CE464 Ground Improvement

Fall 2016

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Increase in undrained shear strength (c_u) of soft clays due to Preloading

PRELOADING:

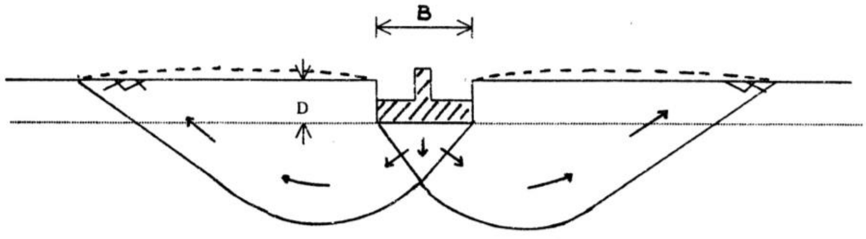
THE PURPOSE IS TO TAKE UP THE SETTLEMENTS UNDER THE STRUCTURES BEFORE THEY ARE BUILT.

PRELOADING RESULTS IN;

- PRIMARY CONSOLIDATION SETTLEMENT
- SECONDARY CONSOLIDATION SETTLEMENT
- INCREASE IN THE UNDRAINED SHEAR STRENGTH OF SOIL.

Therefore preloading of soft clays improves the ground by reducing settlement, and also by increasing bearing capacity.

Reminder: CE366 Foundation Engineering



- Short term (undrained) bearing capacity, in terms of total stress
- Long term (drained) bearing capacity, in terms of effective stress

Reminder: CE366 Foundation Engineering

- Short term (undrained) bearing capacity, in terms of total stress
  - For sands, gravels, we don't have this case (except during earthquake loading).
  - For clays and silts, short term is more critical compared to long term bearing capacity
- Long term (drained) bearing capacity, in terms of effective stress
  - Sands, gravels, allways drained (except during earthquake loading).
  - For clays, long term bearing capacity is greater than short term bearing capacity.

Long term (drained) bearing capacity, in terms of effective stress

Ultimate bearing capacity of a shallow **strip footing** is:

$$q_f = \frac{1}{2}\,\gamma.B.N_\gamma + c.N_c + \gamma.D.N_q$$

N_γ, N_c and N_q are Terzaghi's dimensionless bearing capacity factors (dependent on friction angle), B and D are width and depth of foundation respectively, c is cohesion of soil and γ is unit weight of soil.

Ultimate bearing capacity of a shallow **square footing** is:

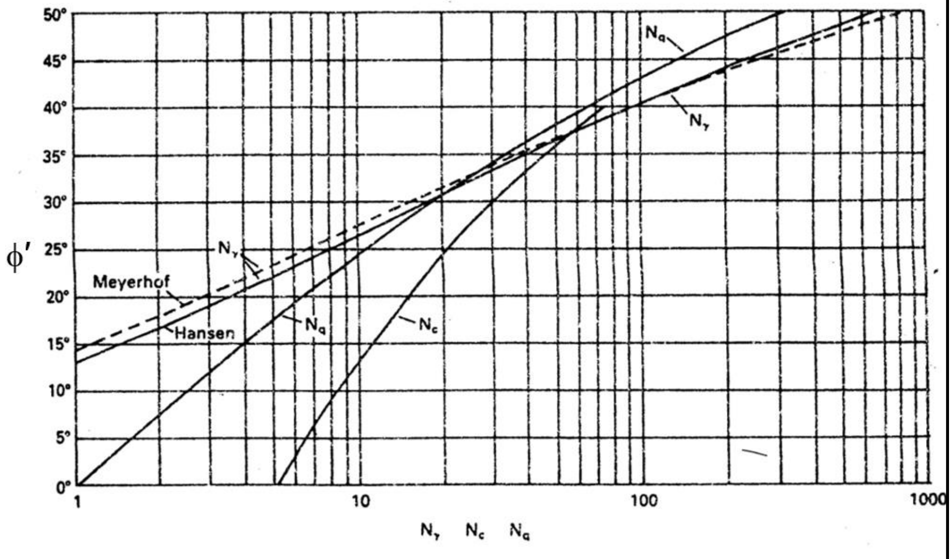
$$q_f = (0.4).\gamma.B.N_\gamma + (1.2).c.N_c + \gamma.D.N_q$$

Ultimate bearing capacity of a shallow **rectangular footing** is:

$$q_f = \frac{1}{2}\,.s_\gamma\,.\gamma.B.N_\gamma + s_c.c.N_c + s_q.\gamma.D.N_q$$

Shape of foundation	s_c	s_q	s_γ
Rectangle	1+0.2(B/L)	1	1-0.2(B/L)

Net Ultimate bearing capacity: $q_{nf} = q_f - \gamma \cdot D$



N_γ, N_c and N_q : Terzaghi's dimensionless bearing capacity factors

Short term (undrained) bearing capacity of clays, in terms of total stress

Immediately after a load is applied to a clay, positive pore water pressures develop in clay, and with time excess pore pressure dissipates, i.e. clay consolidates (average degree of consolidation, U, approaches to 95%) and clay gains strength

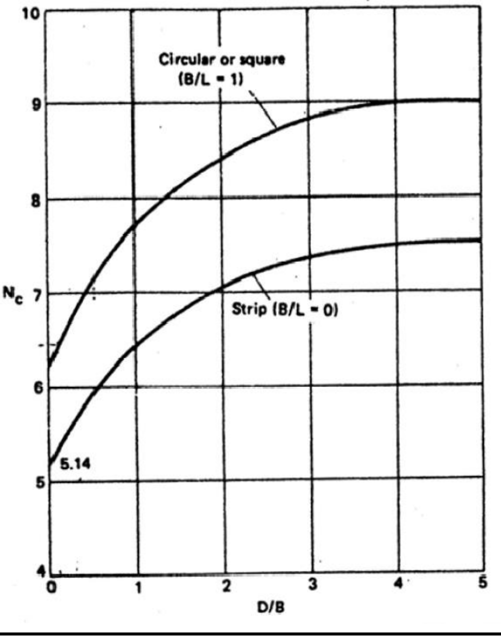
Short term (undrained) bearing capacity of clays, in terms of total stress

Ultimate bearing capacity

$$q_f = c_u.N_c + \gamma.D$$

Net ultimate bearing capacity

$$q_{nf} = q_f - \gamma \cdot D = c_u.N_c$$

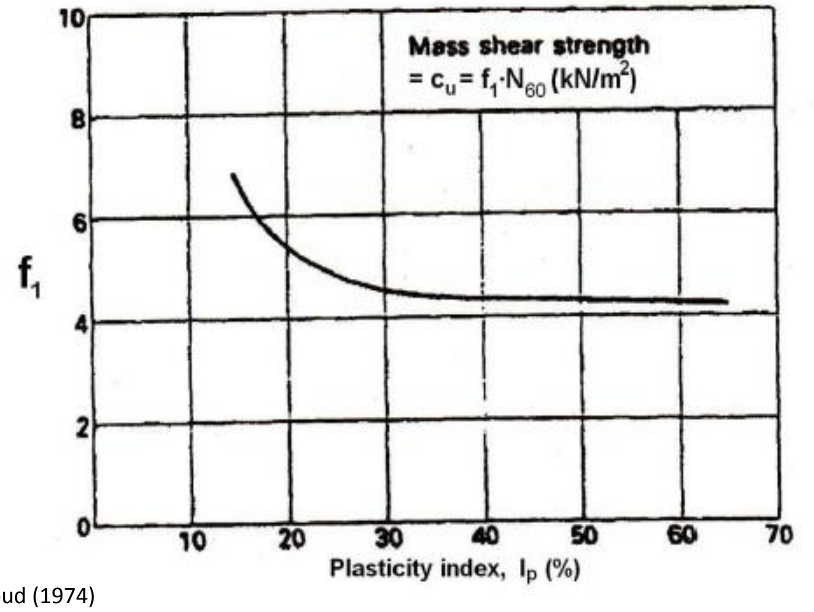


N_c factor for rectangular foundations may be calculated by multiplying the the bearing capacity (N_c) factor for strip with (1 + 0.2 B/L)

How to find undrained shear strength, c_u ?

