14.533 Advanced Foundation Engineering
Fall 2013

SHORT & LONG TERM SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS

Class Notes

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Settlement Criteria and Concept of Analysis

(text Sections 5.1 through 5.20, pp. 283- 285)

1. **Tolerance Criteria of Settlement and Differential Settlement**
   - Settlement most often governs the design as allowable settlement is exceeded before B.C. becomes critical.
   - Concerns of foundation settlement are subdivided into 3 levels of associated damage:
     - Architectural damage - cracks in walls, partitions, etc.
     - Structural damage - reduced strength in structural members
     - Functional damage - impairment of the structure functionality

     The last two refer to stress and serviceability limit states, respectively.

Settlement Criteria and Concept of Analysis

1. **Tolerance Criteria of Settlement and Differential Settlement (cont'd.)**

- In principle, two approaches exist to determine the allowable displacements.

  (a) Rational Approach to Design

  Design Building → Determine allowable deformation & displacements → Design found. accordingly → Check cost

  Check cost → not acceptable → Design Building

  Check cost → ok

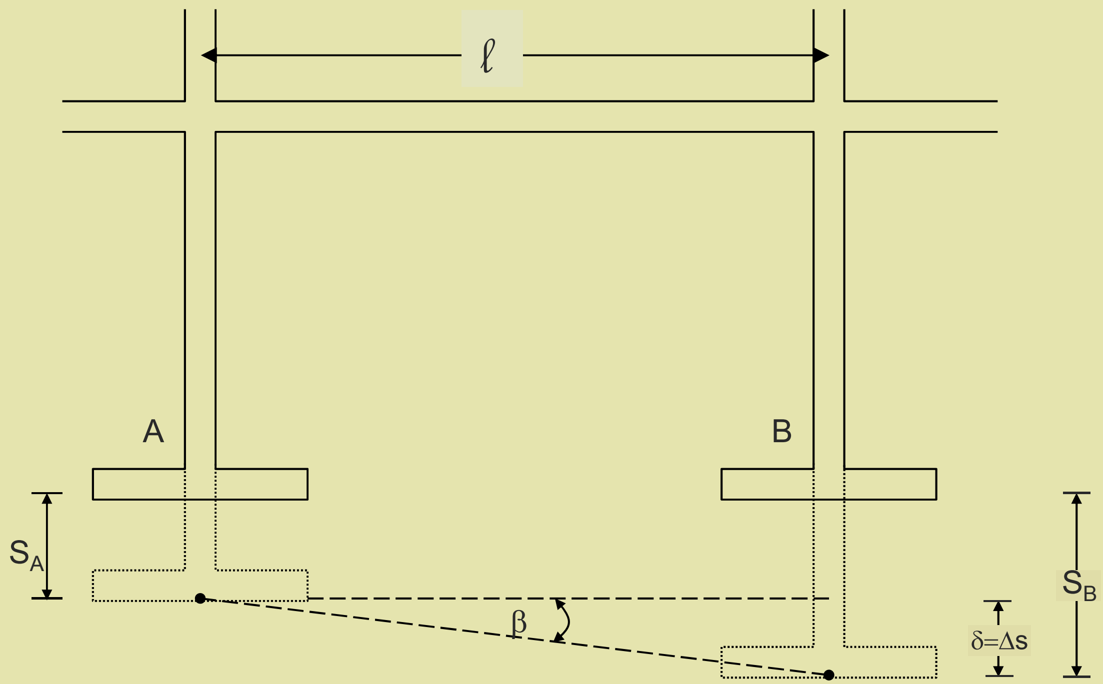
Problems: - expensive analysis
- limited accuracy in all predictions especially settlement & differential settlement

Settlement Criteria and Concept of Analysis

1. **Tolerance Criteria of Settlement and Differential Settlement (cont'd.)**

   (b) Empirical Approach (see text section 5.20, "Tolerable Settlement of Buildings", pp. 283-285)

   - based on performance of many structures, provide a guideline for maximum settlement and maximum rotation



- S_{max} = maximum settlement
- $\delta = \Delta s$ = differential settlement (between any two points)
- $\left(\frac{\delta}{\ell}\right)_{max}$ = maximum rotation

Settlement Criteria and Concept of Analysis

1. **Tolerance Criteria of Settlement and Differential Settlement (cont'd.)**

   (b) Empirical Approach

Angular Distortion = $\tan\beta = \left(\frac{\Delta s}{\ell}\right)_{max} = \frac{\delta}{\ell} = \frac{s_A - s_B}{\ell}$

$\left(\frac{\delta}{\ell}\right)_{max} \geq \frac{1}{300}$ architectural damage

$\left(\frac{\delta}{\ell}\right)_{max} \geq \frac{1}{250}$ tilting of high structures become visible

$\left(\frac{\delta}{\ell}\right)_{max} \geq \frac{1}{150}$ structural damage likely

Settlement Criteria and Concept of Analysis

1. **Tolerance Criteria of Settlement and Differential Settlement (cont'd.)**

   (b) Empirical Approach

maximum settlement (S_{max}) leading to differential settlement

- Masonry wall structure 1 - 2"
- Framed structures 2 - 4"
- Silos, mats 3 - 12"
- Lambe and Whitman "Soil Mechanics" provides in Table 14.1 and Figure 14.8 (see next page) the allowable maximum total settlement, tilting and differential movements as well as limiting angular distortions.

Settlement Criteria and Concept of Analysis

1. **Tolerance Criteria of Settlement and Differential Settlement (cont'd.)**

Correlation Between Maximum Settlement to Angular Distortion

Grant, Christian & Van marke (ASCE - 1974)
correlation between angular settlement to maximum settlement, based on 95 buildings of which 56 were damaged.

Type of Found	Type of Soil	$\frac{s_{max}(in)}{\left(\delta/\ell\right)_{max}}$	$\frac{\rho_{all}\,(in)}{\left(\delta/\ell\right)_{max}=1/300}$
Isol. Footings	Clay	1200	4"
	Sand	600	2"
Mat	Clay	≥ 138 ft	≥ 0.044 B (ft)
	Sand	no relationship	

Limiting values of serviceability are typically s_{max} = 1 ″ for isolated footing and s_{max} = 2″ for a raft which is more conservative than the above limit based on architectural damage. Practically serviceability needs to be connected to the functionality of the building and the tolerable limit.

Settlement Criteria and Concept of Analysis

(Lambe & Whitman, *Soil Mechanics)*

200 PART III DRY SOIL

Table 14.1 Allowable Settlement

Type of Movement	Limiting Factor	Maximum Settlement
Total settlement	Drainage	6–12 in.
	Access	12–24 in.
	Probability of nonuniform settlement:	
	Masonry walled structure	1–2 in.
	Framed structures	2–4 in.
	Smokestacks, silos, mats	3–12 in.
Tilting	Stability against overturning	Depends on height and width
	Tilting of smokestacks, towers	0.004*l*
	Rolling of trucks, etc.	0.01*l*
	Stacking of goods	0.01*l*
	Machine operation-cotton loom	0.003*l*
	Machine operation-turbogenerator	0.0002*l*
	Crane rails	0.003*l*
	Drainage of floors	0.01–0.02*l*
Differential movement	High continuous brick walls	0.0005–0.001*l*
	One-story brick mill building, wall cracking	0.001–0.002*l*
	Plaster cracking (gypsum)	0.001*l*
	Reinforced-concrete building frame	0.0025–0.004*l*
	Reinforced-concrete building curtain walls	0.003*l*
	Steel frame, continuous	0.002*l*
	Simple steel frame	0.005*l*

From Sowers, 1962.
Note. *l* = distance between adjacent columns that settle different amounts, or between any two points that settle differently. Higher values are for regular settlements and more tolerant structures. Lower values are for irregular settlements and critical structures.

