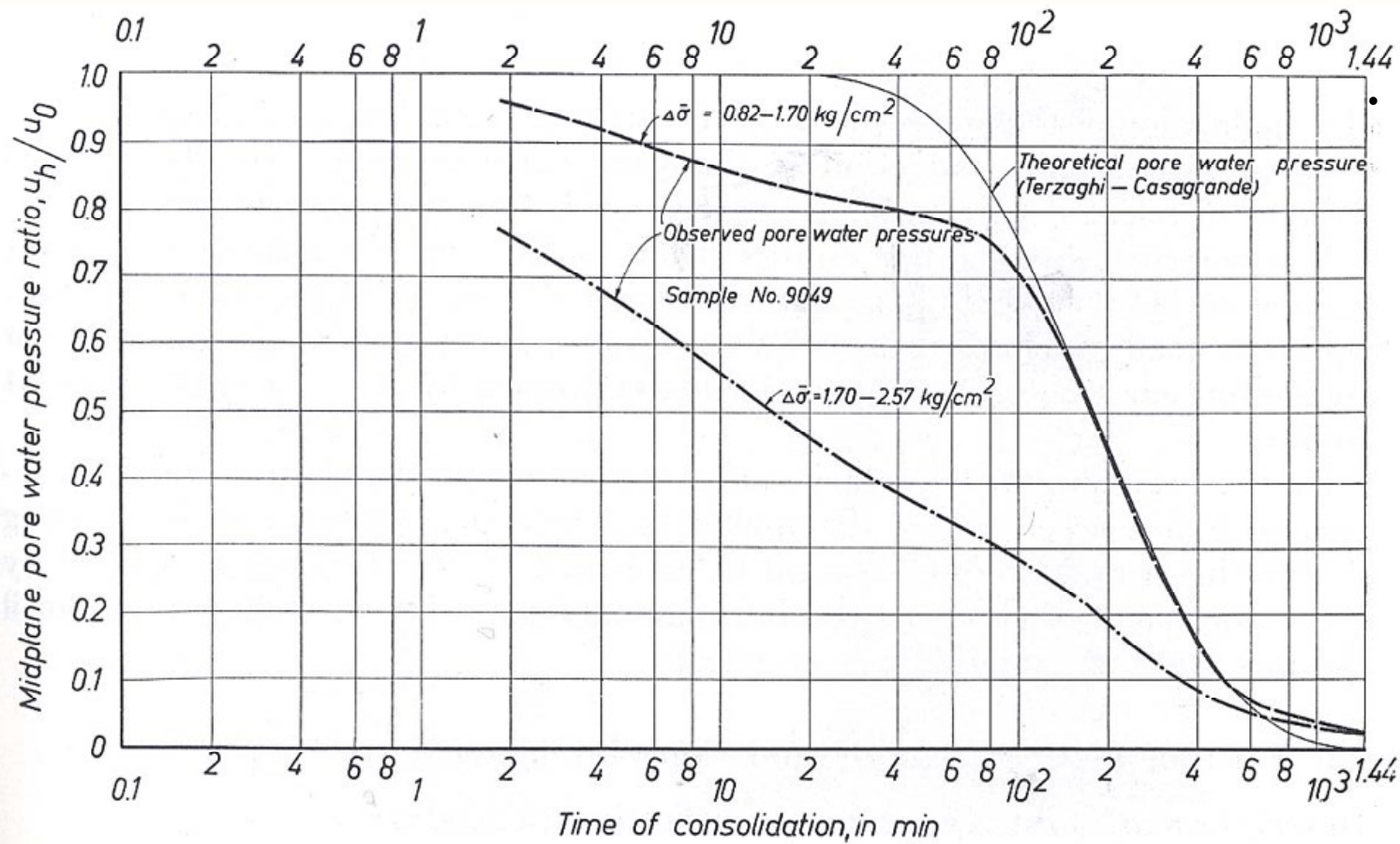
- Equivalent Drain Diameter
- Influence of one-dimensional consolidation
- Smear Effect
- Well Resistance
- Insufficient Depth of Installation
- Kinking (buckling) of drain

Oeometer tests



• Distribution of pore water pressure over impermeable base of the clay specimen at various times after beginning of the consolidation process. Sample 9049 undisturbed while sample 9104 disturbed

Oedometer tests

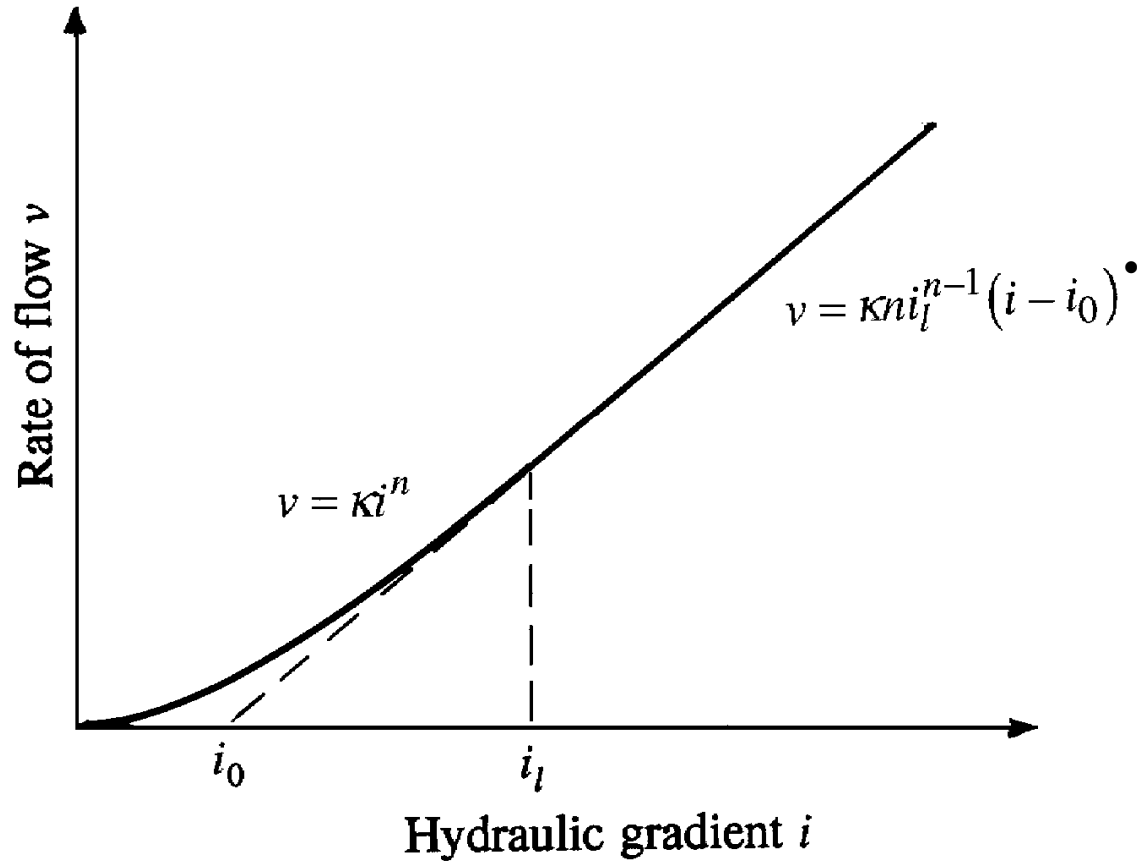


Observed pore water pressure dissipation compared with that determined from Terzaghi's consolidation theory

Flow conditions

- Darcian flow
- $v = ki$
- Non-Darcian flow
- $v = \kappa i^n$ when $i \leq i_l$ and $v = \kappa n i_l^{n-1} (i - i_0)$ when $i > i_l$, where $i_0 = i_l(n - 1)$,

FLOW CONDITIONS



Non-Darcian flow.
Assumed correlation
between rate of flow v
and hydraulic
gradient i .

Onedimensional consolidation

Darcian flow:

$$\frac{\partial u}{\partial t} = M \frac{\partial}{\partial z} \left(\frac{k}{\gamma_w} \frac{\partial u}{\partial z} \right)$$

where k is the permeability, M is the oedometer modulus and γ_w is the unit weight of water.

Assuming that $\kappa M / \gamma_w$ is independent of u , we have: $\frac{\partial u}{\partial t} = \frac{kM}{\gamma_w} \frac{\partial^2 u}{\partial z^2} = c_v \frac{\partial^2 u}{\partial z^2}$

Onedimensional consolidation

Non-Darcian flow

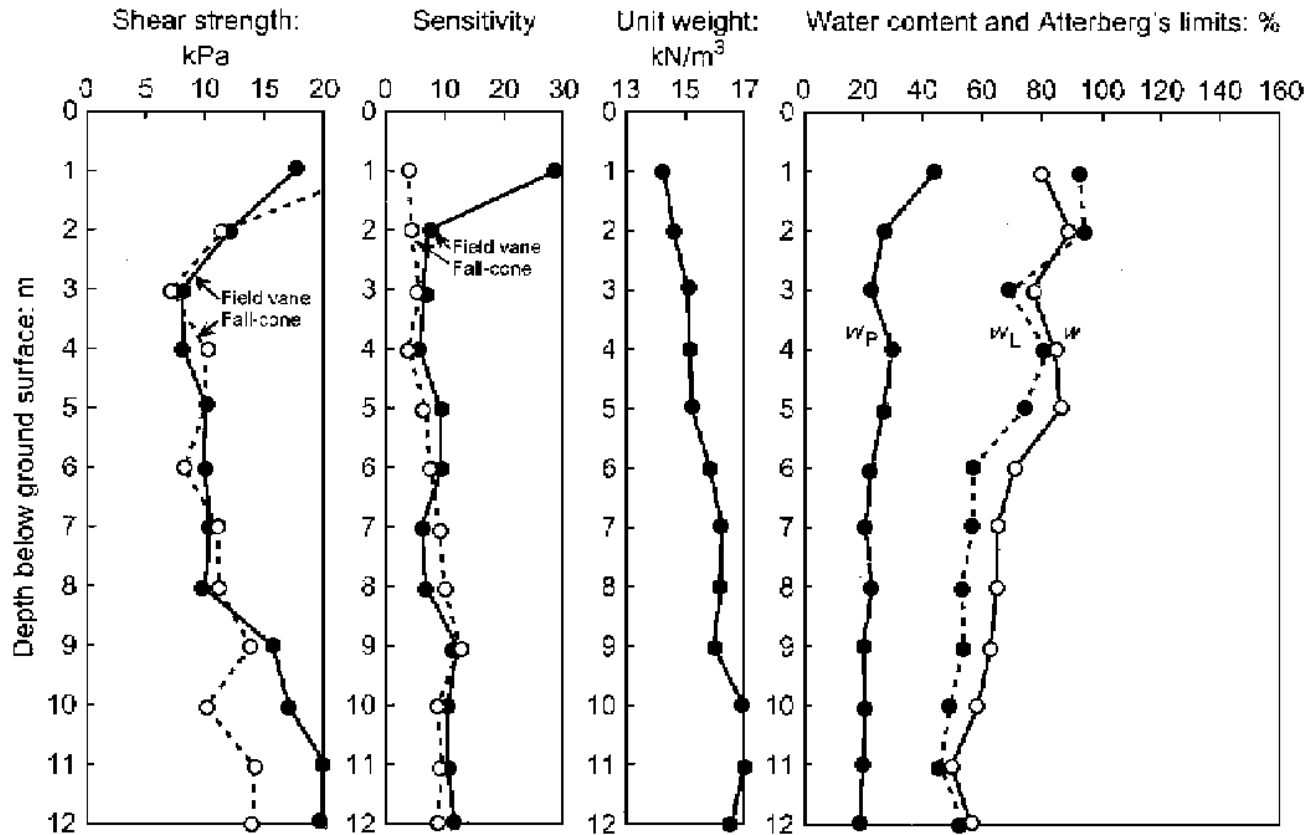
When $(i \leq i_l)$, we have $v = \kappa i^n = \kappa \left(\frac{1}{\gamma_w} \frac{\partial u}{\partial z} \right)^n$ and $\frac{\partial v}{\partial z} = \frac{1}{M} \frac{\partial u}{\partial t}$.

Assuming that $\kappa M / \gamma_w$ is independent of u , we have: $\frac{\partial u}{\partial t} = \frac{\kappa M n}{(\gamma_w)^n} \left(\frac{\partial u}{\partial z} \right)^{n-1} \frac{\partial^2 u}{\partial z^2}$

When $i > i_l$, $v = \kappa n i_l^{n-1} \left(\frac{1}{\gamma_w} \frac{\partial u}{\partial z} - i_0 \right)$ and $\frac{\partial v}{\partial z} = \frac{1}{M} \frac{\partial u}{\partial t}$, which yields: $\frac{\partial u}{\partial t} = \frac{\kappa M n}{\gamma_w} i_l^{n-1} \frac{\partial^2 u}{\partial z^2}$

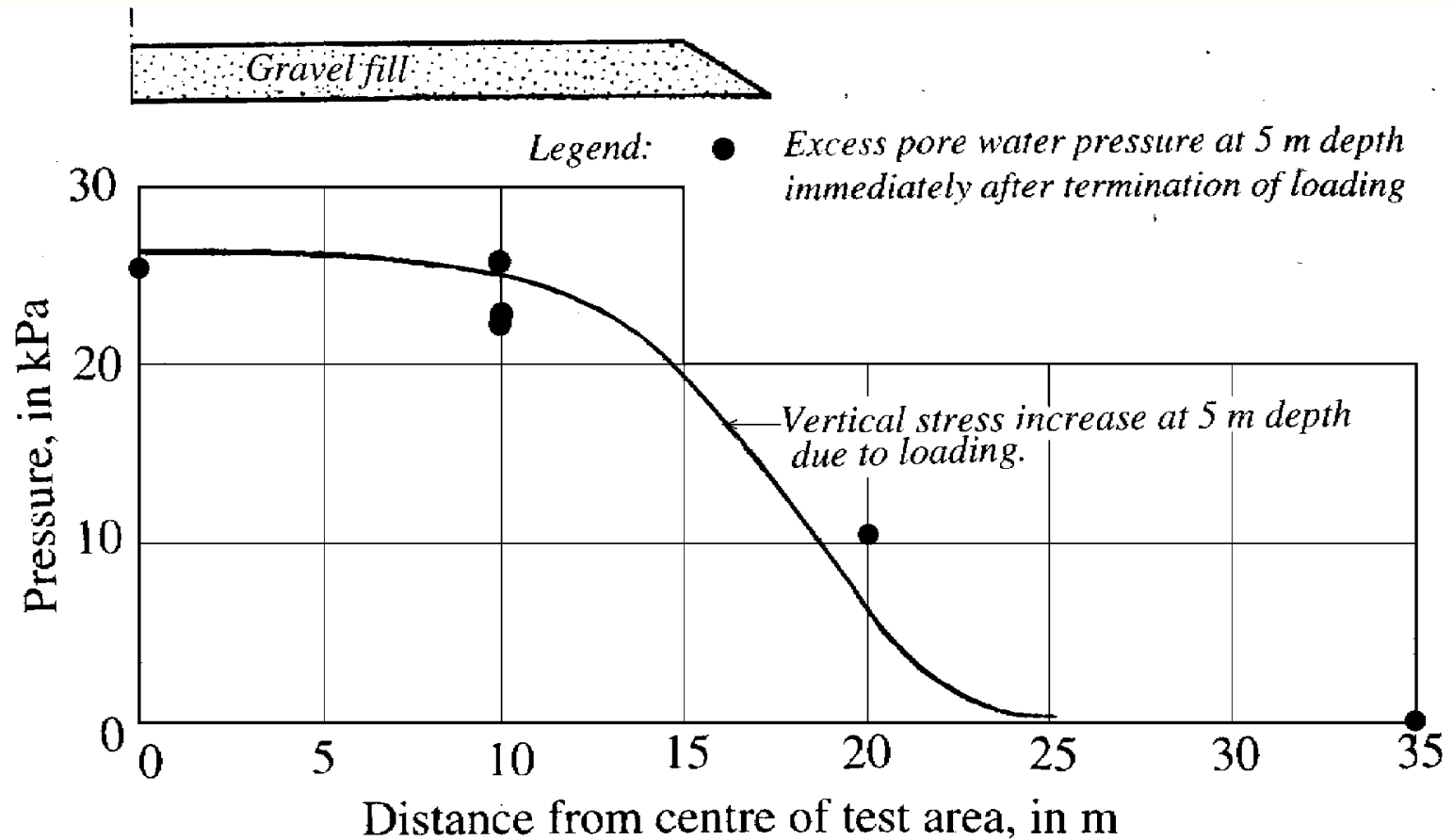
THEORY VS. REALITY IN ONE-DIMENSIONAL CONSOLIDATION

Test area at Skår Edeby, 3.5 m in diameter, overload 27 kN/m^2



Geotechnical properties of the clay subsoil in Test Area IV (Hansbo, 1960)