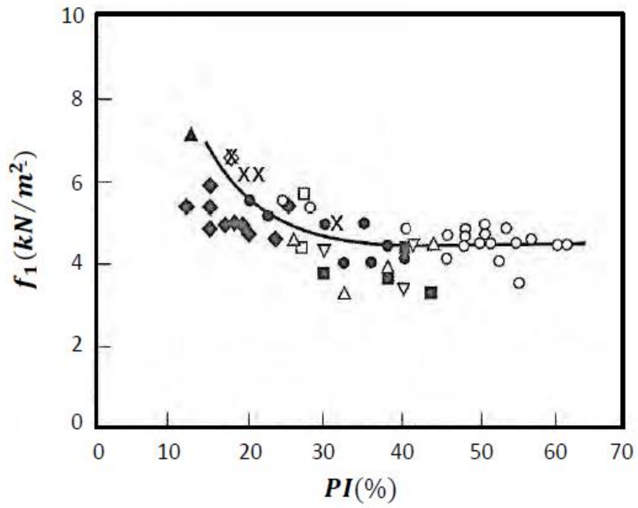
- Lab Unconsolidated Undrained (UU) triaxial compression test
- Lab Unconfined Compression (UC) test
- Field SPT correlation
- Field CPT correlation
- Field vane test
- From other empirical correlations

Undrained shear strength, c_u , from SPT correlation



Stroud (1974)

See the real data



Şekil 1.19. SPT-N_{60} - c_u - PI ilişkisi (Stroud, 1974)

Erol ve Çekinmez (2014) Geoteknik Mühendisliğinde Saha Deneyleri
http://www.yukselproje.com.tr/tr/yayinlar

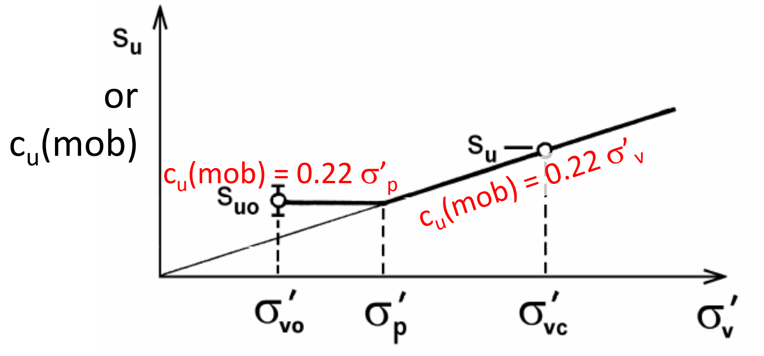
Empirical information for mobilized undrained shear strength: c_u(mob) or s_u(mob)

For inorganic soft clay and silt deposits, independently of plasticity index:

$$s_{uo}(\mathrm{mob}) = 0.22\ \sigma'_p$$

For organic soft clay and silt deposits:

$$s_{uo}(\mathrm{mob}) = 0.26\ \sigma'_p$$



If final stress is less than preconsolidation pressure:

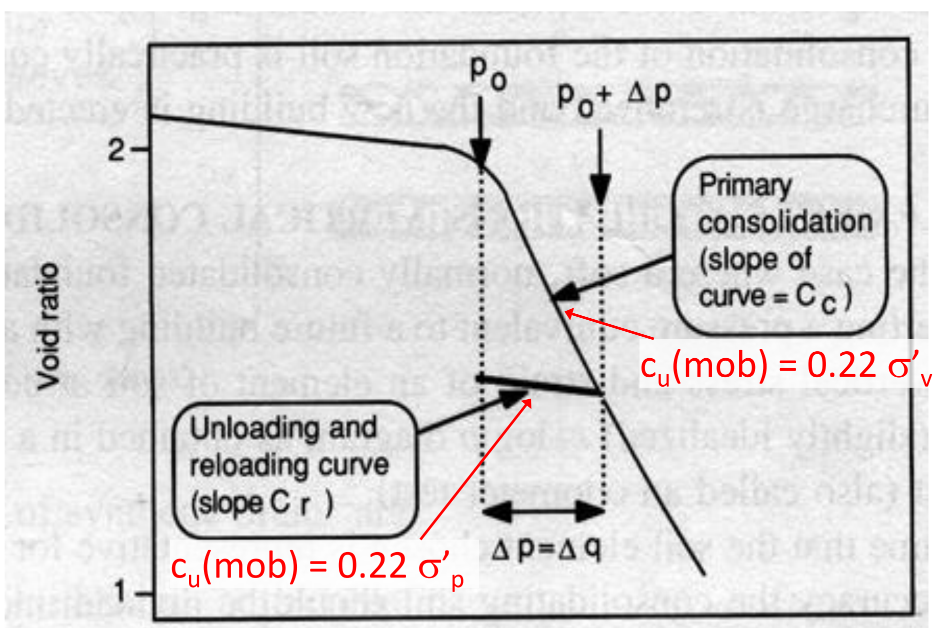
$\sigma'_f < \sigma'_p$ $\quad$ c_u(mob) = 0.22 σ'_p

If final stress is more than preconsolidation pressure:

$\sigma'_f > \sigma'_p$ $\quad$ c_u(mob) = 0.22 σ'_v

If there is no preconsolidation pressure (clay is N.C.):

c_u(mob) = 0.22 σ'_v (as clay consolidates, c_u will increase)



Example:

A 5-m high very wide fill (unit weight 20 kN/m^3) will be placed at ground surface. Clay is 10-m thick and it is underlain by sand. Water table is at the ground surface. For clay OCR = 2.2 and saturated unit weight of clay is 20 kN/m^3. Coeff. of consolidation c_v of clay is 24 m^2/year. Assume that the fill is placed instantaneously. At the mid-depth of clay:

a) Find undrained shear strength before any fill placement
b) Find undrained shear strength immediately after placement of the fill.

$$OCR = \frac{\sigma_p'}{\sigma_{vo}'}$$

a) Find undrained shear strength before any fill placement
At the mid-depth of clay