202 PART III DRY SOIL

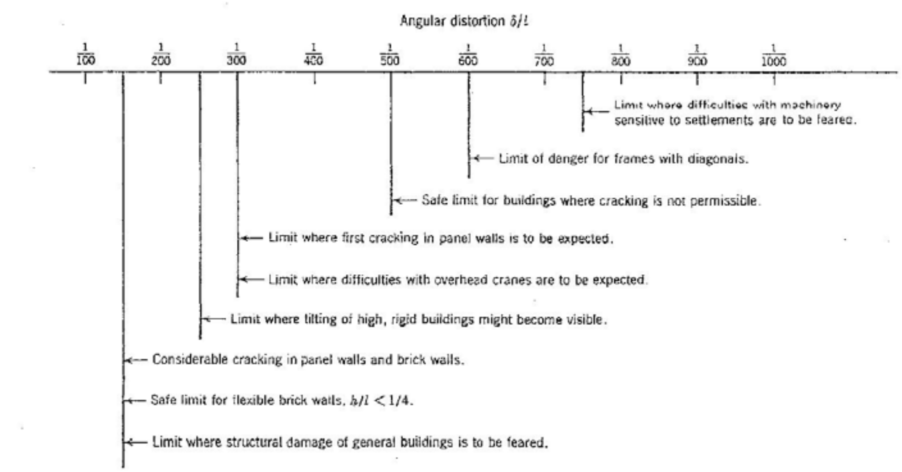
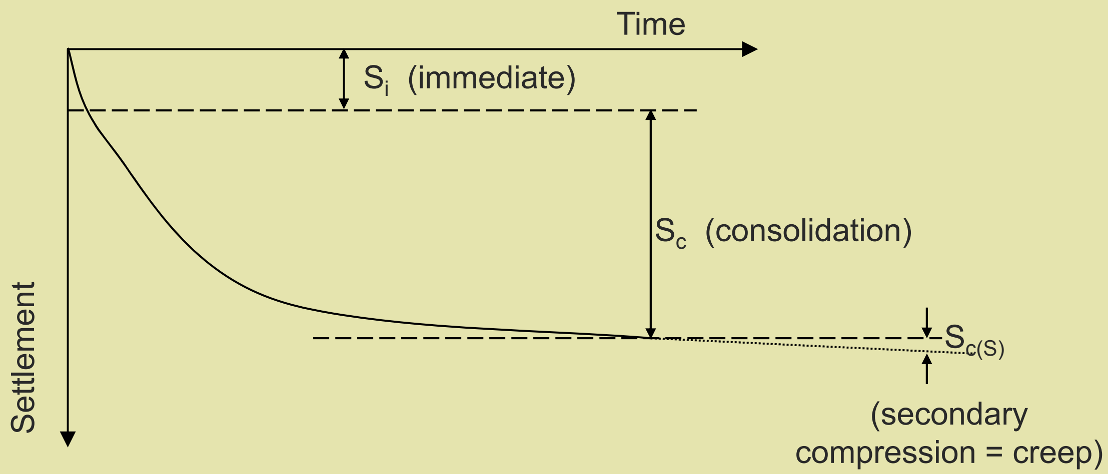


Fig. 14.8 Limiting angular distortions (From Bjerrum, 1963a).

Settlement Criteria and Concept of Analysis

2. Types of Settlement and Methods of Analysis



Si = Granular Soils → Elastic Theory

S_c, $S_{c(s)}$ - Cohesive Soils

S_c → Consolidation Theory; S_c, $S_{c(s)}$ → Empirical Correlations

In principle, both types of settlement; the immediate and the long term, utilize the compressibility of the soil, one however, is time dependent (consolidation and secondary compression).

Settlement Criteria and Concept of Analysis

3. General Concept of Settlement Analysis

Two controlling factors influencing settlements:

- Net applied stress - Δq
- Compressibility of soil - $c = \left({}^{settlement}/_{load}\right)$

when dealing with clay c = f (t) as it changes with time

$$s = \Delta q \times c \times f(B)$$

where	s = settlement	[L]
	Δq = net load	$[F/L^2]$
	c = compressibility	$[L/(F/L^2)]$
	f (B) = size effect	[dimensionless]

obtain c by $\rightarrow$ lab tests, plate L.T., SPT, CPT

c will be influenced by:
- width of footing = B
- depth of footing =
- location of G.W. Table =
- type of loading $\rightarrow$ static or repeated
- soil type & quality affecting the modulus

Vertical Stress Increase in the Soil Due to a Foundation Load

Das 7th ed., Sections 5.2 – 5.6 (pp. 224 - 239)
Bowles sections 5.2 – 5.5 (pp. 286-302)

1. Principle

(a) Required: Vertical stress (pressure) increase under the footing in order to asses settlement.

(b) Solution: Theoretical solution based on theory of elasticity assuming load on ∞, homogeneous, isotropic, elastic half space.

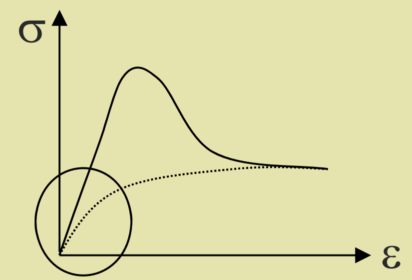
- Homogeneous — Uniform throughout at every point we have the same qualities.
- Isotropic — Identical in all directions, invariant with respect to direction
- Orthotropic — (tend to grow or form along a vertical axis) different qualities in two planes
- Elastic — capable of recovering shape

Vertical Stress Increase in the Soil Due to a Foundation Load

1. **Principle (cont'd.)**

(c) Why can we use the elastic solutions for that problem?

- Is the soil elastic?
  no, but…



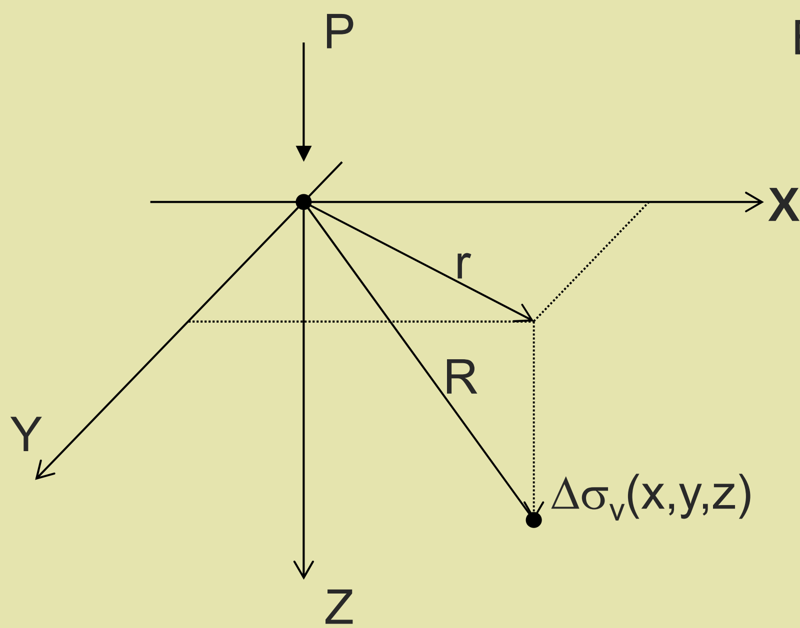
i. We are practically interested in the service loads which are approximately the dead load.
   - The ultimate load = design load x F.S.
   - Design load = (DL x F.S.) + (LL x F.S.)
   - Service load ≅ DL → within the elastic zone

ii. The only simple straight forward method we know

Vertical Stress Increase in the Soil Due to a Foundation Load

2. **Stress due to Concentrated Load**

Boussinesq, 1885



$$\Delta p = \Delta\sigma_v = \frac{3P}{2\pi z^2\left[1+\left(r/z\right)^2\right]^{5/2}} \qquad r = \sqrt{x^2+y^2} \qquad \text{(eq. 5.1)}$$

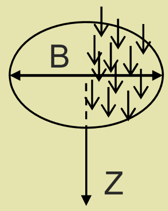
Vertical Stress Increase in the Soil Due to a Foundation Load

3. Stress due to a Circularly Loaded Area

- referring to flexible areas as we assume uniform stress over the area. Uniform stress will develop only under a flexible footing.
- integration of the above load from a point to an area.
  - see equations 5.2, 5.3 (text 225)

$$\Delta p = \Delta\sigma_v = q_0\left\{1 - \frac{1}{\left[1+\left(B/2z\right)^2\right]^{3/2}}\right\}$$

vertical stress under the center

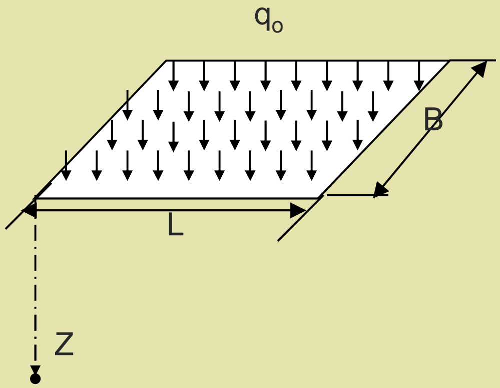


see Table 5.1 (p.226) for $\frac{\Delta\sigma_v}{q_0} = f\left(\frac{r}{(B/2)} \& \frac{z}{(B/2)}\right)$