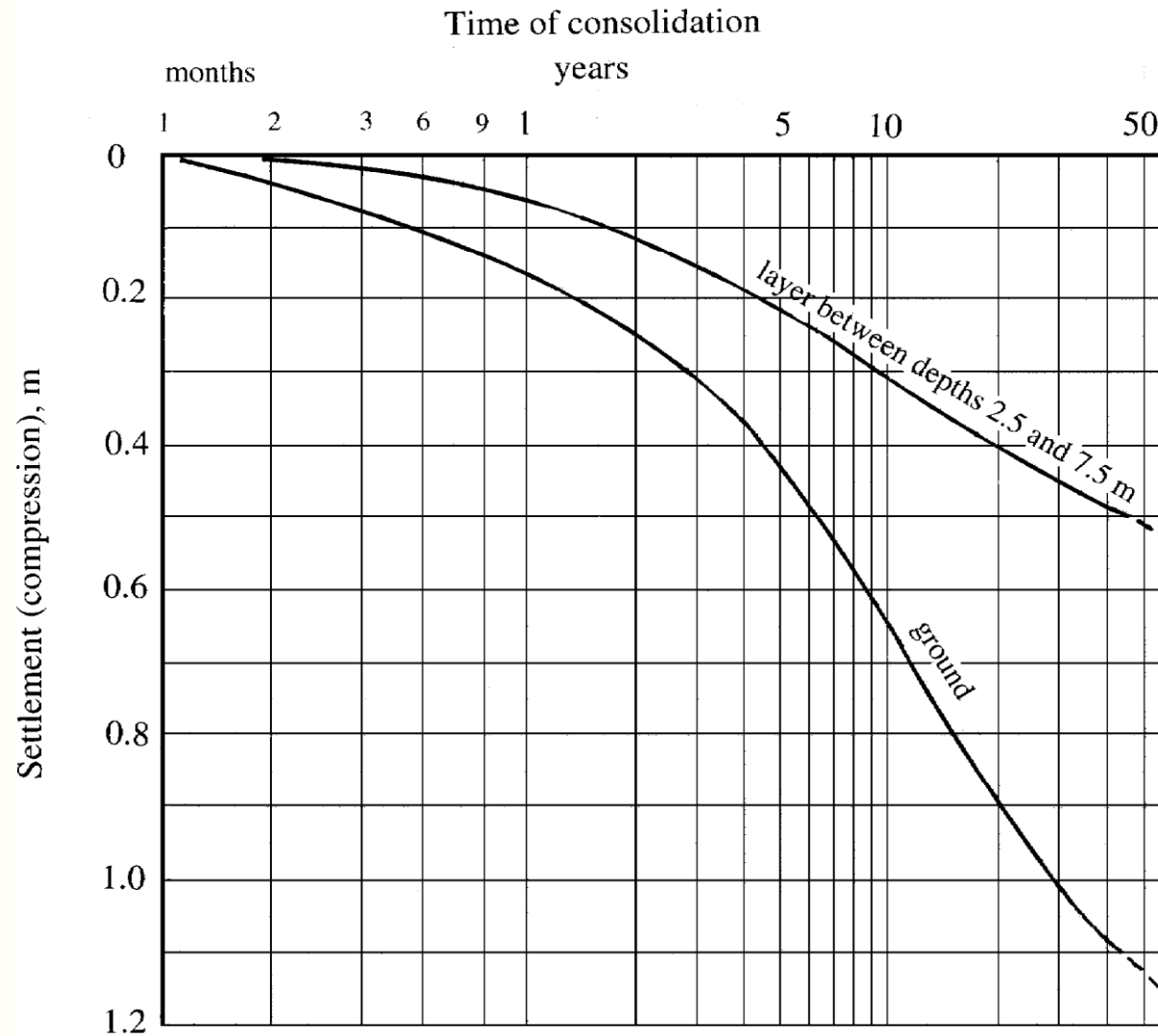
Observed excess pore pressure distribution in Test area IV at 5 m depth, immediately after placement of the gravel fill.



Observed excess pore pressure distribution in Test area IV at 5 m depth, immediately after placement of the gravel fill.

- This can be used in determining the parameters A and B in Skempton's pore pressure equation $u = B[\sigma_3 + A(\sigma_1 - \sigma_3)]$ where σ_1 and σ_3 are the major and minor principle stresses induced by loading (Skempton, 1954). As we are dealing with water saturated clay, $B = 1$. According to the solution presented by Love (1929) $\sigma_1 = 0.98q$ and $\sigma_3 = 0.60q$ below the centre, $\sigma_1 = 0.95q$ and $\sigma_3 = 0.49q$ 10 m from the centre, and $\sigma_1 = 0.50q$ and $\sigma_3 = 0.06q$ 20 m from the centre. Inserting $q = 27$ kN/m² and considering the observations made this yields $A \approx 0.85$.

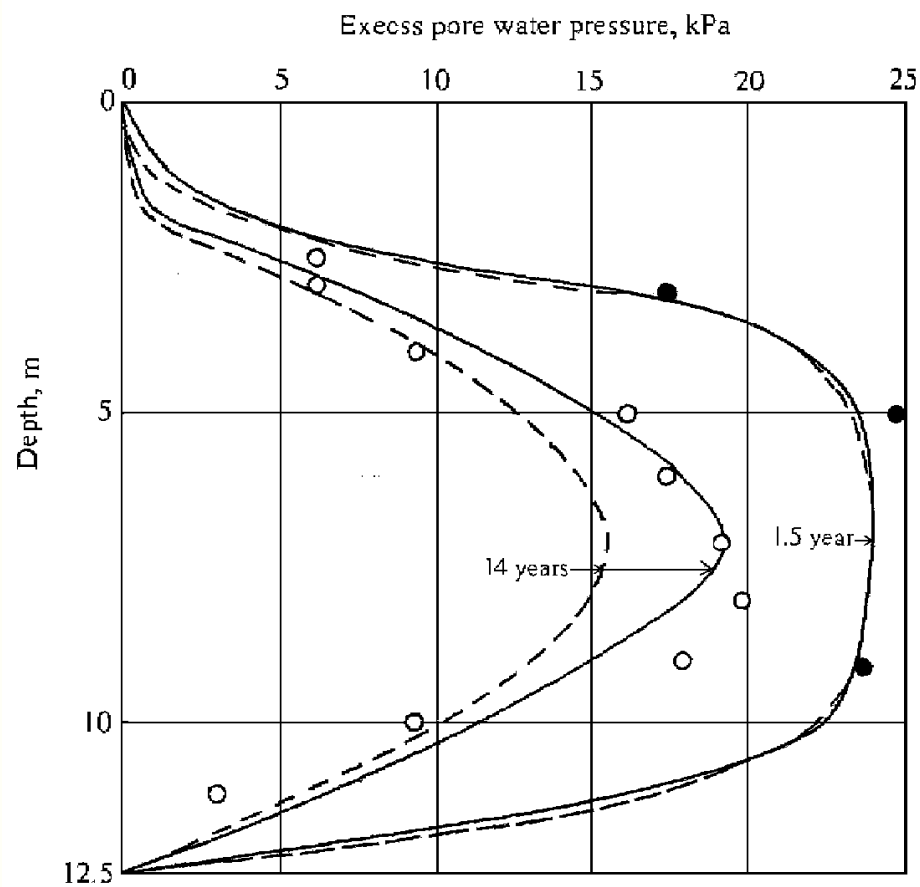
Consolidation settlement of ground surface and compression of clay layer between 2.5 and 7.5 m of depth observed in Test Area IV.



Consolidation settlement of ground surface and compression of clay layer between 2.5 and 7.5 m of depth observed in Test Area IV.

- According to Asaoka (1978), the total primary consolidation settlement s_p can be estimated from the relation $s_i = 0.300 + 0.761s_{i-1}$, which yields $s_p = 1.26$ m ($s_{i-1} = s_i$) and the compression Δs_p of the clay layer between 2.5 and 7.5 m of depth from the relation $\Delta s_i = 0.119 + 0.815\Delta s_{i-1}$, which yields $\Delta s_p = 0.64$ m.

Comparison between observed excess pore pressure years dissipation in Test Area IV, Skå-Edeby, and analytical dissipation according to Darcian flow (broken lines) and non-Darcian flow (unbroken lines) after 1.5 and 14 years



Observed excess pore water pressure:

● 1.5 year of consolidation

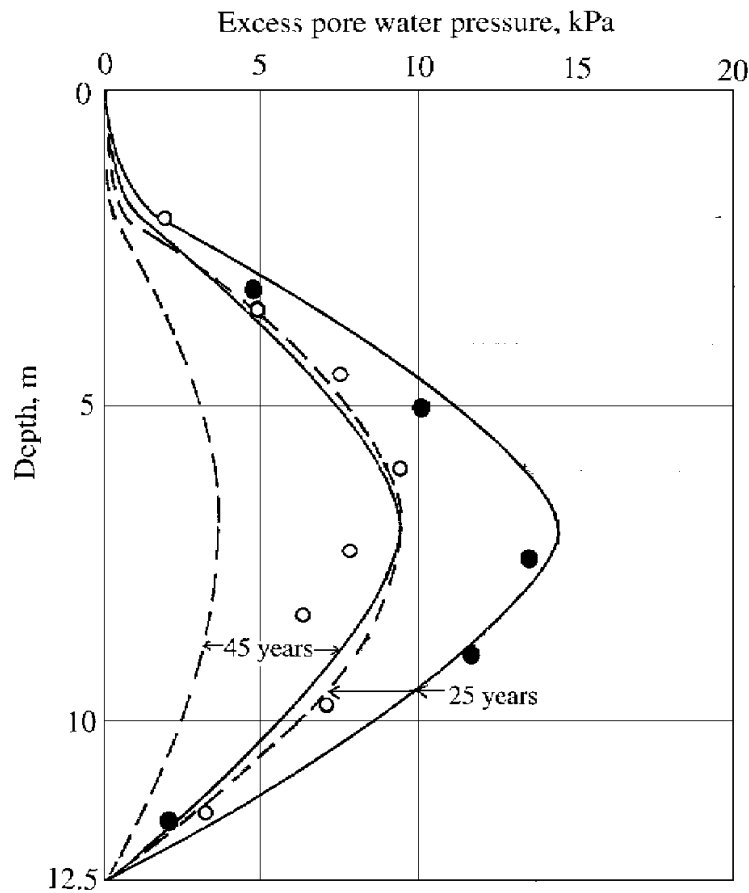
○ 14 years of consolidation

Test Area IV, Skå-Edeby

Parameters used in the consolidation analysis

Depth, m	0Š1.0	1.0Š1.5	1.5Š3	3Š5	5Š7	7Š9	9Š11	11Š12.5
$E_z \sigma_v$ kPa	27	27	27	26.5	26.5	26	26	25
σ'_c kPa	OC	25	23	28	39	51	64	75
M kPa	10000	400	250	240	250	300	400	500
E_s , m	0.01	0.03	0.16	0.24	0.24	0.23	0.22	0.15
k m/year	0.031	0.031	0.025	0.022	0.018	0.018	0.017	0.016
κ m/year	0.020	0.020	0.016	0.014	0.0115	0.0115	0.011	0.0095

Test Area IV, Skå-Edeby



Observed excess pore water pressure:

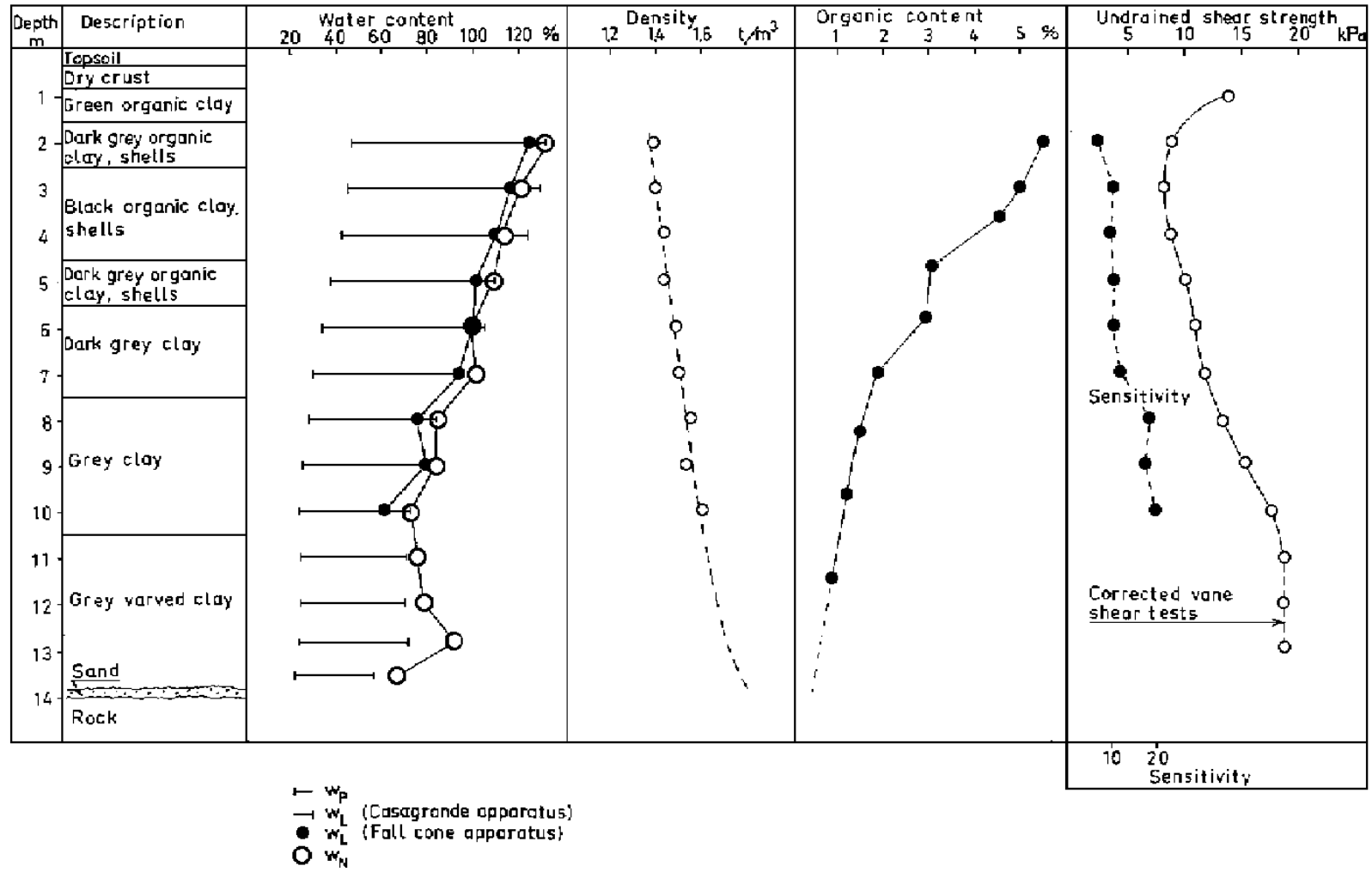
● 25 years of consolidation

○ 45 years of consolidation

- Looking at the remaining excess pore water pressure after 25 and 45 years of consolidation, we find the average degree of consolidation equal to, respectively, 69% and 79%. This leads in both cases to a total consolidation settlement of 1.39 m, about 10% larger than the estimated primary consolidation settlement. For the layer between the depths 2.5 m and 7.5 m, the average degree of consolidation after 25 and 45 years of consolidation becomes, respectively, 62% and 73%, which in both cases would lead to a primary compression of 0.69 m, about 8% larger than the estimated compression according to Asaoka.

The Lilla Mellösa test area

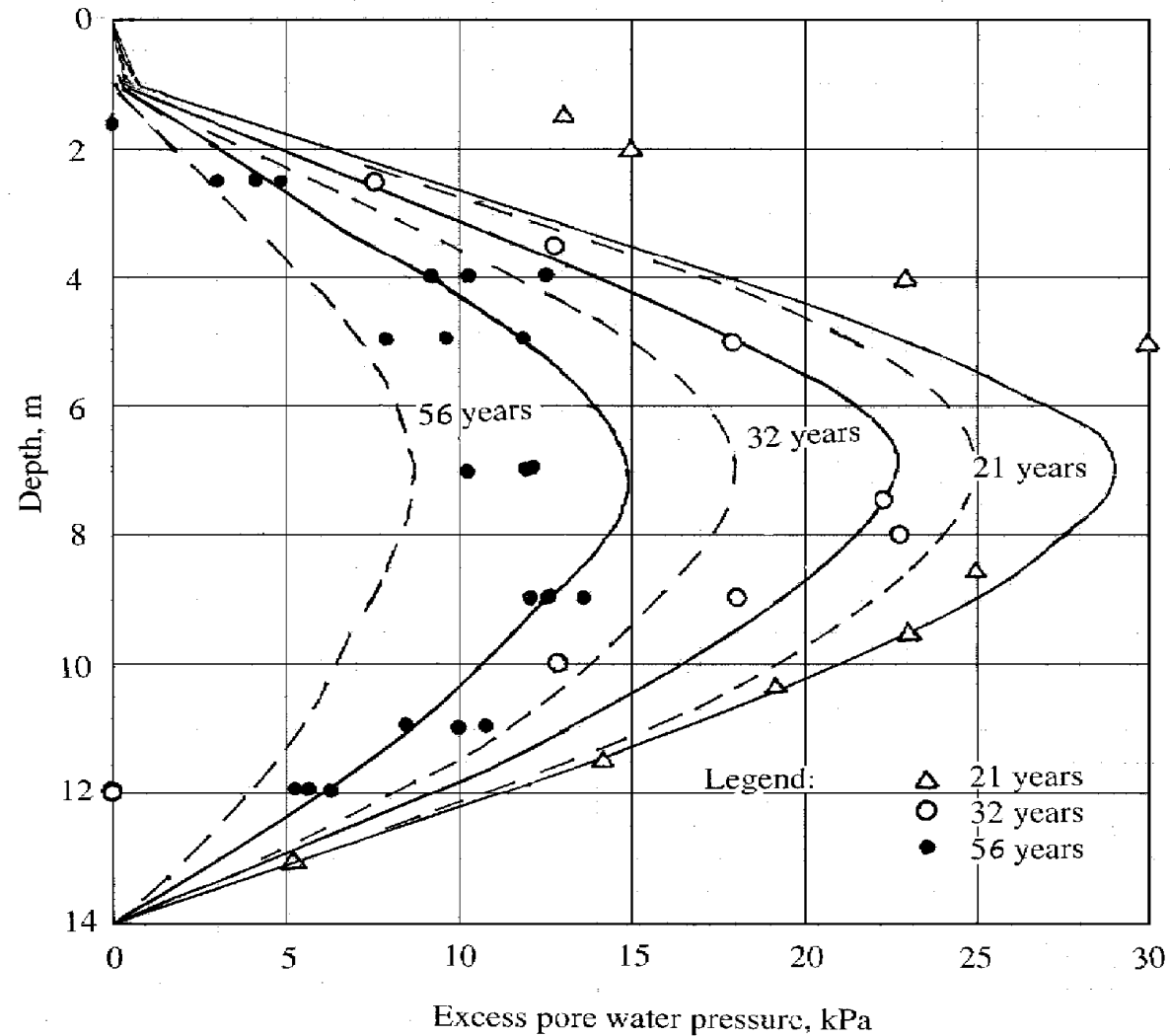
Soil characteristics at the Lilla Mellösa test site



Lilla Mellösa test site

- The test area is square with a base width of 30 m. It was filled up with 2.5 m of gravel, corresponding to an overload of 45 kN/m². The top width of the fill is 22.5 m.
- We do not know the magnitude of the pore pressure coefficient A in Skempton's pore pressure equation. As in the case of the Skå-Edeby test area, we are dealing with water saturated clay, and thus $B = 1$. In our case, A is also assumed equal to 1. In consequence, the initial excess pore water pressure has been considered equal to the vertical stress increase caused by the load, decreasing almost linearly with depth from 42 kPa at the ground surface to 32 kPa at the bottom of the clay layer.

Comparison between observed excess pore pressure dissipation in the undrained test area at Lilla Mellösa

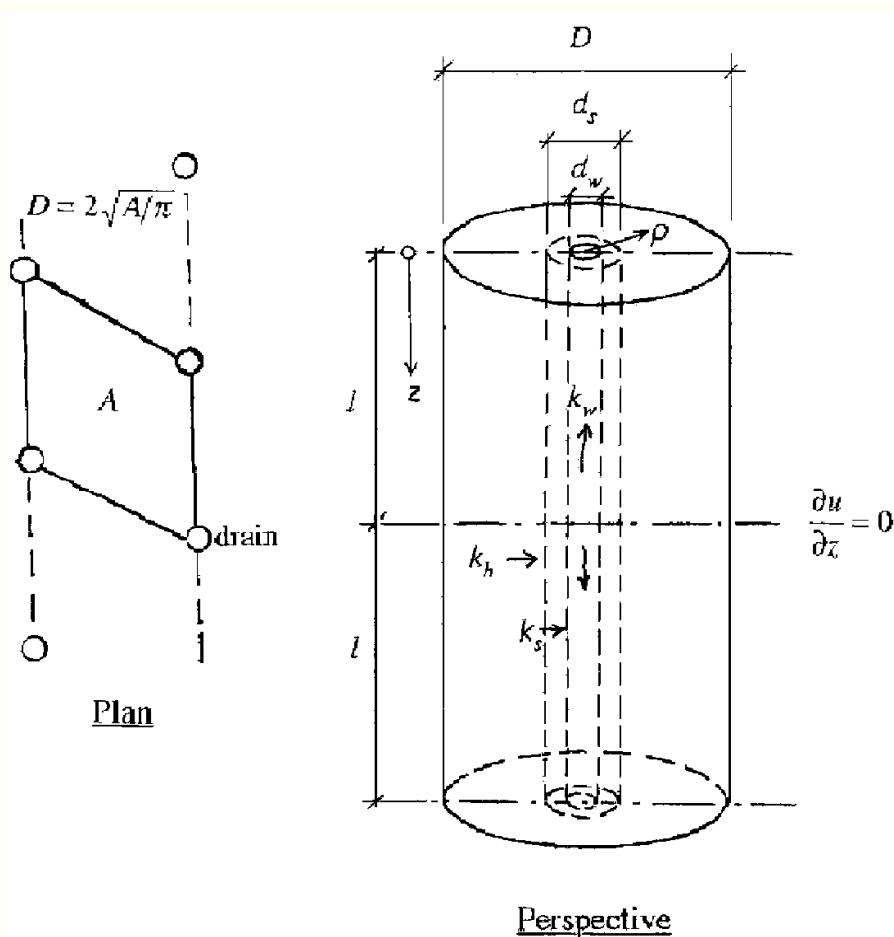


The Lilla mellösa test site

Parameters used in the consolidation analysis

Depth: m	0 Š 1.0	1.0 Š 1.5	1.5 - 3	3 - 5	5 - 7	7 - 9	9 - 11	11 - 14
M kPa	9000	200	200	190	160	240	280	200
$\Delta \sigma_v$ kPa	42	42	41	41	40	39	38	33
E_s m	0.01	0.10	0.31	0.43	0.50	0.32	0.27	0.50
k m/year	0.020	0.020	0.025	0.018	0.015	0.022	0.03	0.03
κ m/year	0.013	0.013	0.017	0.012	0.010	0.015	0.02	0.02

Vertical drainage



- Terms used in the analysis of vertical drains: D = diameter of soil cylinder dewatered by a drain; d_w = drain diameter; d_s = diameter of zone of smear; l = length of drain when closed at bottom ($2l$ = length of drain when open at bottom); z = depth coordinate; k_w = permeability in the longitudinal direction of the drain; k_h = permeability (in the horizontal direction) of soil; k_s = permeability (in the horizontal direction) of zone of smear; ρ = radius vector.

Darcian flow

Average degree of consolidation

The solution to the consolidation problem is greatly simplified if one assumes that horizontal sections due to arching remain horizontal throughout the consolidation process. This is the so-called *equal strain theory* (Barron, 1944, 1947).

The solution based on Darcian flow can be expressed by the relation (Hansbo, 1981):

$$\bar{U}_h = 1 - \exp\left(-\frac{8c_h t}{\mu D^2}\right)$$
$$\mu = \frac{D^2}{D^2 - d_w^2} \left[\ln\left(\frac{D}{d_s}\right) + \frac{k_h}{k_s} \ln\left(\frac{d_s}{d_w}\right) - \frac{3}{4} \right] + \frac{d_s^2}{D^2 - d_w^2} \left(1 - \frac{d_s^2}{4D^2} \right) +$$
$$+ \frac{k_h d_w^2}{k_s (D^2 - d_w^2)} \left(\frac{d_s^4 - d_w^4}{4D^2 d_w^2} - \frac{d_s^2}{d_w^2} + 1 \right) + \frac{k_h}{q_w} \pi z (2l - z) \left(1 - \frac{d_w^2}{D^2} \right)$$
$$c_h = k_h M / \gamma_w$$

This gives results that are in good agreement with those obtained by advanced analytical and numerical solutions (e.g. Yoshikuni & Nakanado, 1974; Onoue, 1988; Zeng & Xie, 1989; Lo, 1991).

Darcian flow

Excess pore pressure variation. Replacing the initial excess pore water pressure \bar{u}_0 with $\gamma_w \Delta \bar{h}_0$, where $\Delta \bar{h}_0$ is the initial average hydraulic head increase, the variation of the hydraulic head increase Δh outside the zone of smear ($D/2 \geq \rho \geq d_s/2$), based on validity of Darcy's law, becomes:

$$\Delta h = \frac{\Delta \bar{h}_0}{\mu D^2} (1 - \bar{U}_h) \left[D^2 \ln \frac{2\rho}{d_s} - \frac{4\rho^2 - d_s^2}{2} + \frac{k_h}{k_s} \left(D^2 \ln \frac{d_s}{d_w} - \frac{d_s^2 - d_w^2}{2} \right) + \frac{k_h}{q_w} \pi z (2l - z) (D^2 - d_w^2) \right]$$

Inside the zone of smear ($d_s/2 \geq \rho \geq d_w/2$) we have:

$$\Delta h = \frac{\Delta \bar{h}_0}{\mu D^2} (1 - \bar{U}_h) \left[\frac{k_h}{k_s} \left(D^2 \ln \frac{2\rho}{d_s} - \frac{4\rho^2 - d_s^2}{2} \right) + \frac{k_h}{q_w} \pi z (2l - z) (D^2 - d_w^2) \right]$$

Darcian flow

Hydraulic gradient. The hydraulic gradient in Darcian flow outside the zone of smear ($D/2 \geq \rho \geq d_s/2$) becomes:

$$i = \frac{\Delta \bar{h}_0}{D} (1 - \bar{U}_h) \frac{1}{\mu} \left(\frac{D}{\rho} - \frac{4\rho}{D} \right)$$

and inside the zone of smear ($d_s/2 \geq \rho \geq d_w/2$):

$$i = \frac{\Delta \bar{h}_0}{D} (1 - \bar{U}_h) \frac{k_h/k_s}{\mu} \left(\frac{D}{\rho} - \frac{4\rho}{D} \right)$$

Exponential flow. Average degree of consolidation

Average degree of consolidation

Assuming an exponential correlation between hydraulic gradient and flow velocity, $v = \kappa i^n$, the consolidation equation becomes (Hansbo, 1997a-b):

$$\bar{U}_h = 1 - \left[1 + \frac{\lambda t}{\alpha D^2} \left(\frac{\Delta \bar{h}_0}{D} \right)^{n-1} \right]^{\frac{1}{1-n}}$$

where $\Delta \bar{h}_0 = \bar{u}_0 / \gamma_w$ (\bar{u}_0 = initial excess pore water pressure, equally distributed), $\lambda = \kappa_h M / \gamma_w$, t = time of consolidation, M = compression modulus determined by oedometer tests ($= 1/m_v$, where m_v is the volume compressibility), γ_w = unit weight of water, $\alpha = n^{2n} \beta^n / [4(n-1)^{n+1}]$