σ'_o= 5 x (20-10) = 50 kPa

σ'_p= OCR x σ'_o = 2.2 x 50 = 110 kPa

c_u(mob) = 0.22 σ'_p = 0.22 x 110 = **24.2 kPa**

$$OCR = \frac{\sigma_p'}{\sigma_{vo}'}$$

b) Find undrained shear strength immediately after placement of the fill.
Immediately after the fill is placed, all of the applied load goes to pore water pressure at time=0 and effective stresses in clay does not change. Therefore, the answer is same as part (a).
Undrained shear strength = **24.2 kPa**

Example (continued):
A 5-m high very wide fill (unit weight 20 kN/m^3) will be placed at ground surface. Clay is 10-m thick and it is underlain by sand. Water table is at the ground surface. For clay OCR = 2.2 and saturated unit weight of clay is 20 kN/m^3. Coeff. of consolidation c_v of clay is 24 m^2/year. Assume that the fill is placed instantaneously. At the mid-depth of clay:

a) Find undrained shear strength before any fill
b) Find undrained shear strength immediately after placement of the fill.
c) Find undrained shear strength 4 months after the placement of the fill.
d) How long time has to pass after the placement of the fill to have 32 kPa undrained shear strength in clay

c) Find undrained shear strength before 4 months after the placement of the fill.

$$T_v = \frac{c_v \cdot t}{d^2} = \frac{24\ \text{m}^2/\text{yr} \cdot (4/12)\ \text{yr}}{5^2} = 0.32$$

U (%)	T_v
0	0
5	0.002
10	0.008
15	0.018
20	0.031
25	0.049
30	0.071

U (%)	T_v
35	0.096
40	0.126
45	0.159
50	0.195
55	0.239
60	0.286
65	0.340

U (%)	T_v
70	0.403
75	0.477
80	0.567
85	0.684
90	0.848
95	1.129

To find average degree of consolidation of the clay layer, use either the table above, or equations below:

$$\text{For constant } u_{ie} \text{ with depth}: \begin{cases} \text{for U} < 60\%,\ T_v = \dfrac{\pi}{4} \cdot U^2 \\ \text{for U} > 60\%,\ T_v = -0.933 \cdot \log(1-U) - 0.085 \end{cases}$$

For T_v=0.32 U = 63.2%

Assume average degree of consolidation of the layer, is the same as degree of consolidation at the mid-depth of clay:

Degree of consolidation at a depth z in clay, at a certain time:

$$U_z = \frac{u_{ie} - u_e}{u_{ie}} = \frac{\sigma' - \sigma'_o}{\sigma'_f - \sigma'_o}$$

u_{ie} = initial excess pore water pressure
u_e = excess pore water pressure at that point, at that time

$$U_z = \frac{\sigma' - \sigma'_o}{\sigma'_f - \sigma'_o} = 0.63$$

$$0.63 = \frac{\sigma' - \sigma'_o}{\sigma'_f - \sigma'_o} = \frac{\sigma' - 50}{100} \qquad \sigma' = 113 \text{ kPa}$$

Change in effective vertical stress due to a wide fill, at any depth is 100 kPa

c) Find undrained shear strength 4 months after the placement of the fill.
4 months after the placement of the fill, σ'=113 kPa
σ' = 113 kPa is greater than σ'_p= 110 kPa
Therefore in calculating c_u, use the new highest σ'
c_u(mob) = 0.22 σ'_v = 0.22 x 113 = **24.9 kPa**

d) How long time has to pass after the placement of the fill to have 32 kPa undrained shear strength in clay

c_u(mob) = 0.22 σ'_v = **32 kPa** σ'_v = 145.5 kPa

$$U_z = \frac{\sigma' - \sigma'_o}{\sigma'_f - \sigma'_o} = \frac{145.5 - 50}{100} = 0.95$$ T_v=1.129

$$T_v = \frac{c_v \cdot t}{d^2} = 1.129 = \frac{24 \text{ m}^2/\text{yr} \cdot (t/12) \text{ yr}}{5^2}$$

t=1.176 years = **14 months**

NOTE:

In Example problem, we assumed average degree of consolidation of the layer, is the same as degree of consolidation at the mid-depth of the double-drained clay.

In fact, it is not.
For T_v=0.32, average degree of cons. of the layer is U = 63.2%, whereas at the mid-depth of clay at t=4 months, U_z=0.43=43%

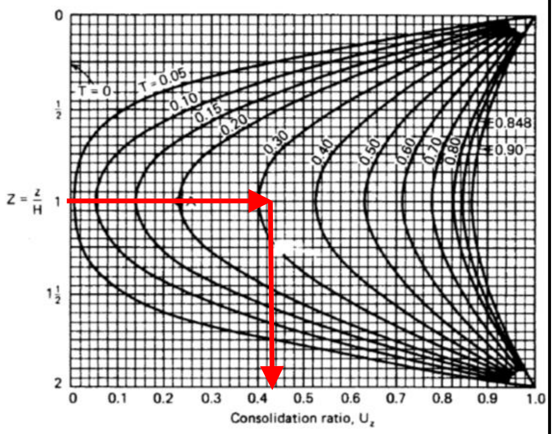


Fig. 9.3 Consolidation for any location and time factor in a doubly drained layer (after Taylor, 1948).

Example (continued):

If we want to construct a 5-storey building having a 20 m by 40 m mat foundation at the ground surface, what would be the **factor of safety against bearing capacity**,

e) if we did not apply a 5-m-high fill, but construct the building?

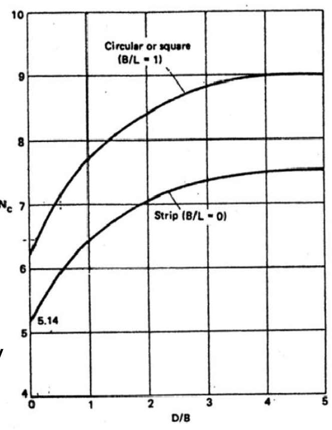
f) if we applied the wide fill and waited for some time to have 32 kPa undrained shear strength in clay

1 storey reinforced concrete building = 15 kPa

$$FS = \frac{q_{nf}}{q_{net}}$$

1 storey reinforced concrete building = 15 kPa
5 storey building applies net foundation pressure, q_{net} = 75 kPa

e) if we did not apply a 5-m-high fill, but construct the building?
Use the undrained shear strength found in Part (a) (before placing any fill), 24.2 kPa.
$q_{nf} = c_u N_c = 24.2 \times 5.65 = 136.7$ kPa



(N_c)rectangle = N_c(strip) x (1 + 0.2 B/L)
(N_c)rectangle = 5.14 x (1 + 0.2 x 20/40) = 5.65

$$FS = \frac{q_{nf}}{q_{net}} = \frac{136.7}{75} = 1.82$$

F.S. = 1.82 is not acceptable. For bearing capacity F.S. should be min. 2.5 – 3.0

f) if we applied the wide fill and waited for some time to have 32 kPa undrained shear strength in clay?
$q_{nf} = c_u N_c = 32 \times 5.65 = 180.8$ kPa

$$FS = \frac{q_{nf}}{q_{net}} = \frac{180.8}{75} = 2.41$$

F.S. = 2.41 could be acceptable. Better to apply the 5-m high fill and increase undrained shear strength to 32 kPa and bearing capacity F.S. to 2.41

Example is finished.