Vertical Stress Increase in the Soil Due to a Foundation Load

4. **Stress Below a Rectangular Area**

$\Delta p = \Delta\sigma_v = q_o \times I$



below **the corner** of a flexible rectangular loaded area

$m = B/z$ $n = L/z$

Table 5.2 (p.228-229) $\rightarrow$ I = f (m,n)

Corner of a Foundation

Table 5.2 Variation of Influence Value I [Eq. (5.6)][a]

	n											
m	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4
0.1	0.00470	0.00917	0.01323	0.01678	0.01978	0.02223	0.02420	0.02576	0.02698	0.02794	0.02926	0.03007
0.2	0.00917	0.01790	0.02585	0.03280	0.03866	0.04348	0.04735	0.05042	0.05283	0.05471	0.05733	0.05894
0.3	0.01323	0.02585	0.03735	0.04742	0.05593	0.06294	0.06858	0.07308	0.07661	0.07938	0.08323	0.08561
0.4	0.01678	0.03280	0.04742	0.06024	0.07111	0.08009	0.08734	0.09314	0.09770	0.10129	0.10631	0.10941
0.5	0.01978	0.03866	0.05593	0.07111	0.08403	0.09473	0.10340	0.11035	0.11584	0.12018	0.12626	0.13003
0.6	0.02223	0.04348	0.06294	0.08009	0.09473	0.10688	0.11679	0.12474	0.13105	0.13605	0.14309	0.14749
0.7	0.02420	0.04735	0.06858	0.08734	0.10340	0.11679	0.12772	0.13653	0.14356	0.14914	0.15703	0.16199
0.8	0.02576	0.05042	0.07308	0.09314	0.11035	0.12474	0.13653	0.14607	0.15371	0.15978	0.16843	0.17389
0.9	0.02698	0.05283	0.07661	0.09770	0.11584	0.13105	0.14356	0.15371	0.16185	0.16835	0.17766	0.18357
1.0	0.02794	0.05471	0.07938	0.10129	0.12018	0.13605	0.14914	0.15978	0.16835	0.17522	0.18508	0.19139
1.2	0.02926	0.05733	0.08323	0.10631	0.12626	0.14309	0.15703	0.16843	0.17766	0.18508	0.19584	0.20278
1.4	0.03007	0.05894	0.08561	0.10941	0.13003	0.14749	0.16199	0.17389	0.18357	0.19139	0.20278	0.21020
1.6	0.03058	0.05994	0.08709	0.11135	0.13241	0.15028	0.16515	0.17739	0.18737	0.19546	0.20731	0.21510
1.8	0.03090	0.06058	0.08804	0.11260	0.13395	0.15207	0.16720	0.17967	0.18986	0.19814	0.21032	0.21836
2.0	0.03111	0.06100	0.08867	0.11342	0.13496	0.15326	0.16856	0.18119	0.19152	0.19994	0.21235	0.22058
2.5	0.03138	0.06155	0.08948	0.11450	0.13628	0.15483	0.17036	0.18321	0.19375	0.20236	0.21512	0.22364
3.0	0.03150	0.06178	0.08982	0.11495	0.13684	0.15550	0.17113	0.18407	0.19470	0.20341	0.21633	0.22499
4.0	0.03158	0.06194	0.09007	0.11527	0.13724	0.15598	0.17168	0.18469	0.19540	0.20417	0.21722	0.22600
5.0	0.03160	0.06199	0.09014	0.11537	0.13737	0.15612	0.17185	0.18488	0.19561	0.20440	0.21749	0.22632
6.0	0.03161	0.06201	0.09017	0.11541	0.13741	0.15617	0.17191	0.18496	0.19569	0.20449	0.21760	0.22644
8.0	0.03162	0.06202	0.09018	0.11543	0.13744	0.15621	0.17195	0.18500	0.19574	0.20455	0.21767	0.22652
10.0	0.03162	0.06202	0.09019	0.11544	0.13745	0.15622	0.17196	0.18502	0.19576	0.20457	0.21769	0.22654
∞	0.03162	0.06202	0.09019	0.11544	0.13745	0.15623	0.17197	0.18502	0.19577	0.20458	0.21770	0.22656

Vertical Stress Increase in the Soil Due to a Foundation Load

4. Stress Below a Rectangular Area (cont'd.)

Stress at a point under different locations

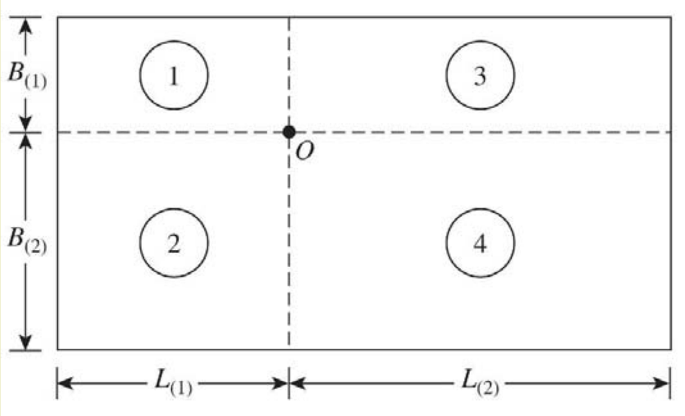


Figure 5.4 Stress below any point of a loaded flexible rectangular area (text p.196)

$$\Delta p = \Delta\sigma_v = q_o\,(I_1 + I_2 + I_3 + I_4)$$

use
$B_1 \times L_1 \rightarrow m_1, n_1 \rightarrow I_2$
$B_1 \times L_2 \rightarrow m_1, n_2 \rightarrow I_1$
$B_2 \times L_1 \rightarrow m_2, n_1 \rightarrow I_3$
$B_2 \times L_2 \rightarrow m_2, n_2 \rightarrow I_4$

Stress at a point under **the center** of the foundation

$$\Delta p = \Delta\sigma_v = q_c \times I_c$$

$I_c = f(m_1, n_1)$ $\qquad m_1 = L/B$ $\qquad n_1 = z/(B/2)$

- Table 5.3 (p.230) provides values of m_1 and n_1.
- See next page for a chart $\Delta p/q_0$ vs. z/B, $f(L/B)$

Center of a Foundation

Table 5.3 Variation of I_c with m_1 and n_1

n_1	m_1 = 1	2	3	4	5	6	7	8	9	10
0.20	0.994	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997	0.997
0.40	0.960	0.976	0.977	0.977	0.977	0.977	0.977	0.977	0.977	0.977
0.60	0.892	0.932	0.936	0.936	0.937	0.937	0.937	0.937	0.937	0.937
0.80	0.800	0.870	0.878	0.880	0.881	0.881	0.881	0.881	0.881	0.881
1.00	0.701	0.800	0.814	0.817	0.818	0.818	0.818	0.818	0.818	0.818
1.20	0.606	0.727	0.748	0.753	0.754	0.755	0.755	0.755	0.755	0.755
1.40	0.522	0.658	0.685	0.692	0.694	0.695	0.695	0.696	0.696	0.696
1.60	0.449	0.593	0.627	0.636	0.639	0.640	0.641	0.641	0.641	0.642
1.80	0.388	0.534	0.573	0.585	0.590	0.591	0.592	0.592	0.593	0.593
2.00	0.336	0.481	0.525	0.540	0.545	0.547	0.548	0.549	0.549	0.549
3.00	0.179	0.293	0.348	0.373	0.384	0.389	0.392	0.393	0.394	0.395
4.00	0.108	0.190	0.241	0.269	0.285	0.293	0.298	0.301	0.302	0.303
5.00	0.072	0.131	0.174	0.202	0.219	0.229	0.236	0.240	0.242	0.244
6.00	0.051	0.095	0.130	0.155	0.172	0.184	0.192	0.197	0.200	0.202
7.00	0.038	0.072	0.100	0.122	0.139	0.150	0.158	0.164	0.168	0.171
8.00	0.029	0.056	0.079	0.098	0.113	0.125	0.133	0.139	0.144	0.147
9.00	0.023	0.045	0.064	0.081	0.094	0.105	0.113	0.119	0.124	0.128
10.00	0.019	0.037	0.053	0.067	0.079	0.089	0.097	0.103	0.108	0.112

Vertical Stress Increase in the Soil Due to a Foundation Load

5. General Charts of Stress Distribution Beneath Rectangular and Strip Footings

(a) → $^{\Delta P}/_{q_0}$ vs. $^{z}/_{B}$ under the center of a rectangular footing with = 1 (square) to = ∞ (strip)