Exponential flow, β

$$\begin{aligned}
 \beta = & \frac{1}{3n-1} - \frac{n-1}{n(3n-1)(5n-1)} - \frac{(n-1)^2}{2n^2(5n-1)(7n-1)} + \\
 & + \frac{1}{2n} \left[\left(\frac{\kappa_h}{\kappa_s} - 1 \right) \left(\frac{D}{d_s} \right)^{(1/n-1)} - \frac{\kappa_h}{\kappa_s} \left(\left(\frac{D}{d_w} \right)^{(1/n-1)} \right) \right] - \\
 & - \left(\frac{1}{2n} - \frac{1}{3n-1} \right) \left[\left(\frac{\kappa_h}{\kappa_s} - 1 \right) \left(\frac{D}{d_s} \right)^{(1/n-3)} - \frac{\kappa_h}{\kappa_s} \left(\frac{D}{d_w} \right)^{(1/n-3)} \right] + \\
 & + \frac{\kappa_h}{2q_w} \pi z (2l - z) \left(1 - \frac{1}{n} \right) \left(\frac{D}{d_w} \right)^{(1/n-1)} \left(1 - \frac{d_w^2}{D^2} \right)^{1/n}
 \end{aligned}$$

Exponential flow

Excess pore pressure variation. Replacing the initial excess pore water pressure \bar{u}_0 with $\gamma_w \Delta \bar{h}_0$, where $\Delta \bar{h}_0$ is the initial average hydraulic head increase, the variation of the hydraulic head increase E_h outside the zone of smear ($D/2 \geq \rho \geq d_s/2$), based on non-validity of Darcy's law, becomes:

The variation of hydraulic head increase outside the zone of smear in exponential flow ($D/2 \geq \rho \geq d_s/2$), assuming that $i \leq i_l$, becomes:

$$\Delta h = \frac{\Delta \bar{h}_0}{2^n \beta n^2} (1 - \bar{U}_h) \left\{ F\left(\frac{2\rho}{D}\right) - F\left(\frac{d_s}{D}\right) + \frac{\kappa_h}{\kappa_s} \left[F\left(\frac{d_s}{D}\right) - F\left(\frac{d_w}{D}\right) \right] + \right. \\ \left. + \frac{\kappa_h}{q_w} \pi z (2l - z) \left(1 - \frac{1}{n}\right) \left(1 - \frac{d_w^2}{D^2}\right)^{1/n} \left(\frac{d_w}{D}\right)^{(1-1/n)} \right\}$$

where

$$F(x) = x^{1-1/n} \left[1 - \frac{1-1/n}{3n-1} x^2 - \frac{(1-1/n)^2}{2(5n-1)} x^4 - \frac{(1-1/n)^2 (2n-1)}{6n(7n-1)} x^6 - \dots \right],$$

in which the variable x represents $2\rho/D$, d_s/D and d_w/D .

Exponential flow

Inside the zone of smear ($d_s/2 \geq \rho \geq d_w/2$) we have:

$$\Delta h = \frac{\Delta \bar{h}_0}{2^n \beta n^2} (1 - \bar{U}_h) \left\{ \begin{aligned} & \frac{\kappa_h}{\kappa_s} \left[F\left(\frac{2\rho}{D}\right) - F\left(\frac{d_w}{D}\right) \right] + \\ & + \frac{\kappa_h}{q_w} \pi z (2l - z) \left(1 - \frac{1}{n}\right) \left(1 - \frac{d_w^2}{D^2}\right)^{1/n} \left(\frac{d_w}{D}\right)^{(1-n)} \end{aligned} \right\}$$

Hydraulic gradient. The hydraulic gradient outside the zone of smear ($D/2 \geq \rho \geq d_s/2$) in exponential flow, i.e. assuming that $i \leq i_l$, becomes:

$$i = \frac{\Delta \bar{h}_0}{D} (1 - \bar{U}_h) \left[\frac{1}{4\alpha(n-1)} \left(\frac{D}{2\rho} - \frac{2\rho}{D} \right) \right]^{1/n}$$

while inside the zone of smear ($d_s/2 \geq \rho \geq d_w/2$):

Correlations

Correlation between λ and c_h

An approximate correlation between c_h and λ can be found by equalising the areas created below the flow vs. hydraulic gradient curves in the two cases non-Darcian and Darcian flow. This yields the correlation:

$$\lambda/c_h = \frac{n+1}{2i^{n-1}} \text{ when } i \leq i_l$$

and

$$\lambda/c_h \approx \frac{i^2}{2} \left[\frac{i_l^{n+1}}{n+1} + ni_l^{n-1} (i - i_l) \left(\frac{i - i_l}{2} + \frac{i_l}{n} \right) \right]^{-1} \text{ when } i > i_l$$

The maximum hydraulic gradient is obtained for $\bar{U}_h = 0$ and $\rho = d_s/2$. Assuming, for example, that the maximum gradients reached during the consolidation process are, respectively, 2, 5, 15, 25 and 75 and that the exponent $n = 1.5$ and the limiting gradient $i_l = 8$, we find in due order $\lambda/c_h = \kappa_h/k_h \approx 0.88, 0.56, 0.34, 0.29$ and 0.25 .

Assuming $n = 1.5$ and $i_{\max} \leq 1.5i_l$, the ratio of λ to c_h can be obtained by the relation:

$$\lambda/c_h = 1.25/\sqrt{i_{\max}}$$

$$\text{where } i_{\max} = \frac{\Delta \bar{h}_0}{D} \left[\frac{1}{2\alpha} \left(\frac{D}{d_s} - \frac{d_s}{D} \right) \right]^{2/3}$$

Contribution of one-dimensional consolidation

Contribution of \bar{U}_v

According to Carillo's relation $\bar{U}_{\text{tot}} = \bar{U}_v + \bar{U}_h - \bar{U}_v \bar{U}_h$, the total average consolidation, including the effect of vertical drainage (horizontal pore water flow) and one-dimensional consolidation (vertical pore water flow) can be expressed by the relation:

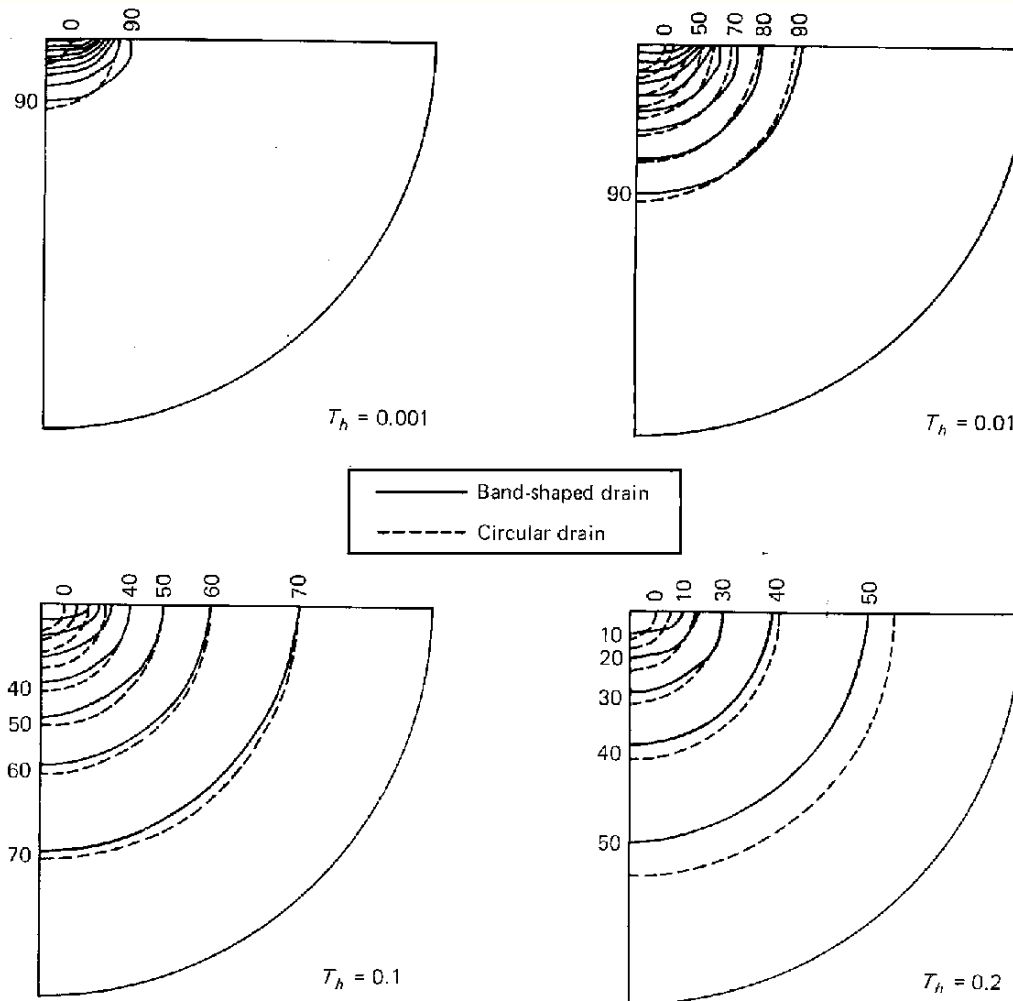
$$\bar{U}_{\text{tot}} = 1 - \left(1 - \frac{2}{l} \sqrt{\frac{c_v t}{\pi}} \right) \exp \left(-\frac{8c_h t}{\mu_{\text{av}} D^2} \right)$$

in Darcian flow, and

$$\bar{U}_{\text{tot}} = 1 - \left(1 - \frac{2}{l} \sqrt{\frac{c_v t}{\pi}} \right) \left[1 + \frac{\lambda t}{\alpha_{\text{av}} D^2} \left(\frac{\bar{u}_0}{D \gamma_w} \right)^{n-1} \right]^{1/(1-n)}$$

in exponential flow.

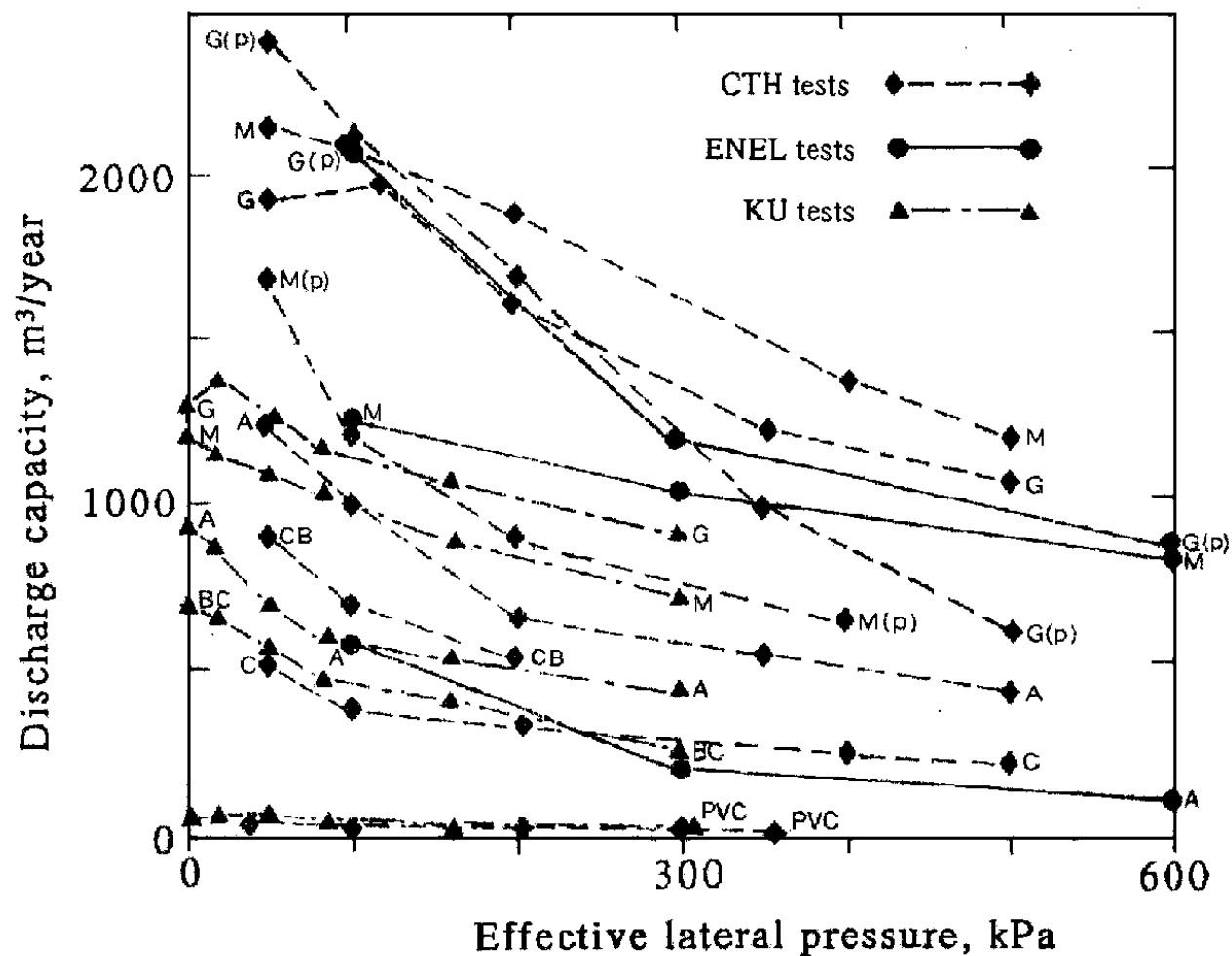
Comparison band drain circular drain



- Comparison of consolidation effects (remaining excess pore water pressure u in % of initial excess pore water pressure u_0) caused by a band drain (100 mm in width and 4 mm in thickness) and a circular drain with the same circumference ($d_w = 66$ mm). $D = 1$ m. $T_h = c_h t / D^2$. No effect of smear or well resistance. (Hansbo, 1979)

Discharge capacity tests

A = Alidrain, BC = Bando Chemical, CB = Castle Board, C = Colbond, G = Geodrain. (p) indicates filter sleeve of paper •



Buckling



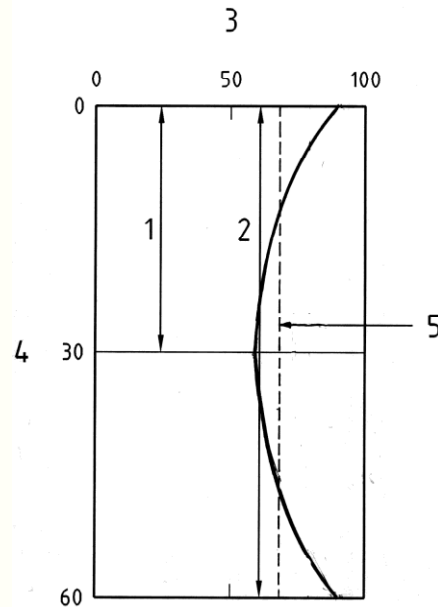
- **Buckling and kinking of drain due to very large relative compression of peat**