Increase in undrained shear strength (c_u) of soft clays with time because of consolidation is the reason for doing stage construction of embankments on soft clays, (if we don't do stage construction, embankment may have bearing capacity failure depending on c_u of the clay.)

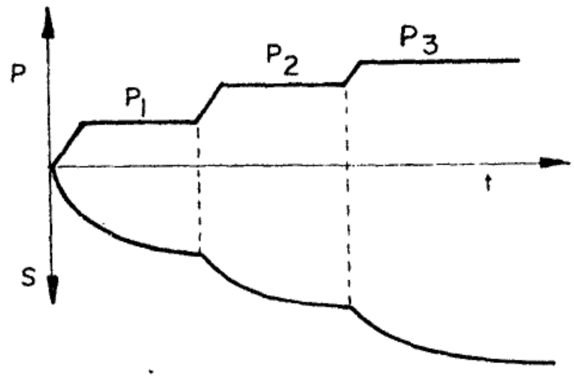


Figure 5. Stage Construction

e.g.:
If we need to construct a 6-m high embankment for a railway project,
Depending on the undrained shear strength of soil, we may not construct the 6-m fill at once, we construct in stages, and wait for some time between each stage.
We make calculations to check how long we should wait after putting each stage

e.g.
1st stage: construct 3-m high fill first, then wait for consolidation and c_u increases
2nd stage: construct 3-m high fill etc.

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PRELOADING with vertical drains

PRELOADING TECHNIQUE MAY NOT WORK SOMETIMES ALONE DUE TO A THICK UNIFORM SOFT CLAY LAYER OR PERMEABILITY OF THE CLAY IS VERY LOW SO THAT TIME FOR PRECOMPRESSION IS VERY LONG AND NOT PRACTICAL OR SURCHARGE WILL BE VERY HIGH FOR REASONABLE WAITING PERIODS.

SOMETIMES RATE OF UNDRAINED SHEAR STRENGTH GAIN IS VERY SMALL WITH TIME SO THAT RAPID PLACEMENT OF A HIGH FILL WILL CAUSE FAILURE.

TO ACCELERATE THE RATES OF SETTLEMENT HENCE TO DECREASE THE PRELOADING TIMES, VERTICAL DRAINS ARE INSTALLED TO SHORTEN THE DRAINAGE PATHS.

IT IS ESPECIALLY EFFECTIVE IN PRIMARY CONSOLIDATION.

PORE WATER PRESSURES DISSIPATE QUICKLY DUE TO SHORTER DRAINAGE DISTANCE.

IT IS NOT EFFECTIVE IN ORGANIC SOILS AND PEATS IN WHICH COMPRESSIONS ARE DOMINATED BY SECONDARY COMPRESSION.

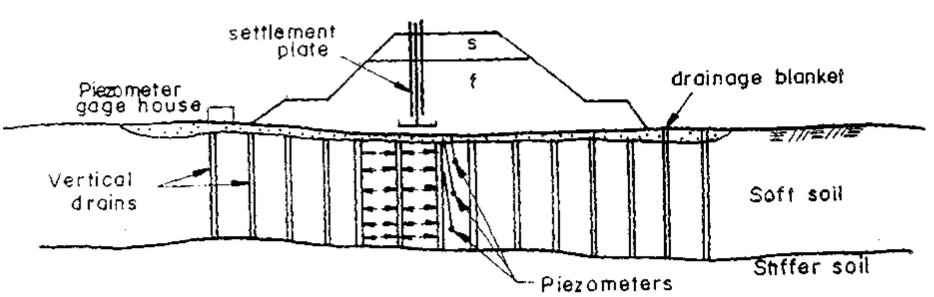
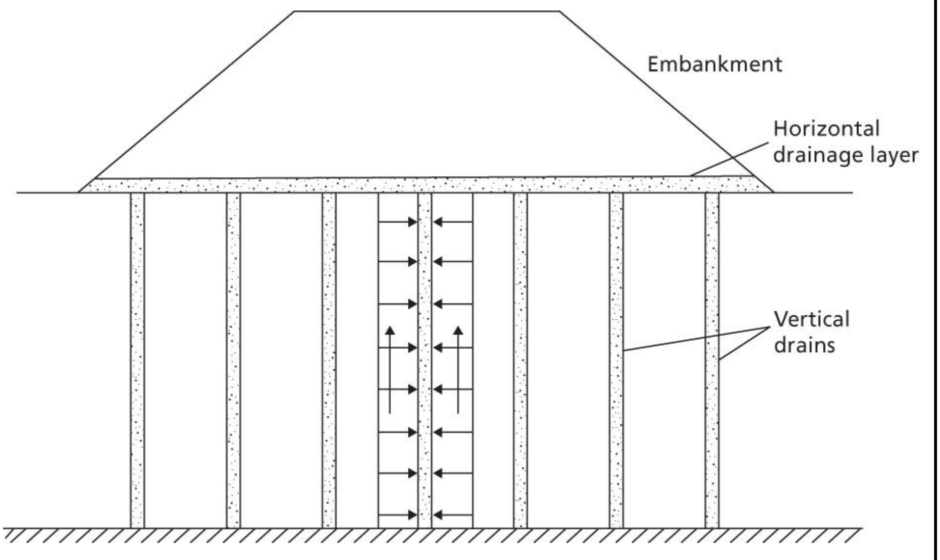
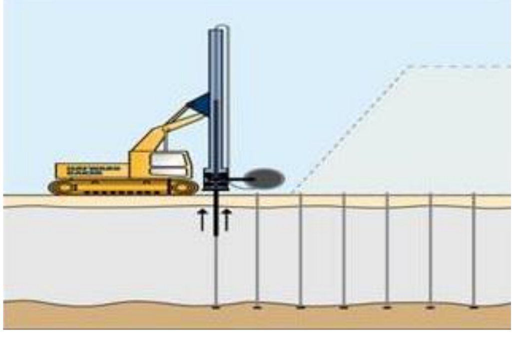
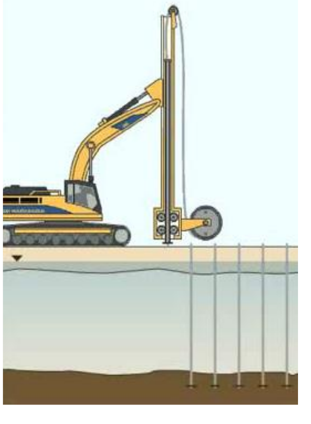
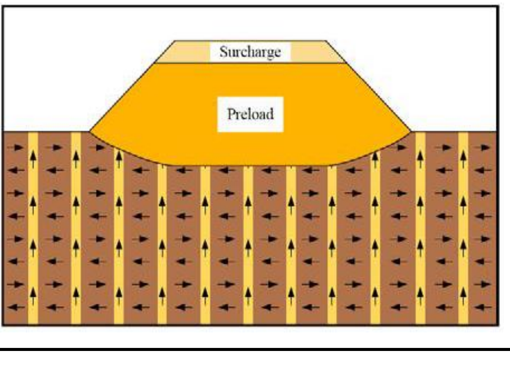


Figure 6. Use of Vertical Drains and Related Instrumentation


Embankment
Horizontal drainage layer
Vertical drains






Surcharge
Preload

THEORY OF CONSOLIDATION FOR RADIAL FLOW AND BOTH RADIAL-VERTICAL CONSOLIDATION (COMBINED) HAVE BEEN DEVELOPED FOR A LONG TIME (BARREN I948; CARILLO 1942). CONSOLIDATION TIME IS MAINLY AFFECTED BY THE DRAIN SPACING RATHER THAN THE DRAIN DIAMETER.