Stress Increase in a Soil Mass Caused by Foundation Load

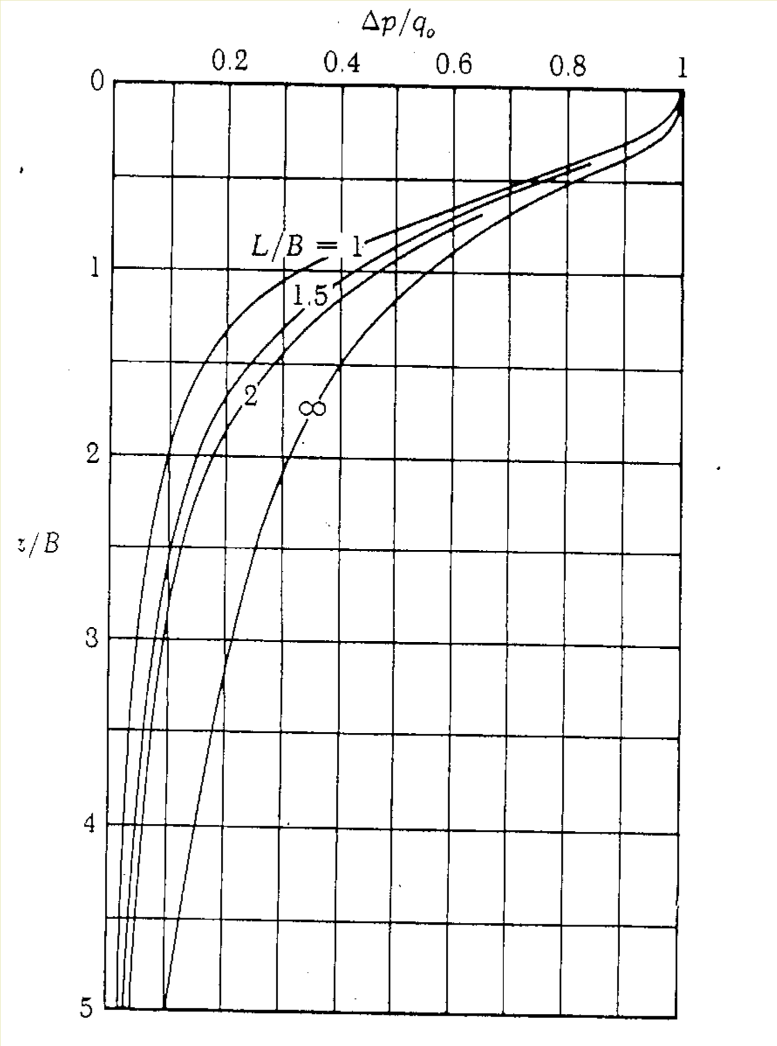


Figure 3.41 Increase of stress under the center of a flexible loaded rectangular area

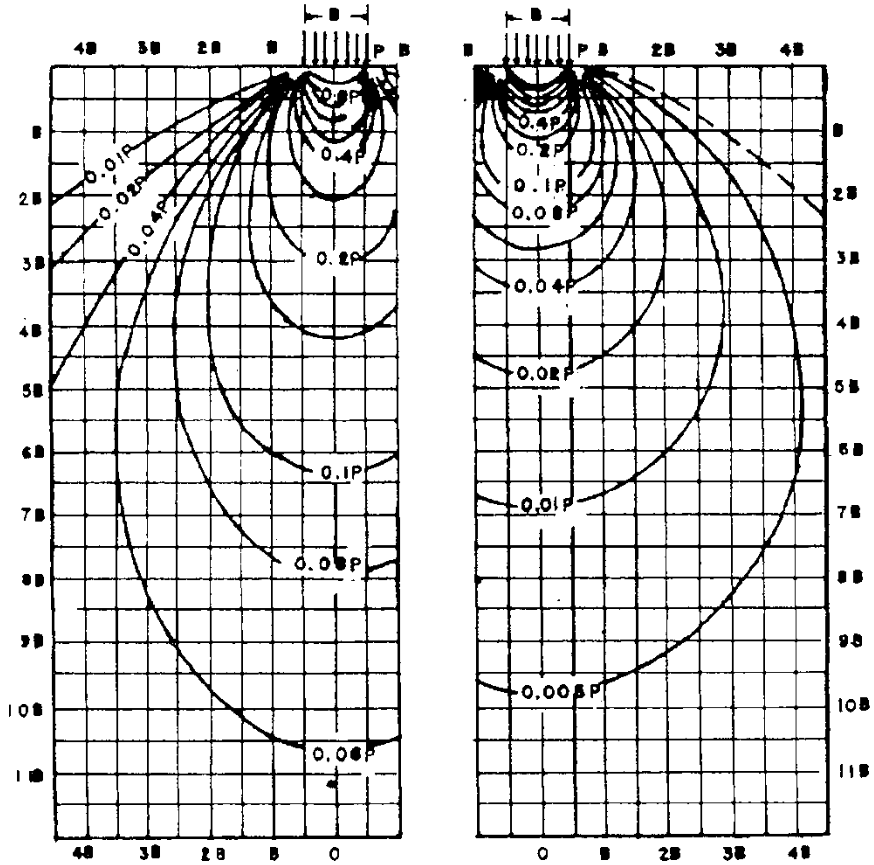
Das "Principle of Foundation Engineering", 3rd Edition

Vertical Stress Increase in the Soil Due to a Foundation Load

5. **General Charts of Stress Distribution Beneath Rectangular and Strip Footings (cont'd.)**

   (b) Stress Contours (laterally and vertically) of a strip and square footings. Soil Mechanics, DM 7.1 – p. 167

Navy Design Manual



a. INFINITELY LONG FOOTING

b. SQUARE FOOTING

SQUARE FOOTING

GIVEN

FOOTING SIZE = 20' X 20'

UNIT PRESSURE P = 2 TSF

FIND

PROFILE OF STRESS INCREASE BENEATH CENTER OF FOOTING DUE TO APPLIED LOAD

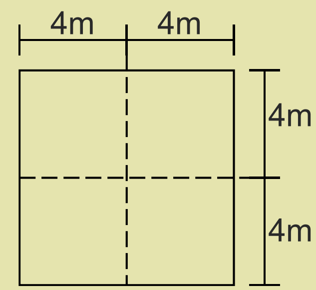
B = 20' P = 2 TSF

Z (FT)	Z/B	σ_z TSF
10	0.5	0.70 X 2 = 1.4
20	1	0.38 X 2 = 0.76
30	1.5	0.19 X 2 = 0.38
40	2.0	0.12 X 2 = 0.24
50	2.5	0.07 X 2 = 0.14
60	3.0	0.05 X 2 = 0.10

FIGURE 3
Stress Contours and Their Application

Vertical Stress Increase in the Soil Due to a Foundation Load

Example: size 8 x 8m, depth z = 4m



Find the additional stress under the center of the footing loaded with q_0

Table 5.2, I = 0.17522

1. Generic relationship 4 x 4 x 4 m = 1, n = 1 } Table 5.2, I = 0.17522

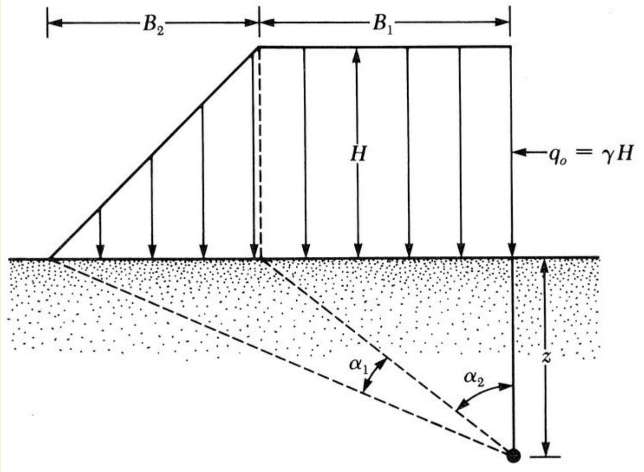
   $\Delta p = (4 \times 0.17522)q_o = 0.7q_o$

2. Specific to center, m1 = 1, n1 = 1 → Table 5.3, Ic = 0.701
3. Use Figure 3 of the Navy → Square Footing z = B/2, $\sigma z \approx 0.7p$
4. Use figure 3.41 (class notes p.12) L/B = 1, Z/B = 0.5 → $\Delta p / qo \approx 0.7$

Vertical Stress Increase in the Soil Due to a Foundation Load

6. Stress Under Embankment

Figure 5.10 Embankment loading (text p.236)



$\Delta p = \Delta\sigma = q_o I'$ (eq.5.23)