Discharge capacity

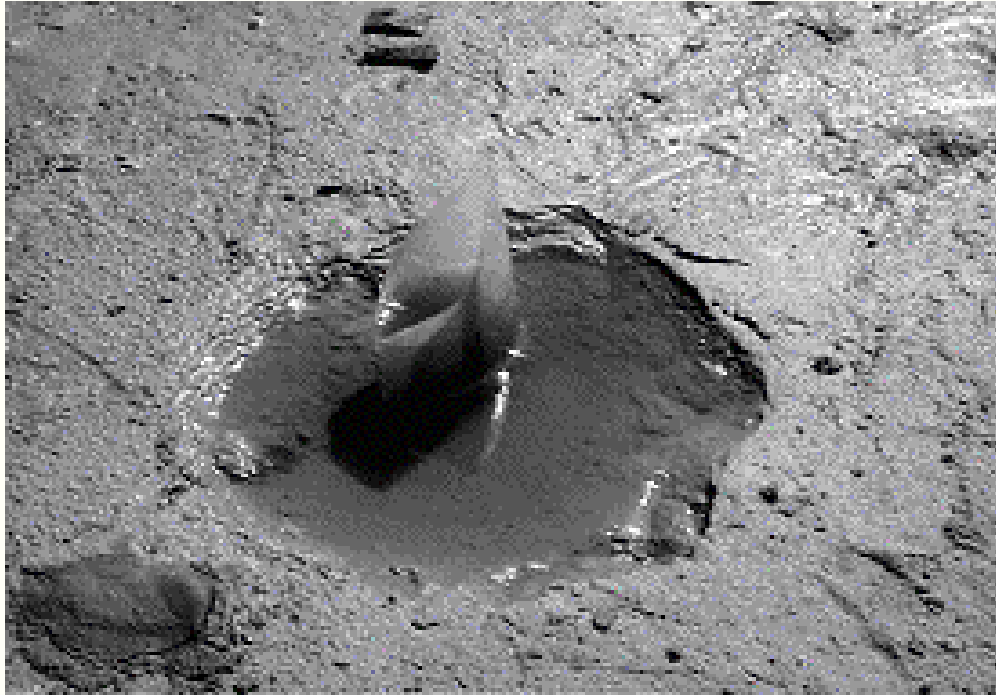
Consolidation parameters: $q_w = 100 \text{ m}^3/\text{year}$ ($\text{\AA} 3,2 \text{ cm}^3/\text{s}$),
 $c_h = 1,0 \text{ m}^2/\text{year}$ ($\text{\AA} 3,2 \times 10^{-8} \text{ m}^2/\text{s}$),
 $k_s = k_h = 0,1 \text{ m/year}$ ($\text{\AA} 3,2 \times 10^{-9} \text{ m/s}$),
time of consolidation $t = 0,5 \text{ year}$.
Drain spacing $0,9 \text{ m}$ (equilateral triangular pattern;
 $D = 0,945 \text{ m}$), drain diameter $d_w = 0,065 \text{ m}$.

Key

- 1 Partially penetrating drain ($l = 30 \text{ m}$)
- 2 Penetrating drain ($2l = 60 \text{ m}$)
- 3 Degree of consolidation \bar{U}_h %
- 4 Depth of drain installation, m
- 5 $\bar{U}_{h, \text{average}}$

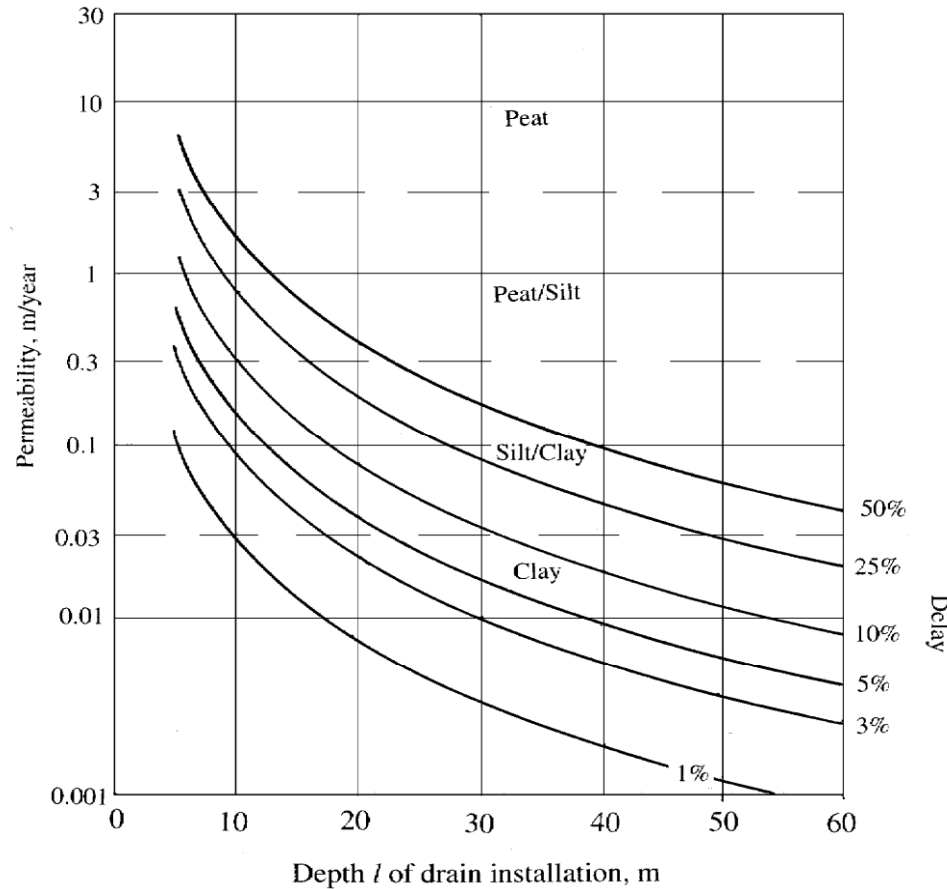


Drainage layer



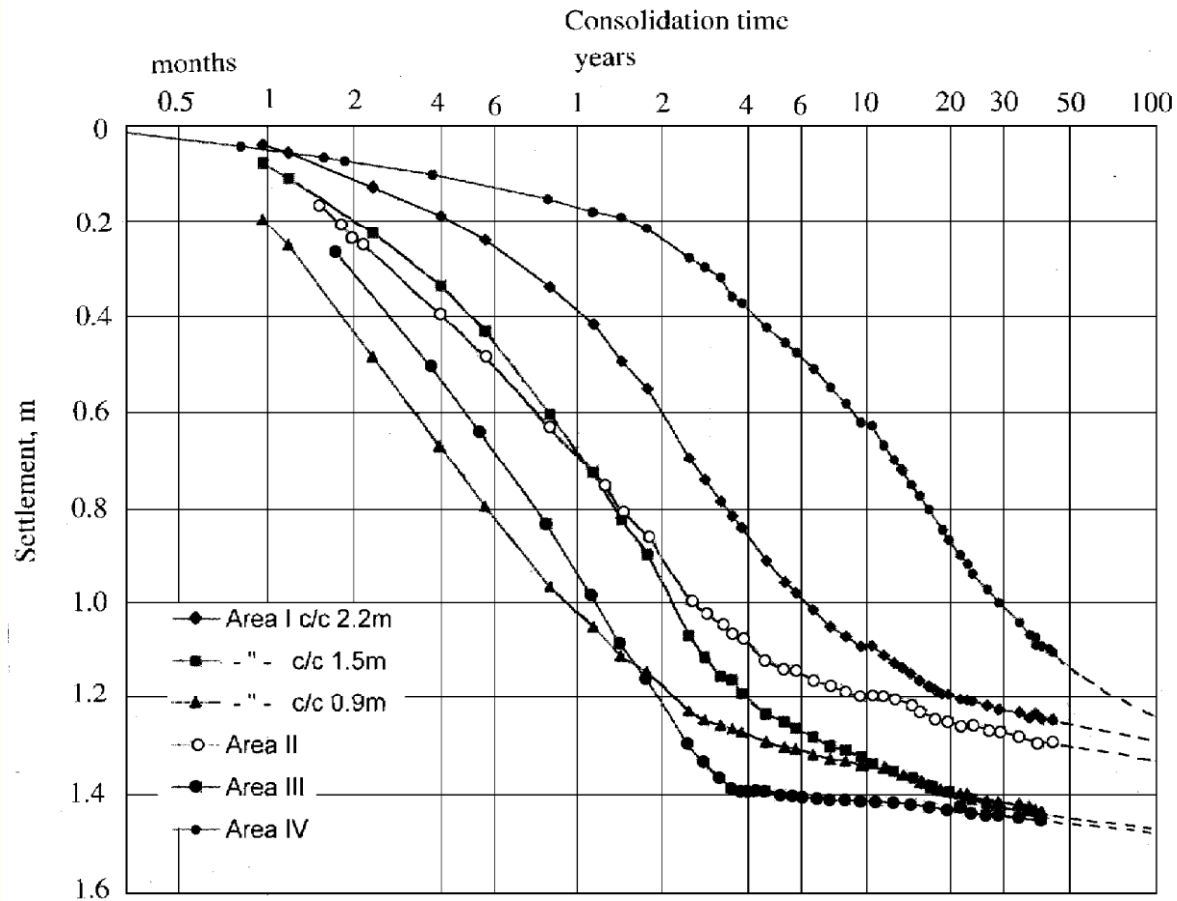
Example of drainage blanket of granular material with insufficient permeability, showing water trapped in the drainage blanket, implying backpressure in the drain

Delay in time of consolidation



Delay in time of consolidation at depth l of drain installation for drains with a discharge capacity of $500 \text{ m}^3/\text{year}$ ($16 \text{ cm}^3/\text{s}$). Drain spacing 0.9 m (equilateral triangular pattern; $D = 0.945 \text{ m}$), drain diameter $d_w = 0.065 \text{ m}$.

Skå Edeby test areas



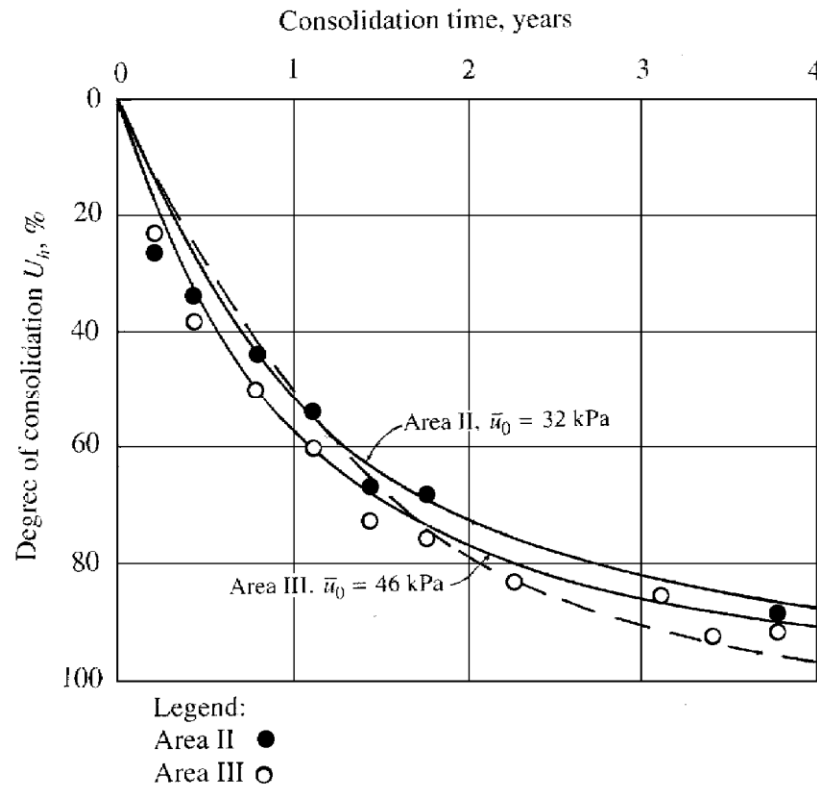
Settlement vs. time of loading in the Skå Edeby test areas, revised to represent a common depth of 12.2 m. After Larsson (1986). The broken lines represent an estimation of the continuation of the settlement process after the end of observations

Comparison analyses, Test Area I

Best fit of the c_h and λ values in Test Area I, based on settlement. (Load = 27 kN/m²)

Drain spacing	0.9 m ($\bar{u}_0 = 37$ kPa)			1.5 m ($\bar{u}_0 = 31$ kPa)			2.2 m ($\bar{u}_0 = 27$ kPa)		
Time, years	1/6	1	2	1/2	1	4	1	4	9
$\bar{U}_{\text{tot}}, \%$	34	82	94	35	52	92	33	71	89
$\bar{U}_v, \%$	6	14	17	10	14	30	14	30	46
$\bar{U}_h, \%$	30	79	93	28	44	88	22	59	79
$c_h, \text{m}^2/\text{year}$	0.88	0.49	0.42	0.70	0.62	0.57	0.61	0.58	0.46
$\lambda, \text{m}^2/\text{year} (n=15)$	0.26	0.26	0.30	0.34	0.32	0.44	0.36	0.38	0.37
$\bar{U}_h, \%$ ($\lambda_{\text{av.}} = 0.34 \text{ m}^2/\text{year}$)	37	85	94	28	46	83	21	55	78

Test areas II and III, Skå Edeby



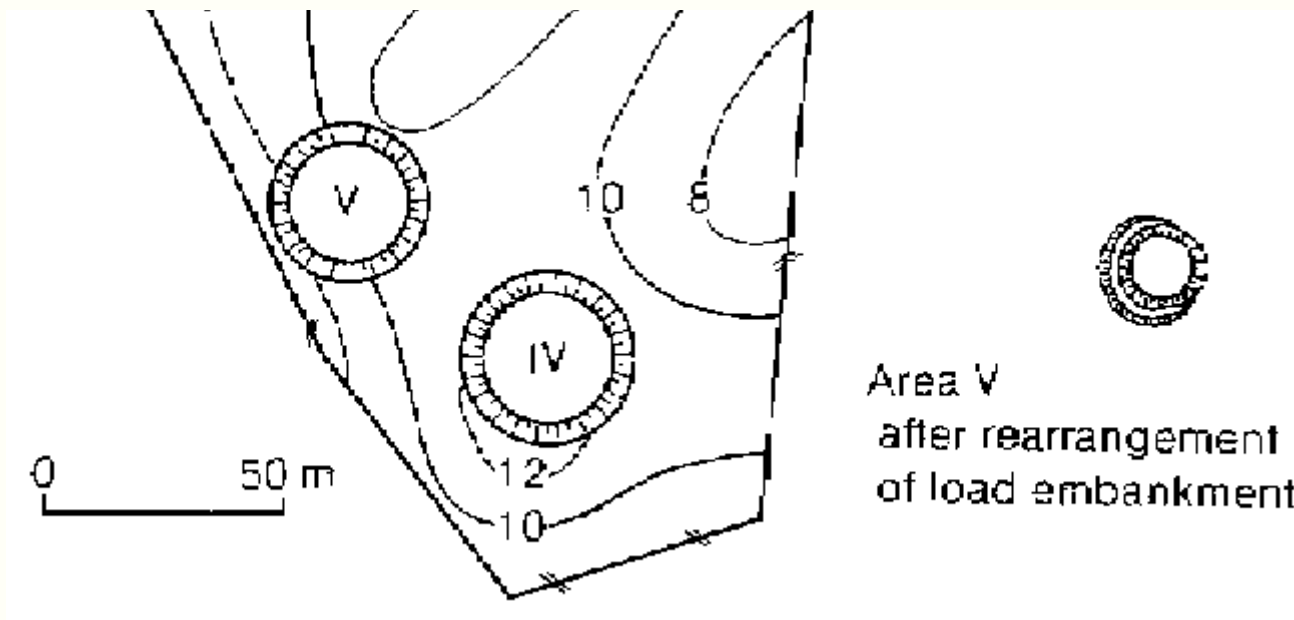
Observed and calculated excess pore pressure dissipation at a depth of 5 m in the Skå Edeby test areas II ($u_0 = 32$ kPa) and III ($u_0 = 46$ kPa). Unbroken lines represent exponential flow with $n = 1.5$ and $\lambda = 0.43$ m²/year, broken line Darcian flow with $c_h = 0.8$ m²/year.