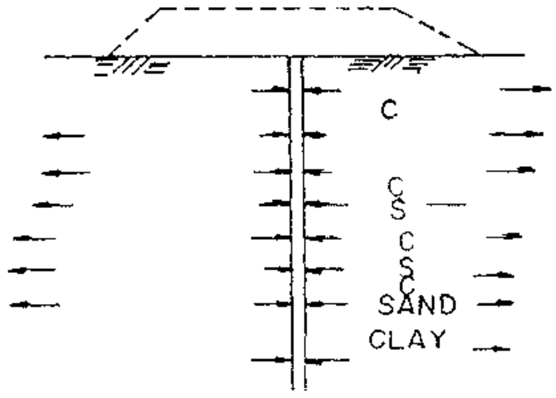
FOR SUCCESSFUL PROJECTS :

1. $(\sigma_{vo}' + \sigma_{vf}') > \sigma_{vo}'$ (PRECONSOLIDATION PRESSURE)
2. Primary settlement/(Primary Settlement+Secondary settlement) MUST BE LARGE (larger than 0.6).
3. THERE SHOULD NOT BE NATURAL DRAINAGE LAYERS.

THE NEED AND EFFICIENCY OF DRAINS ARE LARGELY DEPENDENT ON SOIL CHARACTERISTICS, SOIL PERMEABILITY AND COEFFICIENT OF CONSOLIDATION.

RECENT ALLUVIAL DEPOSITS CONTAIN FREQUENT HORIZONTAL BANDS OF SAND OR GRAVEL. THESE ARE USUALLY THIN AND VERY PERMEABLE COMPARED TO CLAYS.

1. HIGHLY PERMEABLE BANDS OR SEAMS GREATLY INCREASE EFFICIENCY OF DRAINS SINCE THEY ACT AS HORIZONTAL DRAINS CONNECTED TO MAIN ARTERIES.
2. CONTINUOUS AND FREQUENT SEAMS OR BANDS OF HIGH PERMEABILITY SOILS OFTEN MAKE VERTICAL DRAINS UNNECESSARY OR GREATLY REDUCE THEIR REAL EFFECTIVENESS.



Continuous and Frequent Permeable Soil Bands Often Make Drains Unnecessary

SOIL INVESTIGATIONS ARE VERY IMPORTANT

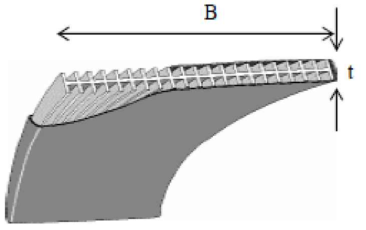
- CONTINUOUS SAMPLING
- LARGE DIAMETER (25-30 CM) LABORATORY CONSOLIDATION TESTS.

VERTICAL DRAINS

Originally sand drains were used for vertical drainage.

Now filter fabric and plastic (100 mm wide, 2 to 6 mm thick) prefabricated vertical drains are more commonly used. Key requirements for these drains are:

1. Permeability $k_{drain} > k_{soil}$
2. filter should not clog
3. fabric should not seal plastic channels
4. fabric should withstand biological and chemical attack



SOME FACTORS AFFECTING THE DRAIN PERFORMANCE :

1. SMEAR AND DISTORTION OF DRAIN WALLS WHICH REDUCE DRAIN PERMEABILITY.
2. DISTURBANCE AND LATERAL DEFORMATIONS OF SOFT GROUND RESULTING FROM DRAIN INSTALLATION. PERMEABILITY DECREASES, UNDRAINED SHEAR STRENGTH DECREASES AND PORE WATER PRESSURES INCREASE (ROWE, 1968)

SAND DRAINS

WIDELY USED BETWEEN 1930 -1980 WITH DIAMETERS 20 - 60 cm AND SPACING 1.5 TO 6 m.

CLOSED MANDREL METHOD : APPLIED BY PERCUSSION OR VIBRATION OR JETTING. THE TUBE IS PUSHED DISPLACING THE SOIL. THERE IS A LOOSE CAP AT THE END WHICH IS DETACHED AFTER PUSHING IS COMPLETE. THEN THE TUBE IS FILLED AND EXTRACTED. IN THIS METHOD THERE IS DISPLACEMENT AND DISTURBANCE WHICH RESULTS IN A DECREASE OF UNDRAINED SHEAR STRENGTH, PERMEABILITY.

OPEN MANDREL METHOD : SOIL IN THE TUBE IS REMOVED BY JETTING OR AUGERING. THE PROBLEM OF SMEAR STILL EXISTS. AUGER METHOD USING SOLID STEM OR HOLLOW STEM AUGERS WHICH IS A NON-DISPLACEMENT METHOD MAY BE CONSIDERED AS THE BEST AS COMPARED TO THE OTHERS. ROTARY JETTING METHOD MAY ALSO BE APPLIED.

SAND-WICKS

THESE ARE READY-MADE SMALL DIAMETER (APPROX. 10 CM DIAMETER) SAND DRAINS WHICH ARE CONTAINED IN LONG CANVAS BAGS.

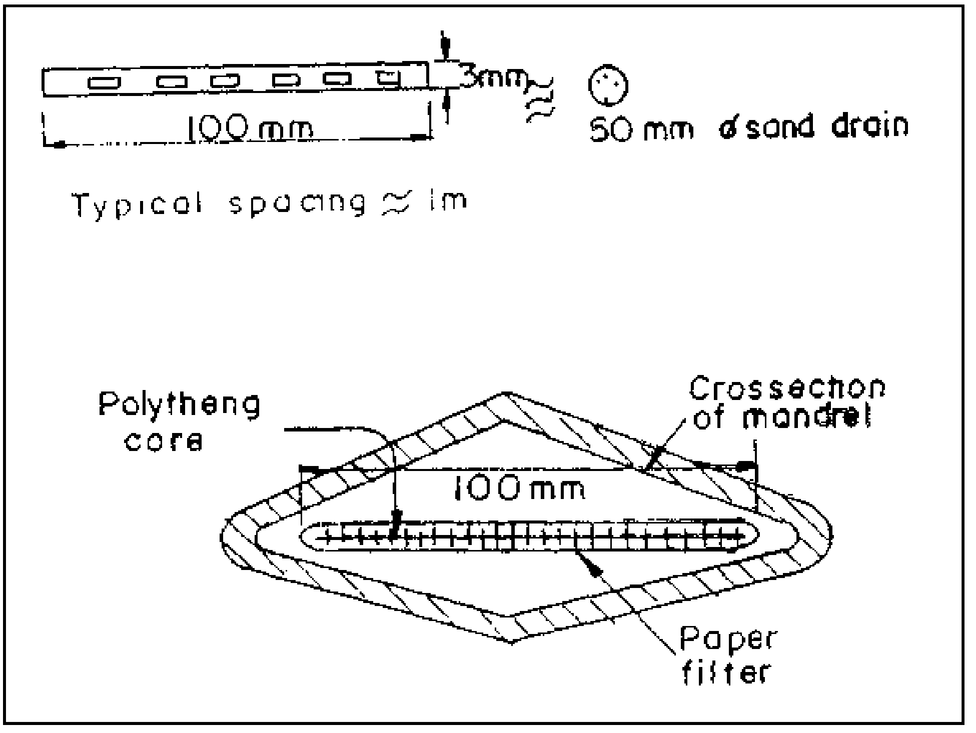
THEY ARE USUALLY INSTALLED BY CLOSE MANDREL TECHNIQUE. THEY ARE RELATIVELY CHEAP AND FIRST USED IN INDIA BY DASTIDAR ET AL. (1969) AND THEN BY SUBBARAJU ETAL. (1973).