$I' = f\left(\frac{B_1}{z}, \frac{B_2}{z}\right)$ ➔ Figure 5.11 (p.237)

Example: $\gamma = 20\ kN/m^3$

$H = 3$ m $\rightarrow$ $q_o = \gamma H = 60$ kPa

$B_1 = 4$ m $\rightarrow$ $\frac{B_1}{z} = \frac{4}{5} = 0.80$

$B_2 = 4$ m $\rightarrow$ $\frac{B_2}{z} = \frac{4}{5} = 0.80$

$z = 5$ m

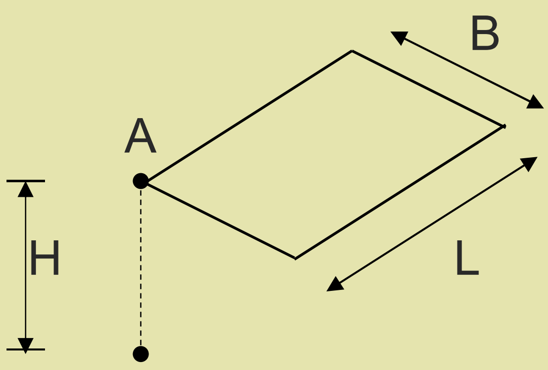
Fig. 5.11 (p.237) ➔ $I' \approx 0.43$ $\rightarrow$ $\Delta p = 0.43 \times 60 = 25.8$kPa

Vertical Stress Increase in the Soil Due to a Foundation Load

7. **Average Vertical Stress Increase due to a Rectangular Loaded Area**

Average increase of stress over a depth H under the corner of a rectangular foundation:

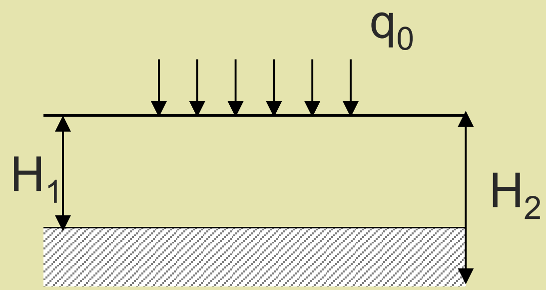
$I_a = f(m,n)$
$m = B/H$
$n = L/H$
use Figure 5.7, p. 234



Vertical Stress Increase in the Soil Due to a Foundation Load

7. **Average Vertical Stress Increase due to a Rectangularly Loaded Area (cont'd.)**

For the average depth between H_1 and H_2



Use the following:

$$\Delta p_{avg} = \Delta\sigma_{avg} = qo\,[H_2 I_{a(H2)} - H_1 I_{a(H1)}]/(H_2 - H_1)$$

(eq. 5.19, p.233 in the text)

Vertical Stress Increase in the Soil Due to a Foundation Load

7. **Average Vertical Stress Increase due to a Rectangular Loaded Area (cont'd.)**

Example: 8x8m footing

$H = 4m$ $(H_1=0, H_2=4m)$

Use 4x4x4 squares $m = 1, n = 1$

Figure 5.7 (p.234) $I_a \approx 0.225$

$\Delta p_{avg} = 4 \times 0.225 \times q_o = 0.9\ q_o$

0.9 q_o is compared to 0.7q_o (see previous example) which is the stress at depth of 4m (0.5B). The 0.9 q_o reflects the average stress between the bottom of the footing (q_o) to the depth of 0.5B.