Comparison analyses test areas II and III

Best fit of c_h and λ values in Test Areas II (load 27 kN/m²) and III (load = 39 kN/m²), based on total settlement.

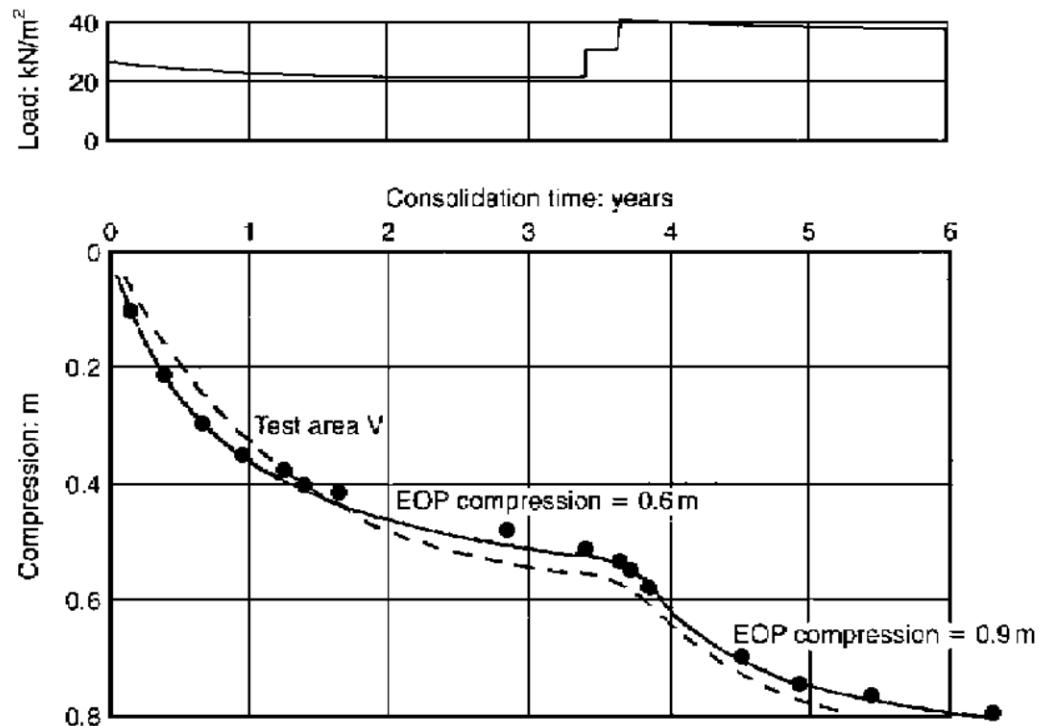
Test Area	II ($\bar{u}_0 = 32$ kPa)					III ($\bar{u}_0 = 46$ kPa)			
Time, years	1/6	1/2	2	4	9	1/6	1/2	2	4
\bar{U}_{tot} , %	20	42	75	90	98	20	45	80	94
\bar{U}_v , %	6	10	19	30	46	6	10	19	30
\bar{U}_h , %	15	36	69	86	96	15	39	77	91
c_h , m ² /year	1.04	0.95	0.63	0.52	0.39	1.05	1.05	0.78	0.65
λ , m ² /year (n =1.5)	0.47	0.46	0.37	0.39	0.42	0.40	0.44	0.42	0.46
\bar{U}_h , % ($\lambda_{\text{av.}} = 0.43$ m ² /year)	14	34	73	88	96	16	38	77	90

Test area V



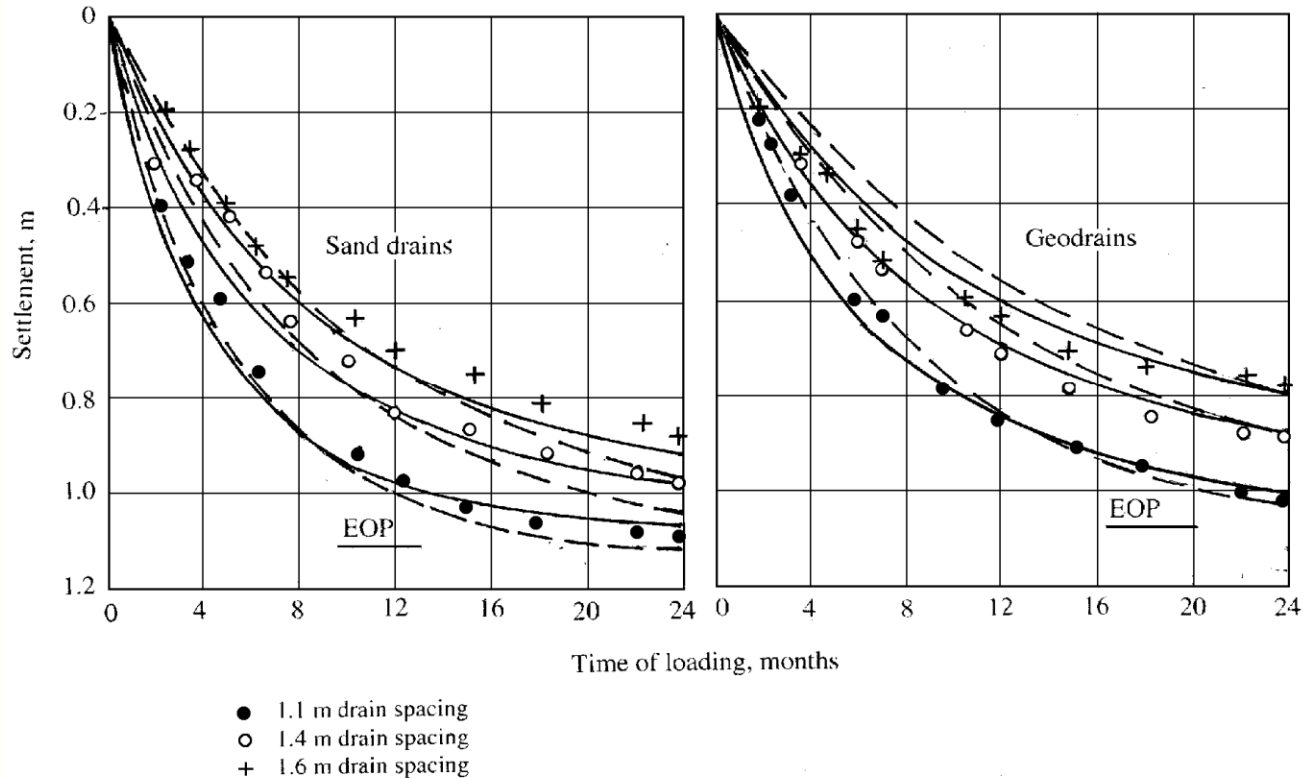
The placement of test area V in relation to test area IV at SkjEd eby.

Test area V, follow-up



Compression of the clay layer between 2.5 and 7.5 m of depths obtained at Sk -Edeby in Test Area V: 0.9 m drain spacing ($D = 0.95$ m) Broken line: Darcian flow with $c_h = 0.45$ m²/year; Unbroken line non-Darcian flow with $\lambda = 0.23$ m²/year and $n = 1.5$.

The Örebro test areas



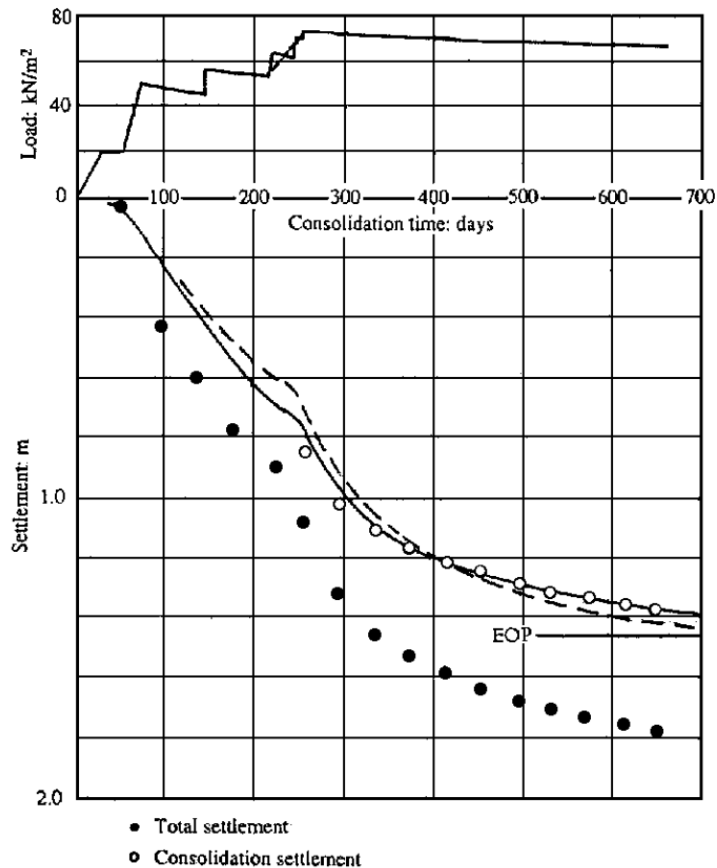
Measured and calculated settlements in test field, ..rebro. Sand drains, 0.18 m in diameter, and band drains, type Geodrain. Load $E_q = 40 \text{ kN/m}^2$. EOP means end of primary settlement. Unbroken lines represent exponential flow, equation (19) ($\lambda = 0.58 \text{ m}^2/\text{year}$; $n = 1.5$), broken lines Darcian flow, equation (18) ($c_h = 1.15 \text{ m}^2/\text{year}$).

Comparison analyses

Best fit of c_h and λ values in the ..rebro test areas (load 40 kN/m²).

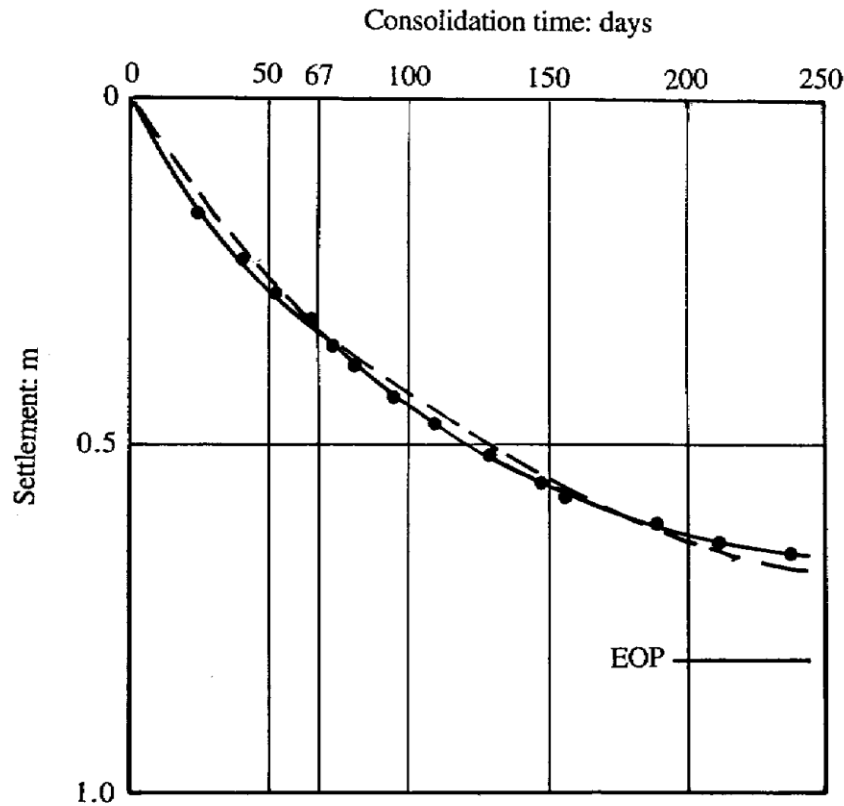
Sand drains						
c_h m ² /year						
Loading time	4 months	8 months	12 months	16 months	20 months	24 months
c/c 1.1 m	1.01	1.02	1.02	1.02	1.03	1.03
c/c 1.4 m	0.93	1.00	1.00	0.89	0.90	0.84
c/c 1.6 m	0.99	1.13	0.97	0.94	0.87	0.79
λ m ² /year						
c/c 1.1 m	0.42	0.50	0.62	0.75	0.93	1.15
c/c 1.4 m	0.38	0.46	0.53	0.48	0.53	0.52
c/c 1.6 m	0.42	0.52	0.47	0.48	0.47	0.48
Geodrains						
c_h m ² /year						
Loading time	4 months	8 months	12 months	16 months	20 months	24 months
c/c 1.1 m	1.08	1.24	1.20	1.15	1.06	1.05
c/c 1.4 m	1.39	1.49	1.44	1.42	1.27	1.28
c/c 1.6 m	1.70	1.88	1.53	1.42	1.25	1.06
λ m ² /year						
c/c 1.1 m	0.36	0.50	0.57	0.64	0.60	0.67
c/c 1.4 m	0.66	0.74	0.67	0.65	0.59	0.52
c/c 1.6 m	0.57	0.57	0.62	0.69	0.61	0.67