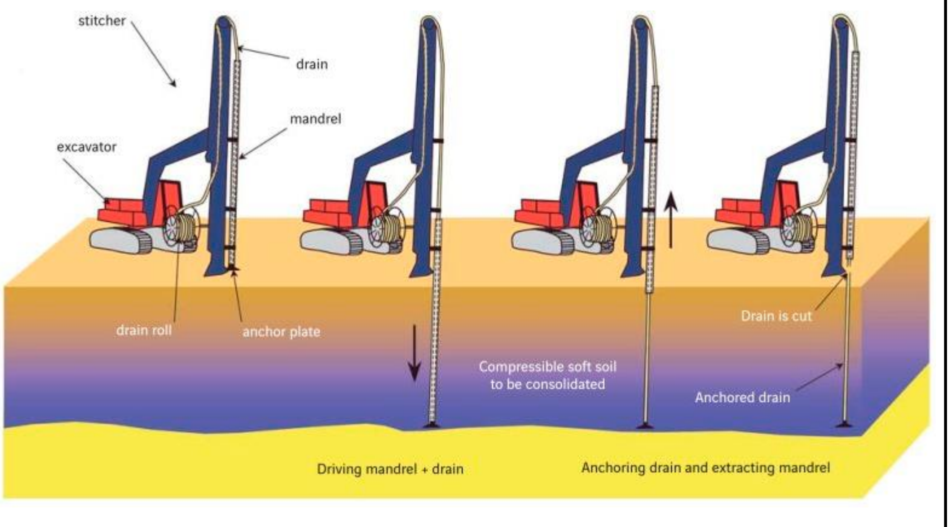
PREFABRICATED VERTICAL DRAINS (PVD)

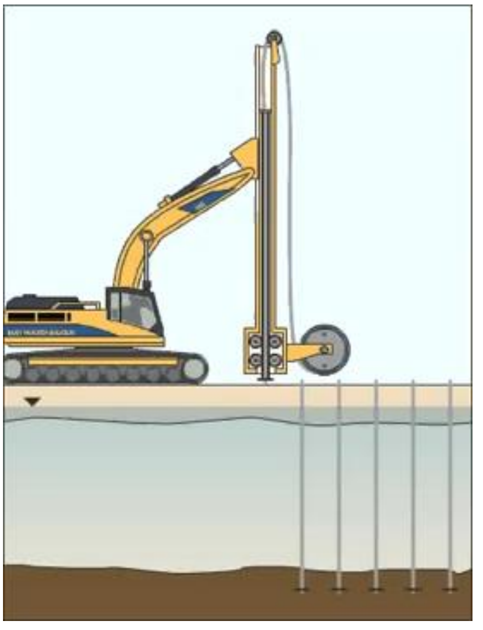
THEY ARE FIRST TRIED IN 1937 AND 1948 BY KJELLMAN. THERE ARE DYNAMIC AND STATIC METHODS OF INSTALLATION. DRAINS ARE DRIVEN INTO THE GROUND BY PURPOSE-MADE MANDREL WHICH IS THEN REMOVED. THE ADVANTAGES CAN BE LISTED AS FOLLOWS:

- THEY ARE EASY TO INSTALL
- THEY CAN BE SPACED CLOSELY
- THEY HAVE LONG LIFE
- THEY HAVE THE ABILITY TO RESIST LARGE DEFORMATIONS.

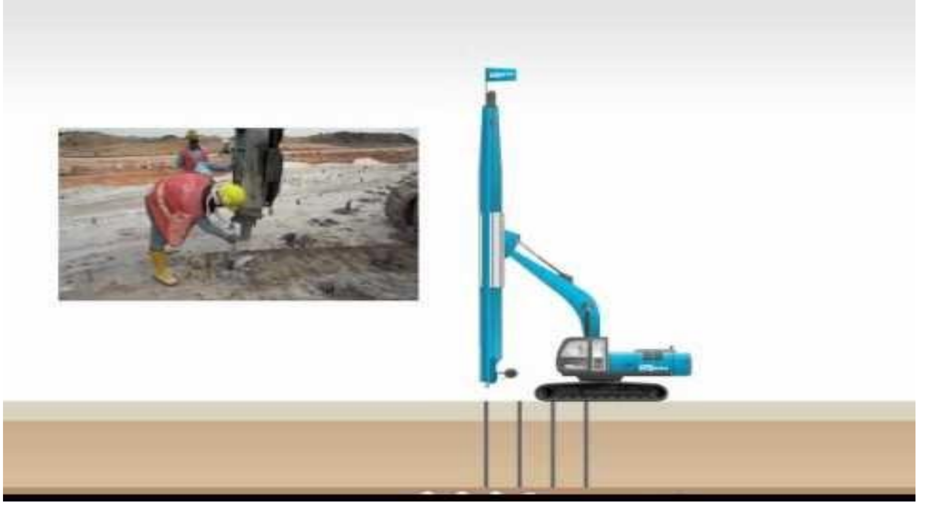
CONSIST OF A CORE PLASTIC AND FILTER SLEEVE OF GEOTEXTILE.


3mm
100 mm
50 mm ø sand drain
Typical spacing ≈ 1m
Polytheng core
Crossection of mandrel
100 mm
Paper filter


stitcher
drain
mandrel
excavator
drain roll
anchor plate
Drain is cut
Compressible soft soil to be consolidated
Anchored drain
Driving mandrel + drain
Anchoring drain and extracting mandrel



https://www.youtube.com/watch?v=W5Uc0-EtOn4



https://www.youtube.com/watch?v=WP-4_5gMb14

https://www.youtube.com/watch?v=eTGa0fG9HWY

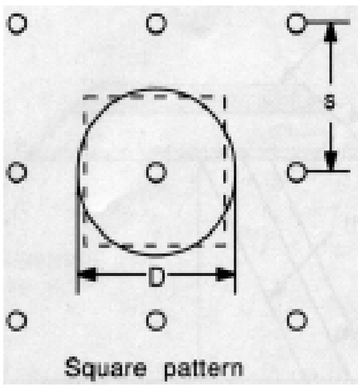
DESIGN OF VERTICAL DRAINS

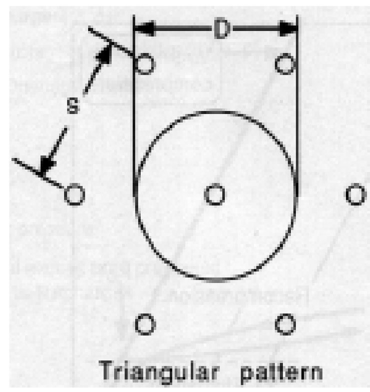
THE MAIN ASSUMPTIONS MADE FOR THE DESIGN OF VERTICAL DRAINS ARE ;

- EACH DRAIN IS INDEPENDENT AT THE CENTRE OF A CYLINDRICAL SOIL MASS AND IS ONLY AFFECTED BY THE DRAINAGE OF THE SOIL IN IT.