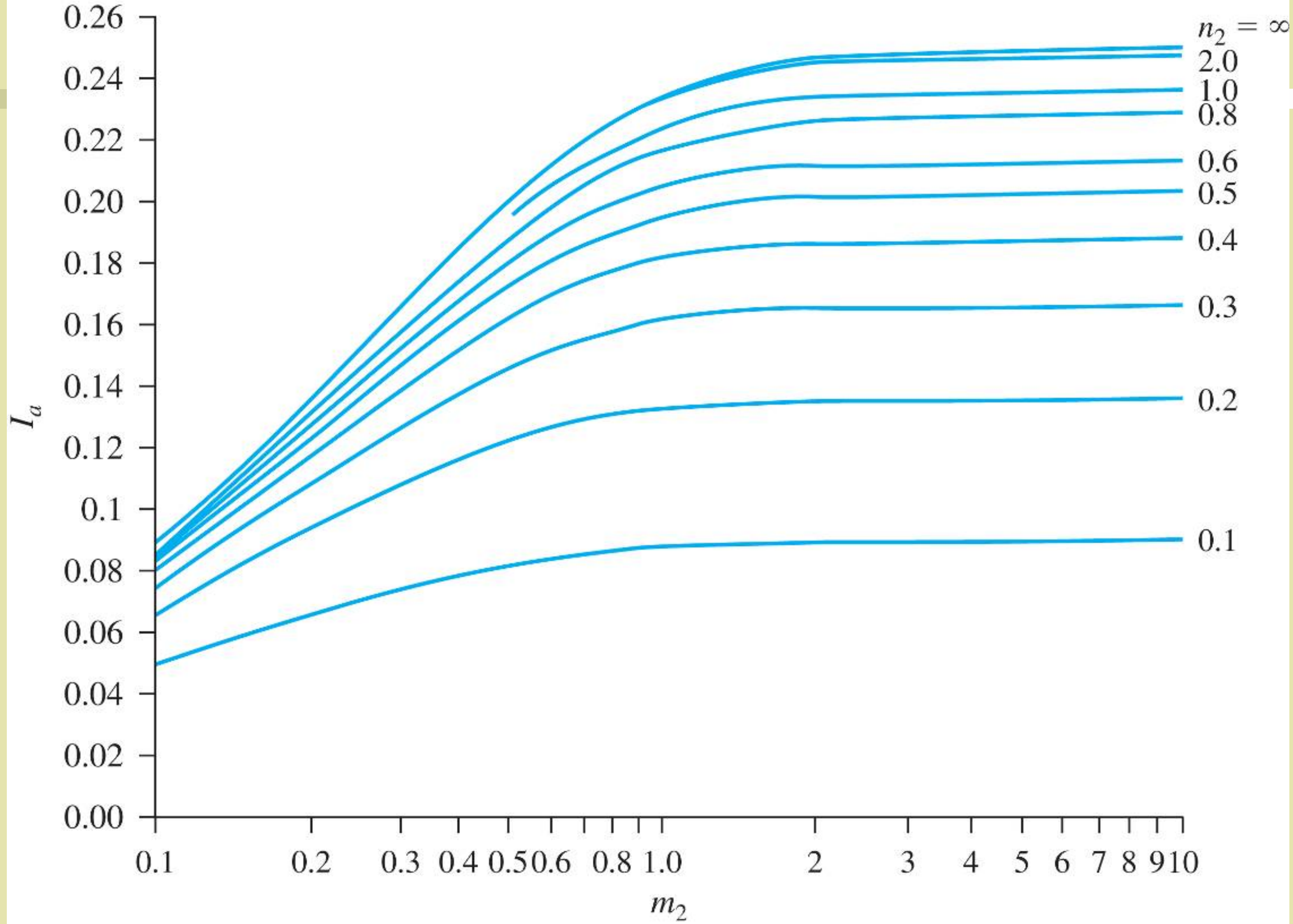


Figure 5.7 Griffiths' influence factor I_a

Vertical Stress Increase in the Soil Due to a Foundation Load

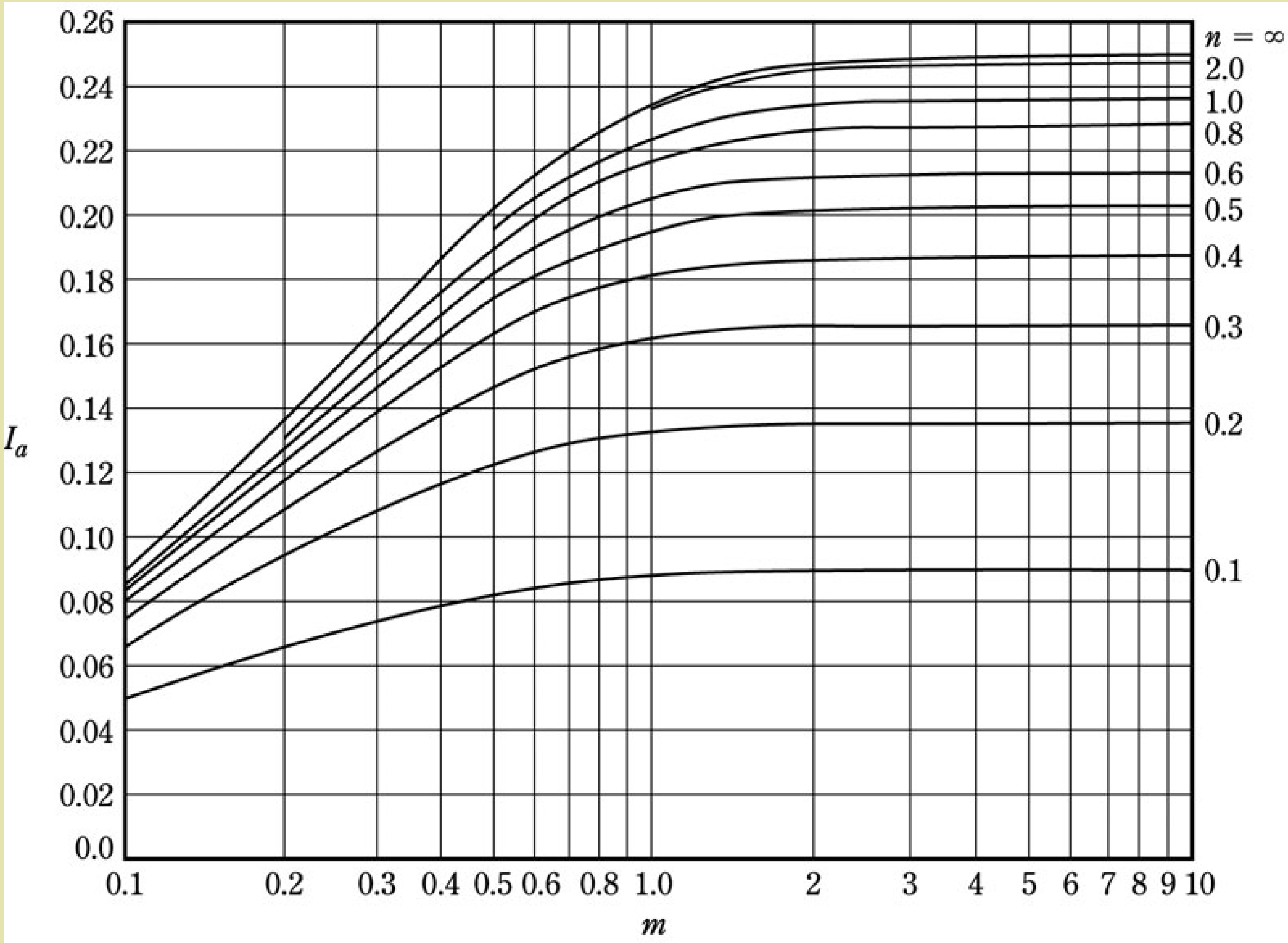
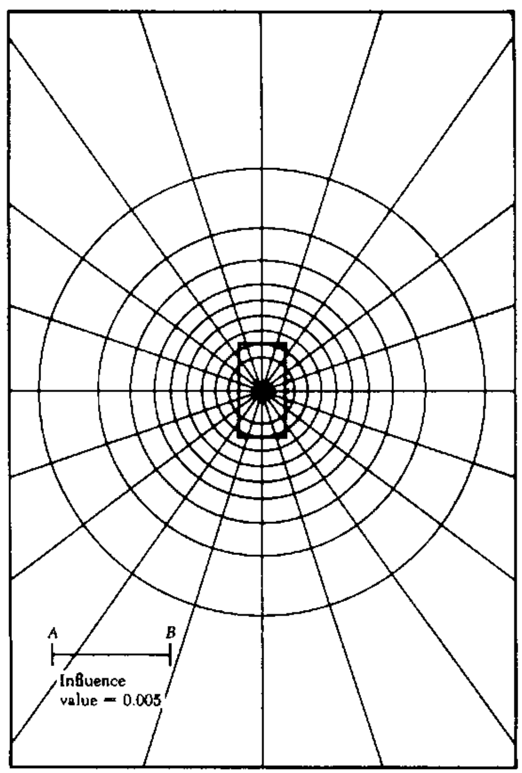


Figure 5.7 Griffiths' Influence factor I_a (text p.234)

Vertical Stress Increase in the Soil Due to a Foundation Load

8. Influence Chart – Newmark's Solution

Perform numerical integration of equation 5.1



Influence value = $\frac{1}{200}$ (# of segments)

Each segment contributes the same amount:

1. Draw the footing shape to a scale where z = length AB (2 cm = 20 mm)
2. The point under which we look for $\Delta\sigma_v'$, is placed at the center of the chart.
3. Count the units and partial units covered by the foundation
4. $\Delta\sigma_v' = \Delta p = q_o \times m \times I$

where m = # of counted units

q_o = contact stress

I = influence factor = $\frac{1}{200}$ = 0.005

Vertical Stress Incre
in the Soil Due to a Foundation Load

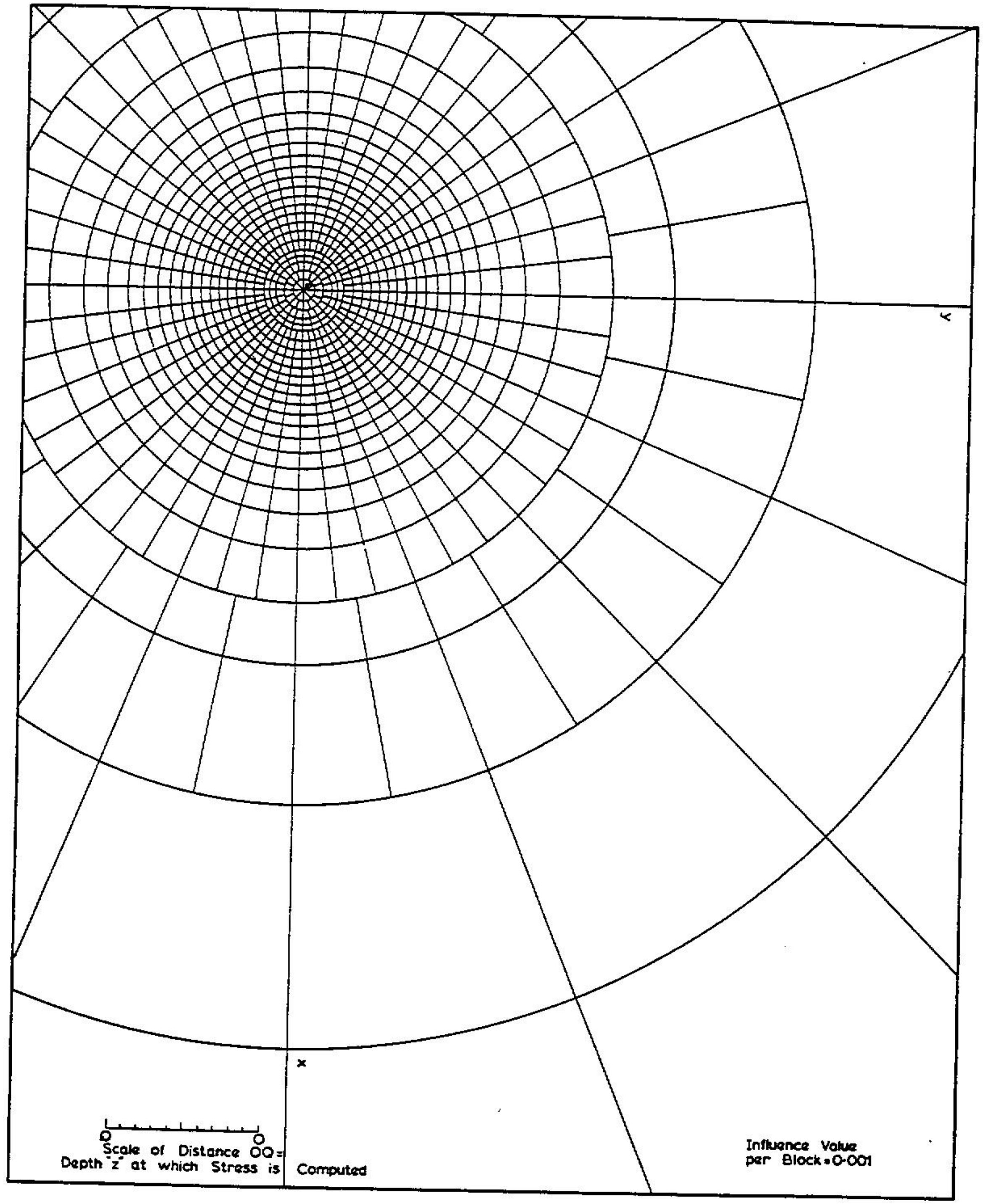
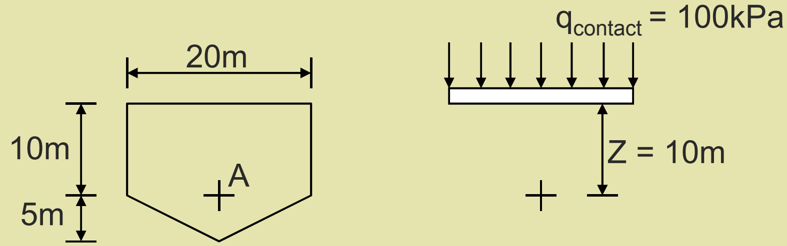