The Bangkok test area



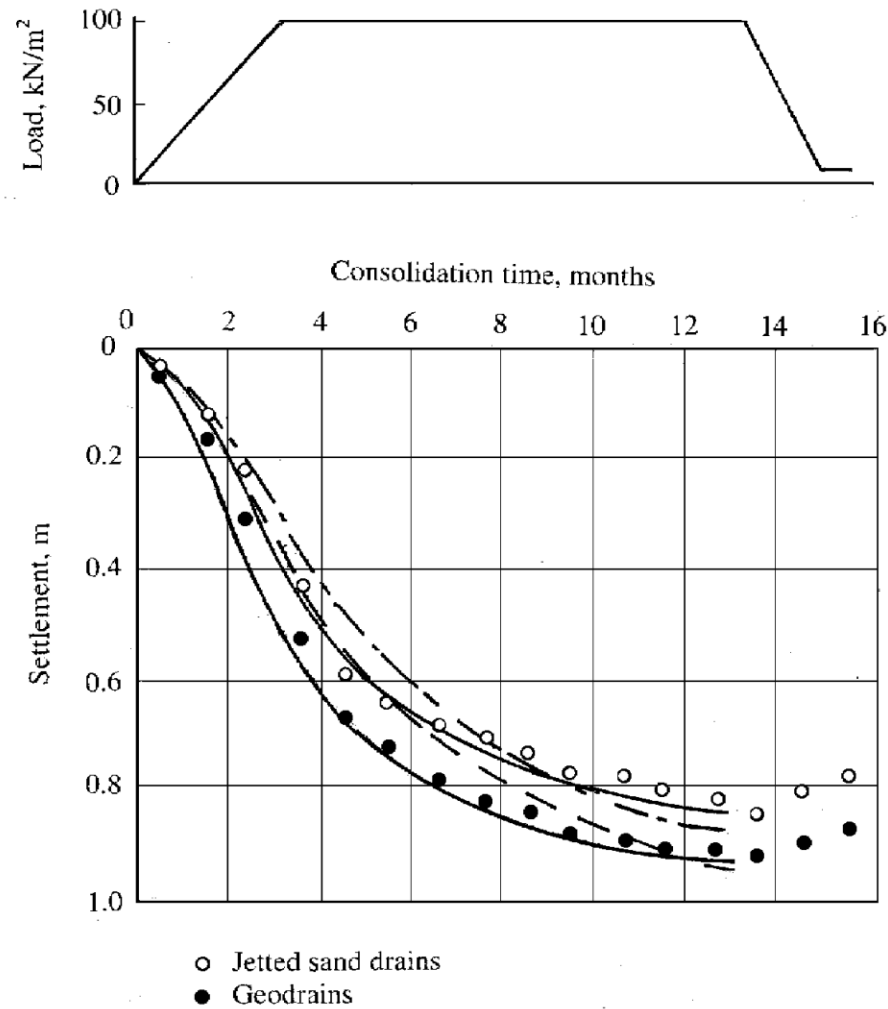
Settlement of ground surface in the Bangkok test field, Thailand. Test area TS3: 1.0 m drain spacing ($D = 1.13$ m). Settlement corrected with regard to immediate and long-term horizontal displacements. EOP = end of primary consolidation settlement estimated according to Asaoka's method. Full lines: analytical results according to exponential flow with $\lambda = 0.37 \text{ m}^2/\text{year}$ and $n = 1.5$. Broken lines: analytical results according to Darcian flow with $c_h = 0.93 \text{ m}^2/\text{year}$.

The Vagnhärad vacuum test



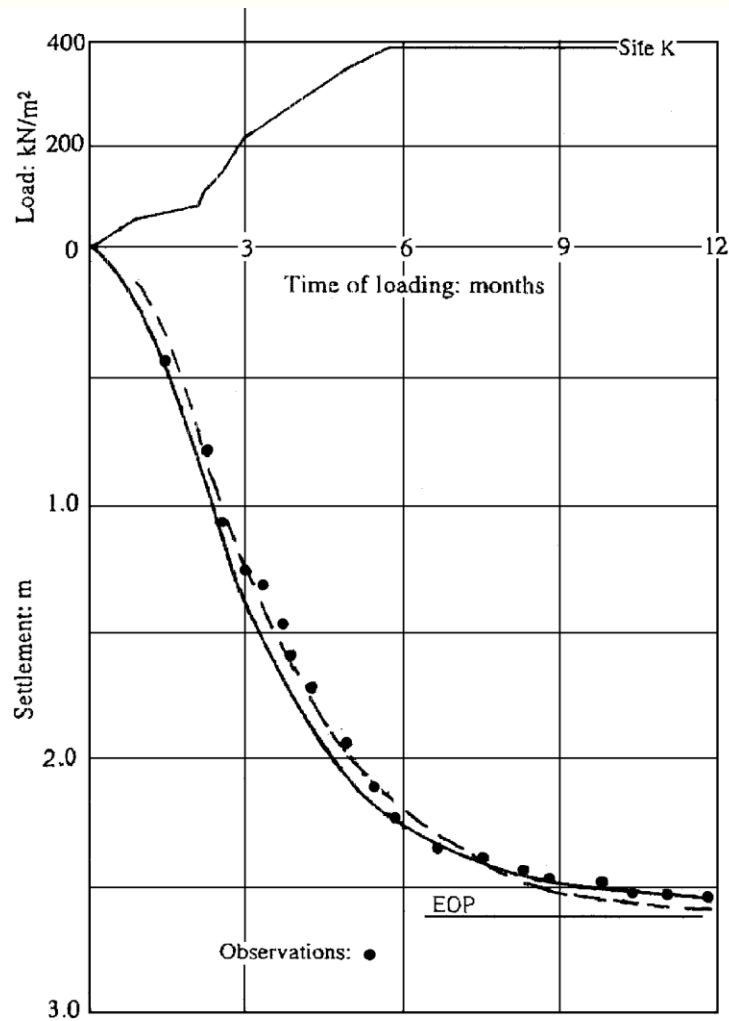
Results of settlement observations at Vagnhärad, Sweden. Consolidation by vacuum. 1.0 m drain spacing ($D = 1.13$ m). EOP = end of primary consolidation settlement estimated according to Asaoka's method. Full lines: analytical results according to non-Darcian flow, equation (19), with $\lambda = 0.95$ m²/year and $n = 1.5$. Broken lines: analytical results according to Darcian flow, equation (18), with $c_h = 2.4$ m²/year

The Porto-Tolle test site, Italy



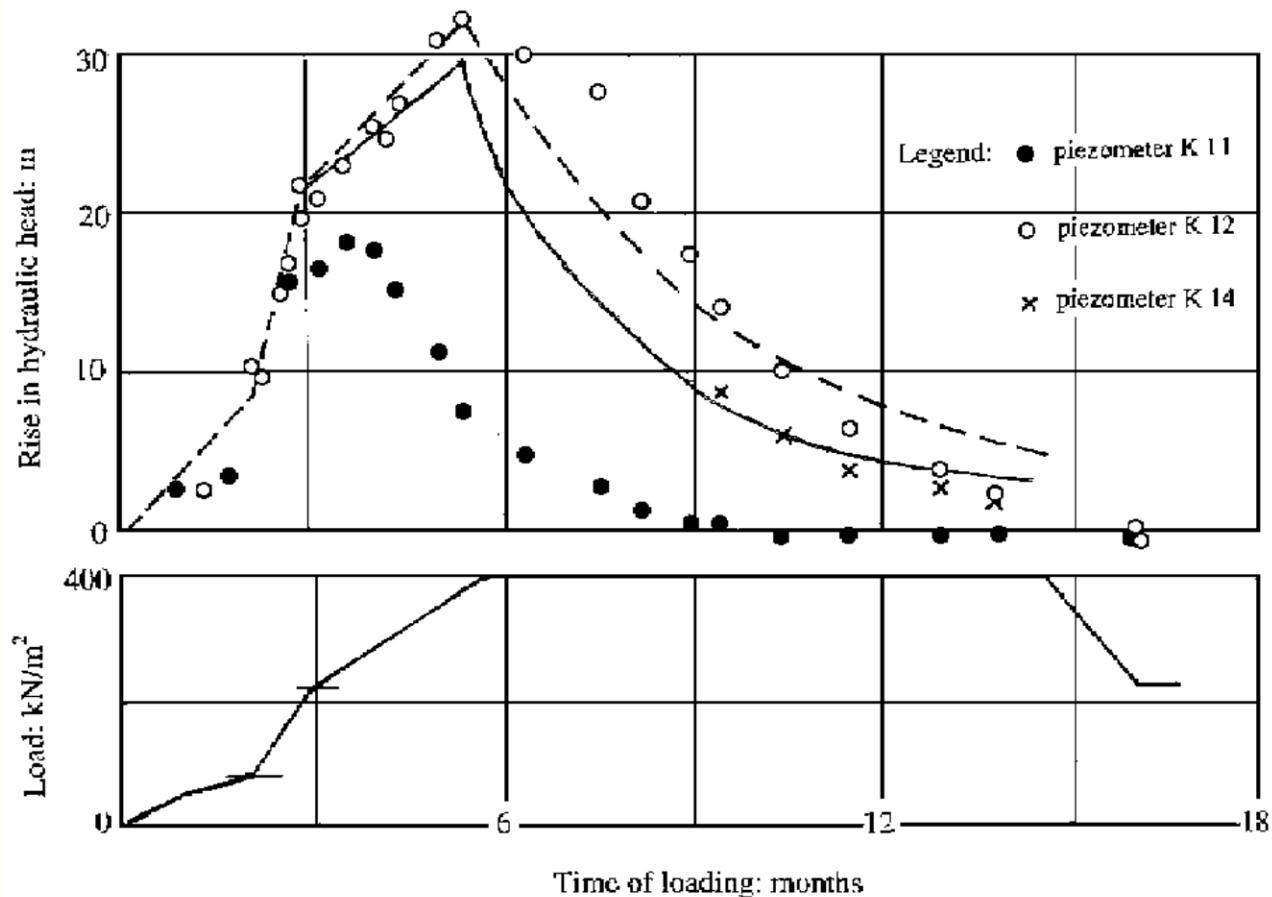
Trial embankment at Porto Tolle, Italy. Measured and calculated settlements and loading conditions for test sections provided with band drains, type Geodrain and jetted sand drains. Broken line represents Geodrain, and the dash-dotted line jetted sand drains according to Darcian flow with $c_h = 24 \text{ m}^2/\text{year}$, and full lines exponential flow with $\lambda = 14 \text{ m}^2/\text{year}$ and $n = 1.5$.

The Stockholm-Arlanda project



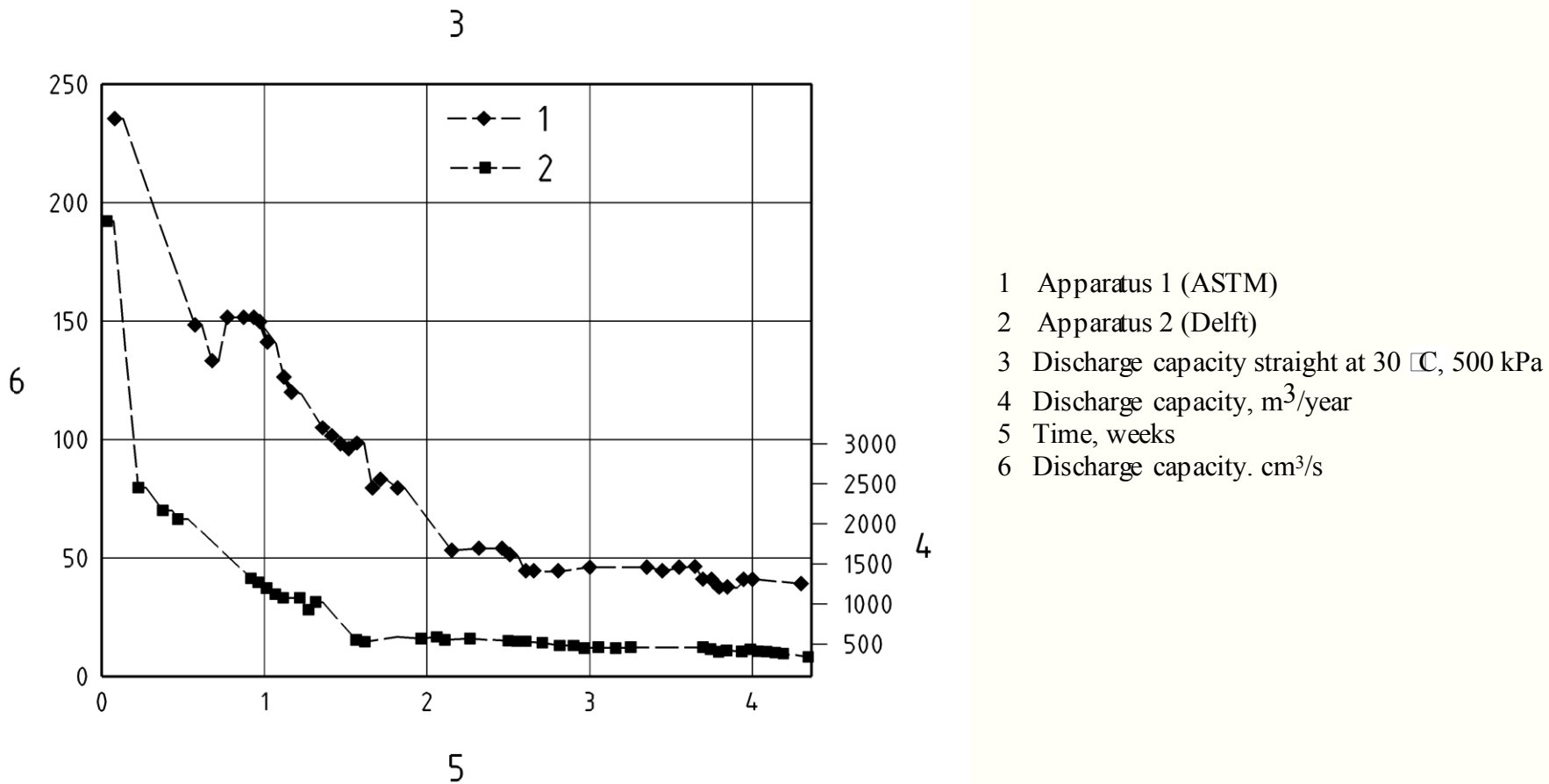
Comparison between calculated course of settlement and observations at Stockholm Arlanda Airport. Broken line represents Darcian flow according to equation (18) with $c_k = 2.6 \text{ m}^2/\text{year}$, unbroken line non-Darcian flow according to equation (19) with $\lambda = 0.7 \text{ m}^2/\text{year}$ and $n = 1.5$.

The Stockholm-Arlanda project

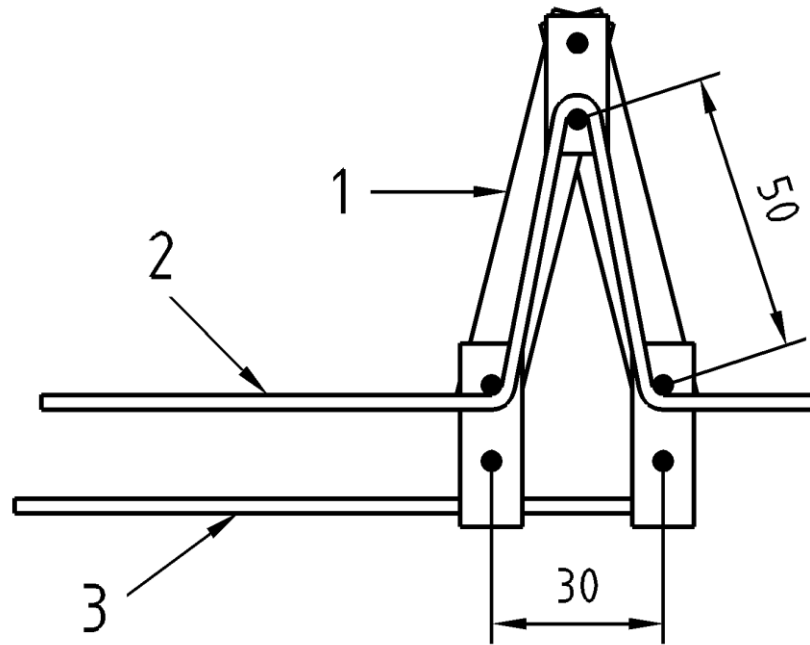


Comparison between theoretical and observed excess pore pressure dissipation at Stockholm Arlanda Airport. Piezometers placed at the following levels (level of original ground surface + 21.0): K 11 + 17.0; K 12 + 15.0; K 14 (after 10 months of loading) +14.3. Broken line represents Darcian flow with $c_h = 2.6 \text{ m}^2/\text{year}$, unbroken line non-Darcian flow with $\lambda = 0.7 \text{ m}^2/\text{year}$ and $n = 1.5$.