-INSTANTANEOUS LOADING OF THE HOMOGENEOUS SOIL RESULTS IN SOLELY RADIAL CONSOLIDATION (AND THEREFORE RADIAL FLOW) UNDER CONDITIONS OF CONSTANT PERMEABILITY (k_h) AND RADIAL CONSOLIDATION COEFFICIENT (c_h)

The drains are placed in either a square or triangular pattern, and this has an effect on the calculation.





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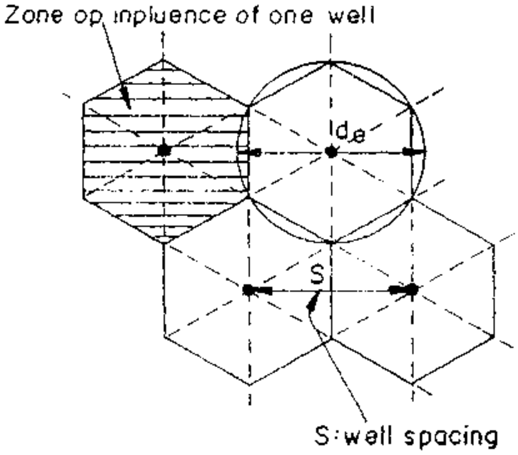
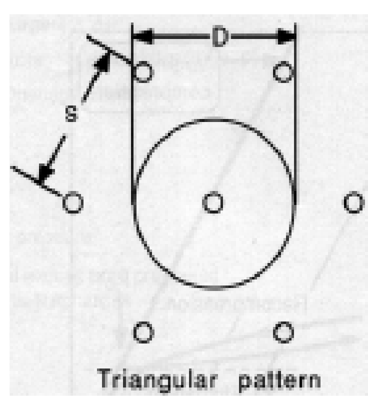
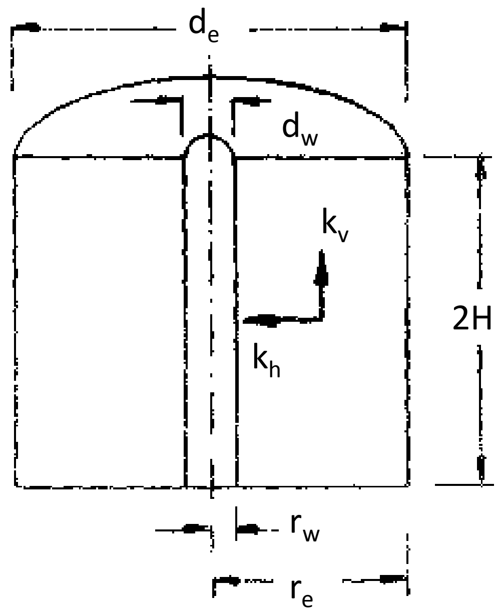


Triangular pattern





$$U_{radial} = U_{horizontal} = 1 - e^{\frac{-8 \cdot T_h}{F(n)}}$$

$$T_r = T_h = \frac{c_h \cdot t}{(d_e)^2}$$

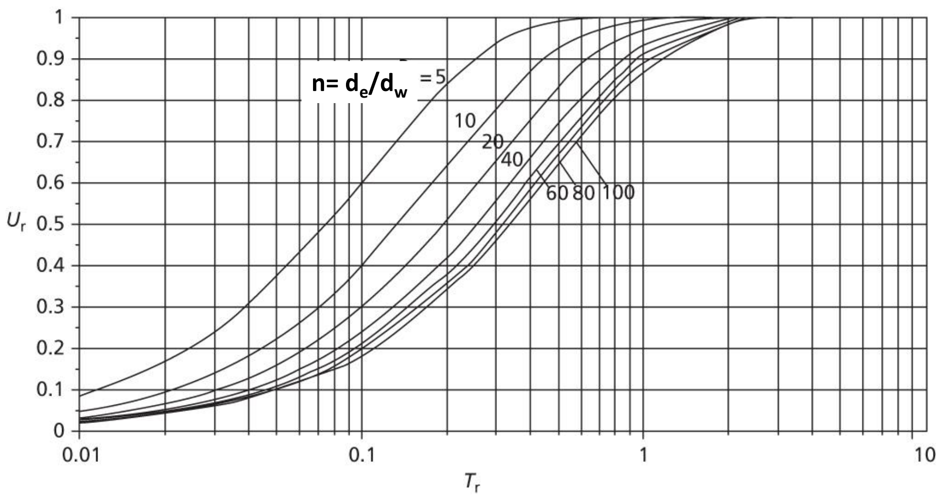
U_r	average degree of radial consolidation
T_r	time factor
c_h	coefficient of horizontal consolidation
d_e	equivalent diameter of soil around drain
	1.06S for triangular pattern, 1.13S for square pattern

c_h is dependent on horizontal permeability k_h. Usually c_h/c_v = range of 2 to 10.

S = drain spacing

n= d_e/d_w where d_w is drain diameter or equivalent diam. of PVD

F(n)= $\frac{n^2}{n^2-1}\left(\ln n - \frac{3}{4} + \frac{1}{n^2} + \frac{1}{n^4}\right)$ **OR ≅ F(n) = ln(n) - 0.75**

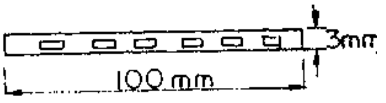


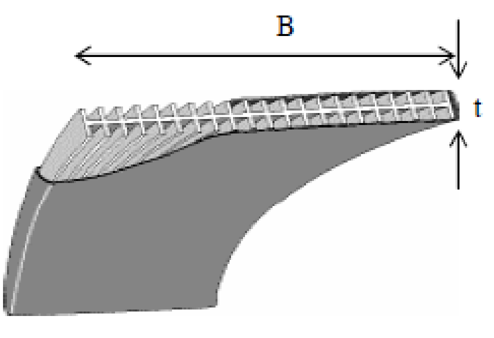
THE EQUIVALENT DIAMETER d_w OF A BAND SHAPED DRAIN

WIDTH B, AND THICKNESS t, is
CALCULATED BY USING THE RELATIONSHIP GIVEN BELOW

$$d_w = \frac{2(B+t)}{\pi}$$

Equal circumference





Since, more than likely, vertical and radial drainage will occur simultaneously, the combined effects can be quantified as follows:

$$(1-U) = (1-U_v)\cdot(1-U_r)$$

Where U is the average degree of consolidation under combined vertical and radial drainage

Installation Effects (SMEAR EFFECTS)

The values of soil properties for the soil immediately surrounding the drains may be significantly reduced due to remolding during installation, especially if boring is used, an effect known as **smear effect**.