Fig. 3.50 Influence chart for vertical stress σ_z (Newmark, 1942) (All values of ν) (Poulos and Davis, 1991)
$\sigma_z = 0.001N_p$ where N = no. of blocks

Vertical Stress Increase in the Soil Due to a Foundation Load

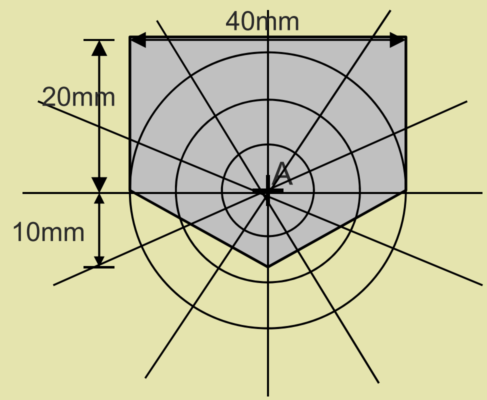
8. Influence Chart – Newmark's Solution

Example



What is the additional vertical stress at a depth of 10 m under point A ?

1. z = 10 m scale 20 mm = 10 m
2. Draw building in scale with point A at the center
   No. of elements – is (say) 76
   $\Delta\sigma_v = \Delta p = 100 \times 76 \times \frac{1}{200} = 38\text{kPa}$



Vertical Stress Increase in the Soil Due to a Foundation Load

9. Using Charts Describing Increase in Pressure

See figures from the Navy Design Manual and Das 3rd edition Fig 3.41 (notes pp. 12 & 13)
Many charts exist for different specific cases like Figure 5.11 (p.237) describing the load of an embankment (for extensive review see "Elastic Solutions for Soil and Rock Mechanics" by Poulus and Davis)

Most important to note:

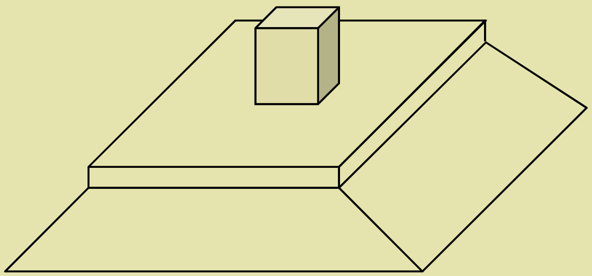
1. What and where is the chart good for?
   e.g. under center or corner of footing?
2. When dealing with lateral stresses, what are the parameters used (mostly μ) to find the lateral stress from the vertical stress

Vertical Stress Increase in the Soil Due to a Foundation Load

10. Simplified Relationship

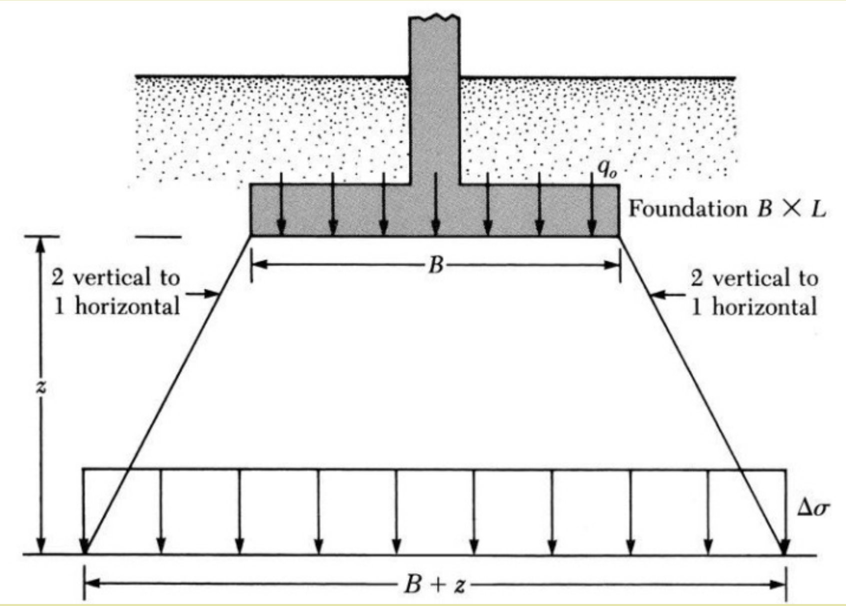
Back of an envelope calculations

2 : 1 Method (text p.231)



$$\Delta\sigma_v = \Delta P = \frac{Q}{(B+z)(L+z)}$$

Figure 5.5, (p.231)

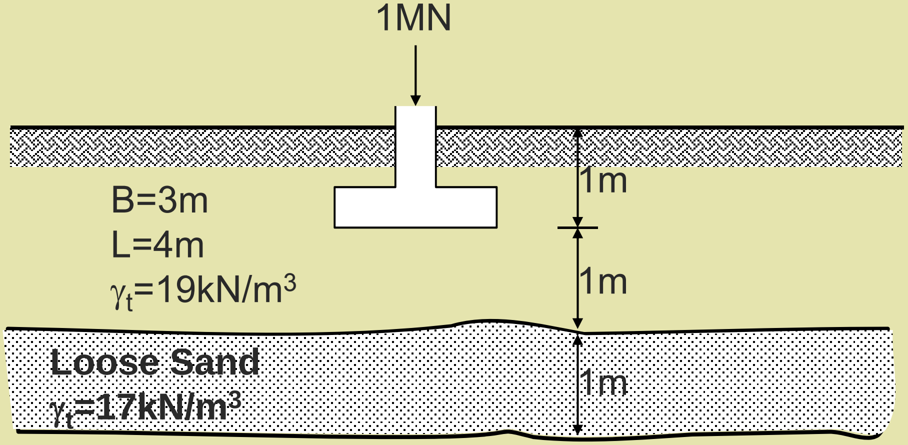


Vertical Stress Increase in the Soil Due to a Foundation Load

10. Simplified Relationship (cont'd.)

Example:

What is the existing, additional, and total stress at the center of the loose sand under the center of the foundation ?



σ_v = (2 x 19) + (0.5 x 17) = 46.5 kPa

Using 2:1 method:

$$\Delta\sigma_v = \frac{1000kN}{(3+1.5)(4+1.5)} = 40kPa \qquad q_{contact} = 83.3kPa\left(\Delta q/q_0 \cong 0.50\right)$$

Vertical Stress Increase in the Soil Due to a Foundation Load

10. Simplified Relationship (cont'd.)

Example:

Total average stress at the middle of the loose sand σ_t = 86.5 kPa

Using Fig. 3.41 of these notes (p.12):

$$\frac{z}{B} = \frac{1.5}{3} = 0.5$$

$$\frac{L}{B} = \frac{4}{3} = 1.33 \qquad \frac{\Delta p}{q_0} \approx 0.75$$

$$\Delta p = 0.75 \times 83.3 = 62.5 \text{ kPa}$$

The difference between the two values is due to the fact that the stress calculated by the 2:1 method is the average stress at the depth of 1.5m while the chart provides the stress at a point, under the center of the foundation.

Vertical Stress Increase in the Soil Due to a Foundation Load

10. Simplified Relationship (cont'd.)

Example:

This can be checked by examining the stresses under the corner of the foundation.

$$m = \frac{3}{1.5} = 2 \qquad\qquad n = \frac{4}{1.5} = 2.67$$

Table 5.2 (p.228-229) **I ≈ 0.23671 interpolated between**

	0.23614	0.23782
	n = 2.5	n = 3

$\Delta p = 0.23671 \times 83.3 = 19.71$

Checking the average stress between the center and the corner:

$$= \frac{\Delta p_{corner} + \Delta p_{center}}{2} = \frac{62.5 + 19.71}{2} = 41.1\ kPa$$

the obtained value is very close to the stress calculated by the 2:1 method that provided the average stress at the depth of 1.5m. (40kPa).