— Creep effect on discharge capacity observed in the duration of a discharge capacity test

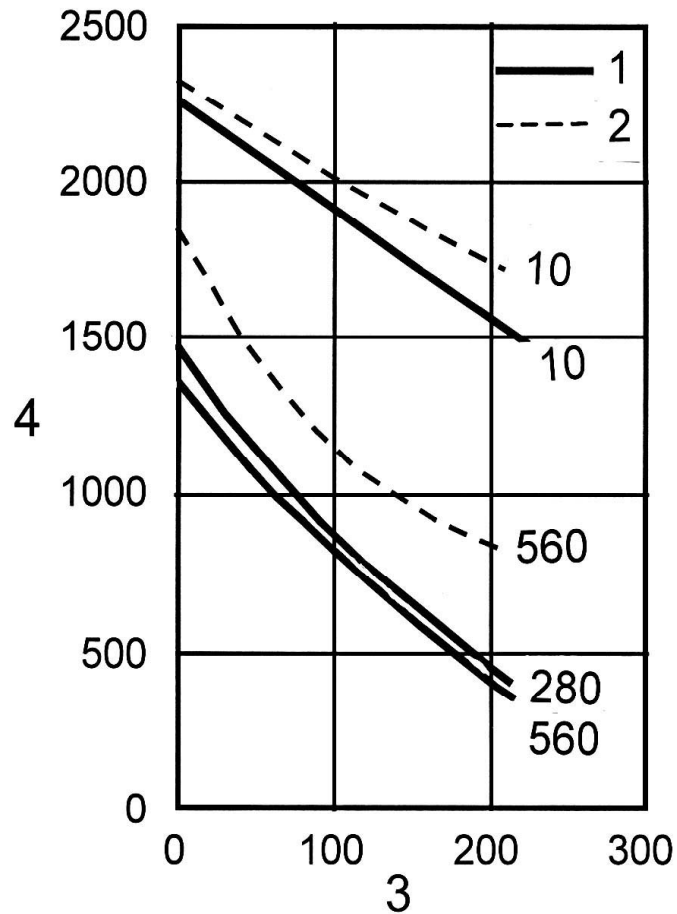


Apparatus to test discharge capacity of buckled drain



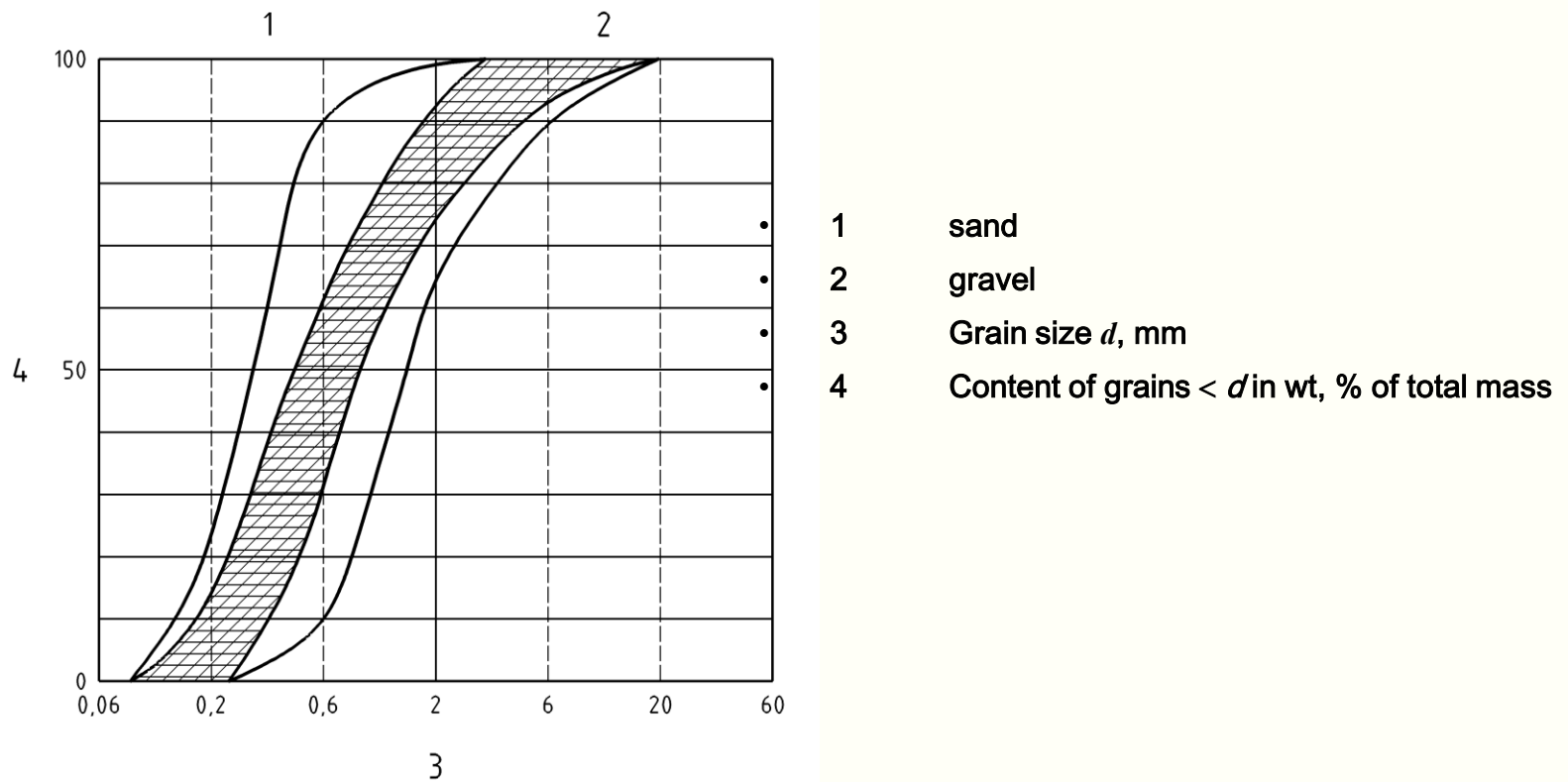
- 1 Rod A
- 2 Drain
- 3 Guide rod

Influence of time-dependent filter deterioration



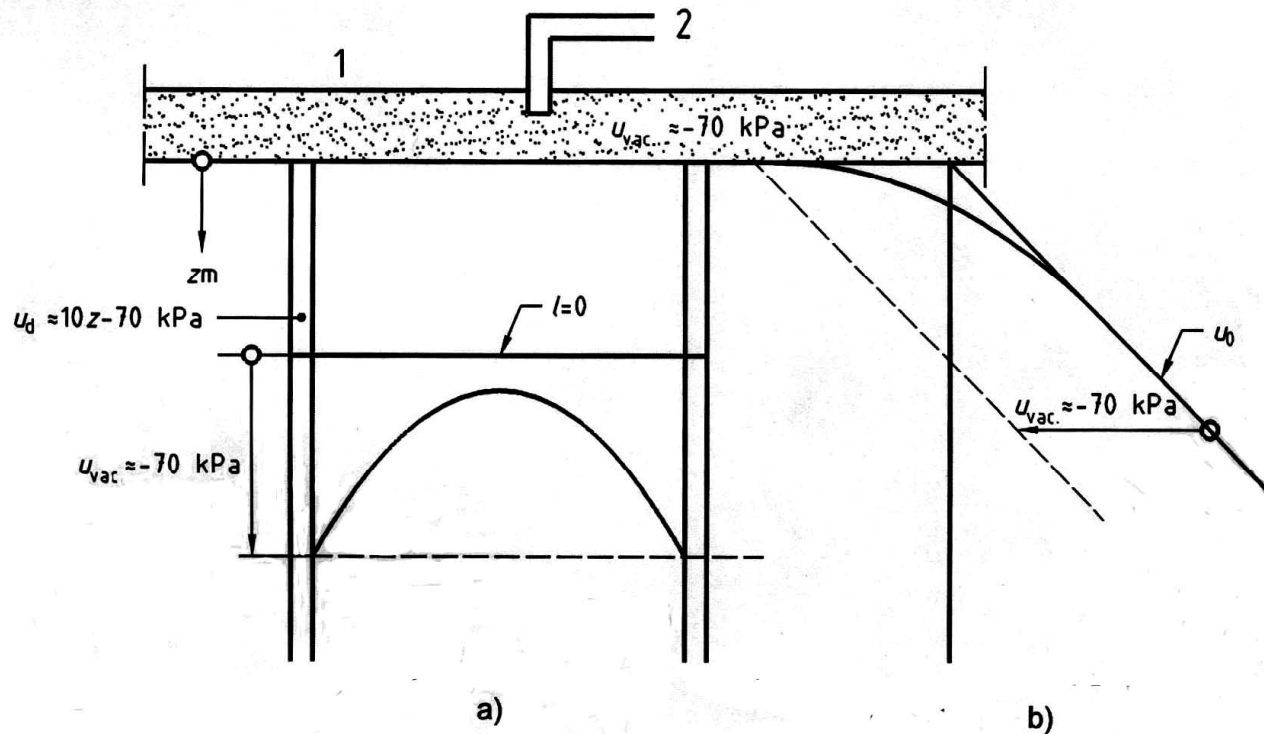
- 1 Peat
- 2 Gytja
- 3 Effective lateral pressure, kPa
- 4 Discharge capacity, m³/year

Grain size limits of sand drains

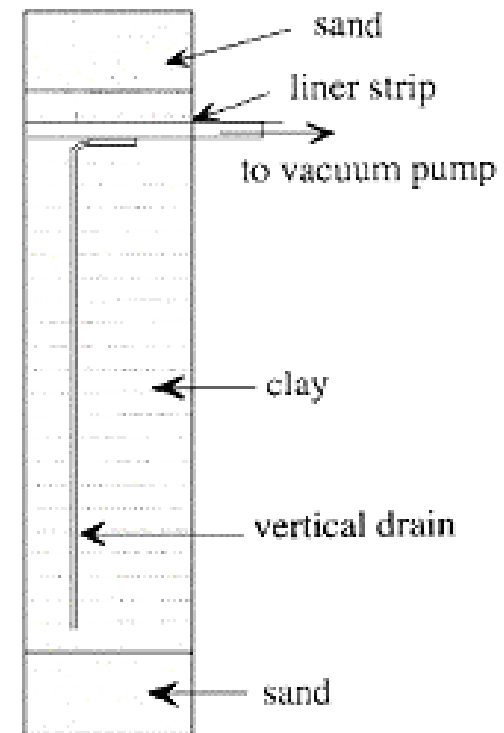
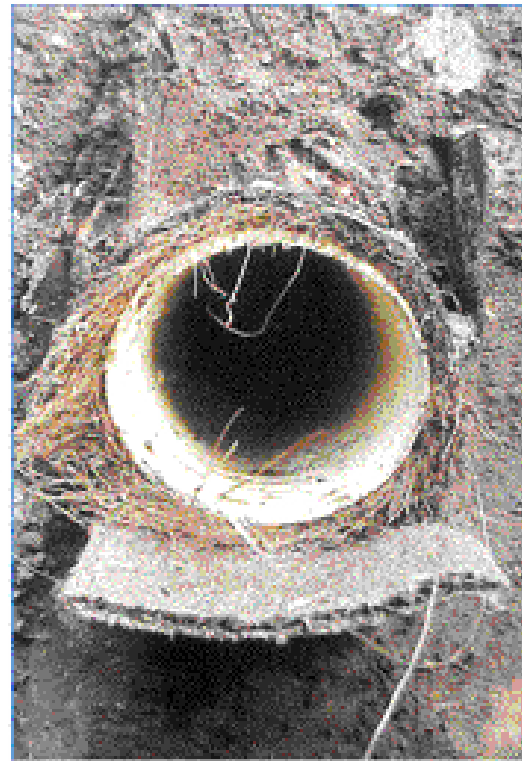


Vacuum method and its effect on pore water pressure

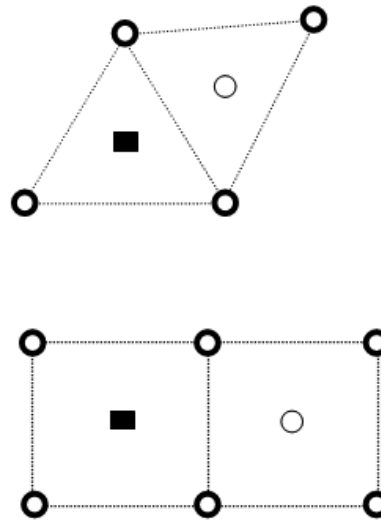
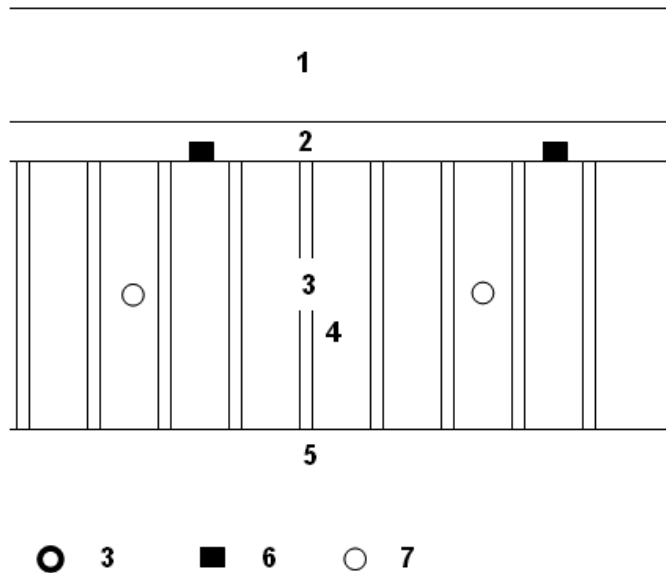
- 1 Airtight cover
- 2 To vacuum pump



Installation of horizontal cylindrical drain (left) and its connection to the vertical drains



Typical instrumentation for monitoring the efficiency of vertical drainage



- 1. embankment
- 2. drainage blanket and working platform
- 3. vertical drain
- 4. compressible soil
- 5. underlying permeable layer
- 6. settlement gauge
- 7. piezometer
- 8. permeable sand layer
- 9. compressible soil