The smear effect can be taken into account either by assuming a reduced value of c_h or by using a reduced drain diameter.

Alternatively, if the extent and permeability (k_s) of the smeared material are known, or can be estimated, the F(n) expression can be modified such as (Hansbo, 1979):

$$\textbf{F(n)=} \approx \ln\frac{n}{S} + \frac{k}{k_s}\ln S - \frac{3}{4}$$

S: drain spacing

SMEAR EFFECT DURING INSTALLATION MAY CAUSE THE ACTUAL TIMES FOR CONSOLIDATION TO BE GREATER THAN PREDICTED BY THE ABOVE EQUATIONS.

Example:

A wide embankment is to be constructed over a layer of clay 10-m thick, with an impermeable lower boundary. Construction of embankment will increase the total vertical stress in the clay layer by 65 kPa. For the clay, c_v=4.7 m^2/year, c_h=7.9 m^2/year and m_v=0.25 m^2/MN. The design requirement is that all but 25 mm of the settlement due to consolidation of the clay layer will have taken place after 6 months. Determine the spacing, in a square pattern, of 400-mm diameter sand drains to achieve the above requirement.

Final settlement

$$S = m_v \cdot H \cdot \Delta\sigma' = 0.25(\text{m}^2/\text{MN}) \cdot 10\ (\text{m}) \cdot 65\ (\text{kPa}) = 162\ \text{mm}$$

For t=6 months

$$U = \frac{162-25}{162} = 0.85$$

For vertical drainage only, the layer is half-closed, therefore d=10 m

$$T_v = \frac{c_v \cdot t}{d^2} = \frac{4.7\ \text{m}^2/\text{yr} \cdot (6/12)\ \text{yr}}{10^2} = 0.0235 \qquad U_v = 17\%$$

$$\begin{cases} \text{for U} < 60\%,\ T_v = \frac{\pi}{4} \cdot U^2 \\ \text{for U} > 60\%,\ T_v = -0.933 \cdot \log(1-U) - 0.085 \end{cases}$$

U (%)	T_v
0	0
5	0.002
10	0.008
15	0.018
20	0.031
25	0.049
30	0.071

$$(1-U) = (1-U_v) \cdot (1-U_r)$$

$$(1-0.85) = (1-0.17) \cdot (1-U_r) \qquad \boxed{U_r = 0.82}$$

$$U_{radial} = 1 - e^{\frac{-8 \cdot T_h}{F(n)}} \qquad 0.82 = 1 - e^{\frac{-8 \cdot T_h}{F(n)}}$$

For radial drainage, the diameter of the sand drains is 0.4 m

$$T_h = \frac{c_h \cdot t}{(d_e)^2} = \frac{7.9\ \text{m}^2/\text{yr} \cdot (6/12)\ \text{yr}}{(0.40 \cdot n)^2}$$

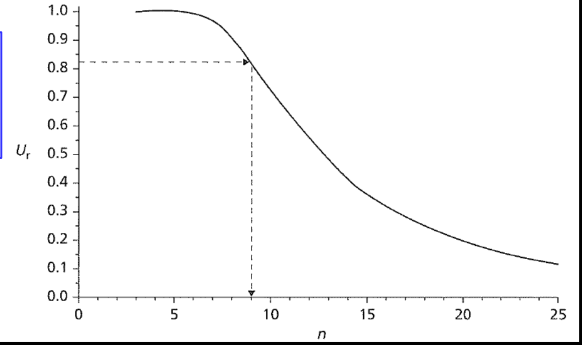
d_e = equivalent diameter of soil around drain

n= $d_e/d_w = d_e / 0.40$ $\qquad$ d_e= 0.40 x n

F(n) = ln(n) - 0.75

$$\boxed{0.82 = 1 - e^{\frac{-8 \cdot \left(\frac{3.95}{(0.40 \cdot n)^2}\right)}{\ln(n) - 0.75}}}$$

Find n by trial and error:
n=9



n=9

n= d_e/d_w = 1.13 S / 0.40

d_e = equivalent diameter of soil around drain
d_e =1.13 S for square pattern, where S is drain spacing

Drain spacing, S = 3.2 m

Example:

A highway bridge will cause a permanent stress increase of 115 kPa. What surcharge is required to eliminate total bridge settlement in 9 months for a double drained layer and sand drains.

Height of clay layer = 6 m
C_c= 0.28
e_o= 0.9
C_v=C_r=C_{vr}=0.36 m^2/month
initial effective vertical stress at the mid-depth of clay 210 kPa
Drain diameter = 0.2 m
Drain spacing = 2.65 m, square pattern

$$S_{bridge} = \frac{C_c}{1+e_o} \cdot H \cdot \log(\frac{\sigma'_f}{\sigma'_o}) = \frac{0.28}{1+0.9} \cdot 6 \cdot \log(\frac{210+115}{210}) = 0.168 \text{ m}$$

$$S_{surcharge} = \frac{0.28}{1+0.9} \cdot 6 \cdot \log(\frac{x?}{210})$$

$$U\% = \frac{S_{bridge}}{S_{surcharge}} \qquad (1-U) = (1-U_v) \cdot (1-U_r)$$

Radial drainage:

$$T_h = \frac{c_h \cdot t}{(d_e)^2} = \frac{0.36 \text{ m}^2/\text{mo} \cdot (9)}{(3)^2} = 0.36$$

d_e = equivalent diameter of soil around drain
d_e =1.13 S for square pattern, where S is drain spacing
d_e =1.13 x 2.65 m = 3 m

F(n) = ln(n) - 0.75

n= d_e/d_w = 3 / 0.20 = 15

d_w is drain diameter = 0.2 m

F(n) = ln(15) - 0.75=1.958

$$U_{radial} = 1 - e^{\frac{-8 \cdot T_h}{F(n)}} = 1 - e^{\frac{-8 \cdot (0.36)}{1.958}} = 1 - e^{-1.47} = 77\%$$

Vertical drainage:

$$T_v = \frac{c_v \cdot t}{(d)^2} = \frac{0.36 \text{ m}^2/\text{mo} \cdot (9)}{(3)^2} = 0.36$$

$$\begin{cases} \text{for U} < 60\%, \ T_v = \frac{\pi}{4} \cdot U^2 \\ \text{for U} > 60\%, \ T_v = -0.933 \cdot \log(1-U) - 0.085 \end{cases}$$

$$U_v = 0.667 = 66.7\%$$

U (%)	T_v
35	0.096
40	0.126
45	0.159
50	0.195
55	0.239
60	0.286
65	0.340

U (%)	T_v
70	0.403
75	0.477
80	0.567
85	0.684
90	0.848
95	1.129

$$(1-U) = (1-U_v)\cdot(1-U_r)$$
$$(1-U) = (1-0.667)\cdot(1-0.77)$$
$$U = 92.3\%$$
$$U\% = \frac{S_{bridge}}{S_{surcharge}} = 0.923 = \frac{0.168}{\frac{0.28}{1+0.9}\cdot 6\cdot \log(\frac{x?}{210})}$$
$$x? = 337 \text{ kPa}$$
$$x? = 337 \text{ kPa} = \sigma'_o + \text{surcharge} = 210 + \text{surcharge}$$
$$\text{surcharge} = 127 \text{ kPa}$$