Immediate Settlement Analysis

(text Sections 5.9-5.14, pp. 243-273)

1. **General Elastic Relations**

Different equations follow the principle of the analysis presented on class notes pg. 6.
For a uniform load (flexible foundation) on a surface of a deep elastic layer, the text presents the following detailed analysis:

$$S_e = q_0 (\alpha B') \frac{1-\mu_s^2}{E_s} I_s I_f \qquad \text{(eq. 5.33)}$$

q_o	= contact stress
B'	= B′=B for settlement under the corner
	= B′=B/2 for settlement under the center
E_s, μ	= soil's modulus of elasticity and Poisson's ratio within zone of influence
α	= factor depending on the settlement location
	➢ for settlement under the center; α=4, m′=L/B, n′=H/(B/2)
	➢ for settlement under the corner; α=1, m′=L/B, n′=H/B
I_s	= shape factor, $I_s = F_1 + \frac{1-2\mu}{1-\mu} F_2$
	F_1 & F_2 f(n′ & m′) use Tables 5.8 and 5.9, pp. 248-251
I_f	= depth factor, $I_f = f\left({}^{D_f}/_B, \mu_s, {}^{L}/_B\right)$, use Table 5.10 (pp.252), I_f = 1 for D_f = 0

For a rigid footing, $S_e \approx 0.93 S_e$ (flexible footing)

Immediate Settlement Analysis

2. **Finding E_s, μ: the Modulus of Elasticity and Poisson's Ratio**

For E_s : direct evaluation from laboratory tests (triaxial) or use general values and/or empirical correlation. For general values, use Table 5.8 from Das (6^{th} ed., 2007).

Table 5.8 Elastic Parameters of Various Soils

Type of Soil	Modulus of elasticity, E_s MN/m²	Modulus of elasticity, E_s lb/in²	Poisson's ratio, μ_s
Loose sand	10.5 – 24.0	1500 – 3500	0.20 – 0.40
Medium dense sand	17.25 – 27.60	2500 – 4000	0.25 – 0.40
Dense sand	34.50 – 55.20	5000 – 8000	0.30 – 0.45
Silty sand	10.35 – 17.25	1500 – 2500	0.20 – 0.40
Sand and gravel	69.00 – 172.50	10,000 – 25,000	0.15 – 0.35
Soft clay	4.1 – 20.7	600 – 3000	
Medium clay	20.7 – 41.4	3000 – 6000	0.20 – 0.50
Stiff clay	41.4 – 96.6	6000 – 14,000	

For μ (Poisson's Ratio):
Cohesive Soils

- Saturated Clays $\Delta V = 0$, $\mu = \nu = 0.5$
- Other Soils, usually $\mu = \nu \cong 0.3$ to 0.4

Immediate Settlement Analysis

2. **Finding E_s, μ: the Modulus of Elasticity and Poisson's Ratio (cont'd.)**

Empirical Relations of Modulus of Elasticity

$\frac{E_s}{p_a} = \alpha N_{60}$ α = 5 to 15 (eq. 2.29)

(5–sands with fine s, 10–Clean N.C. sand, 15–clean O.C. sand)

Navy Design Manual (Use field values): E_s/N (E in tsf)

➢ Silts, sandy silts, slightly cohesive silt-sand mixtures	4
➢ Clean, fine to medium, sands & slightly silty sands	7
➢ Coarse sands & sands with little gravel	10
➢ Sandy gravels with gravel	12

Immediate Settlement Analysis

2. **Finding E_s, μ: the Modulus of Elasticity and Poisson's Ratio (cont'd.)**

E_s = 2 to 3.5q_c (cone resistance) CPT General Value

(Some correlation suggest 2.5 for equidimensional foundations and 3.5 for a strip foundation.)

General range for clays:

N.C. Clays Es = 250c_u to 500c_u

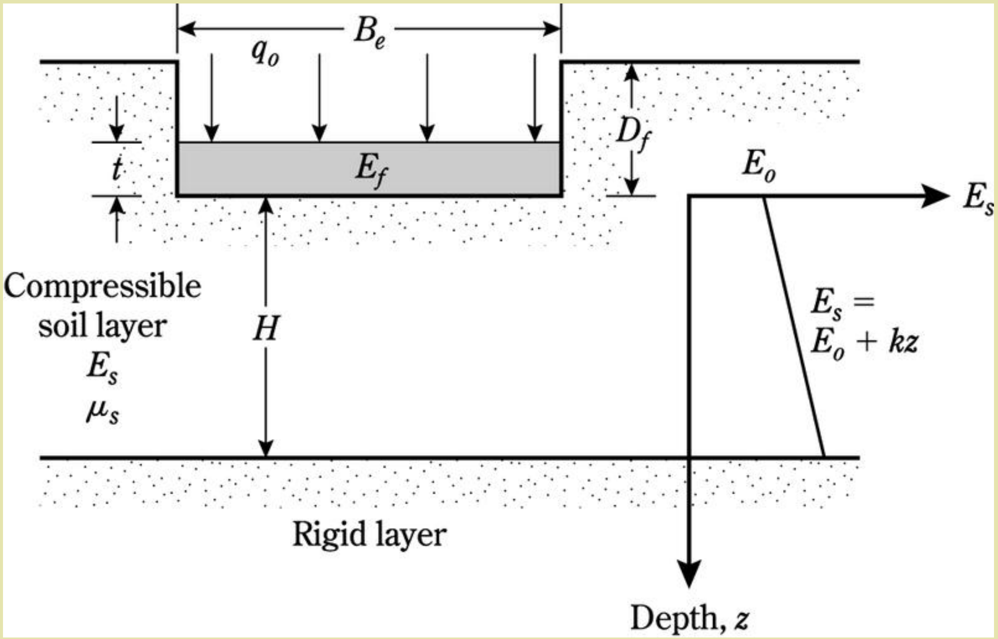
O.C. Clays Es = 750c_u to 1000c_u

See Table 5.7 for $E_s = \beta \cdot C_u$ and β = f(PI, OCR)

Immediate Settlement Analysis

3. **Improved Equation for Elastic Settlement (Mayne and Poulos, 1999)**

Considering: foundation rigidity, embedment depth, increase of E_s with depth, location of rigid layers within the zone of influence.



Immediate Settlement Analysis

3. **Improved Equation for Elastic Settlement (Mayne and Poulos, 1999) (cont'd.)**

The settlement below the center of the foundation:

$$S_e = \frac{q_0 B_e I_G I_F I_E}{E_0}\left(1 - \mu_s{}^2\right) \qquad \text{(eq. 5.46)}$$

- $B_e = \sqrt{\frac{4BL}{\pi}}$ or for a circular foundation, $B_e = B$
- $E_s = E_0 + kz$ being considered through I_G
- $I_G = f(B, H/B_e)$, $\beta = E_0/kB_e$

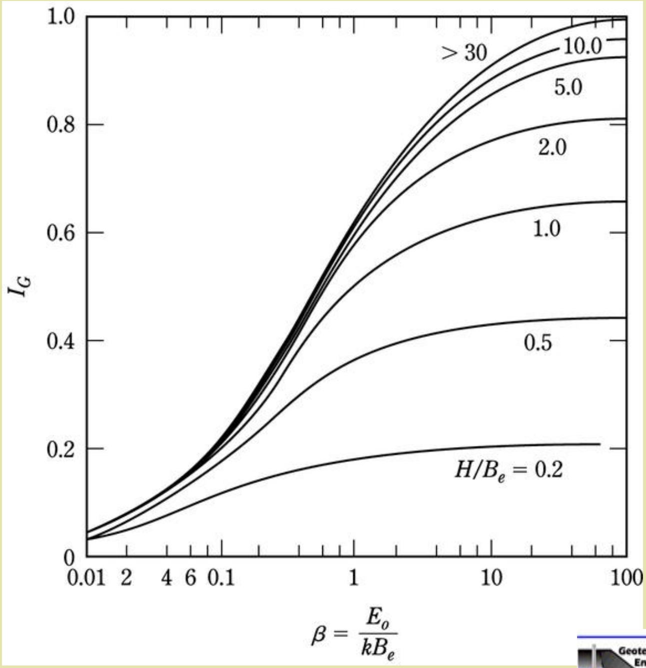


Figure 5.18 (p.255)
Variation of I_G with β

Immediate Settlement Analysis

3. Improved Equation for Elastic Settlement (Mayne and Poulos, 1999) (cont'd.)

- Effect of foundation rigidity is being considered through I_F

$$I_F = f(k_f) \text{ flexibility factor } k_F = \left(\frac{E_f}{E_0 + \frac{B_e}{2}k}\right)\left(\frac{2t}{B_e}\right)^3$$

k needs to be estimated

E_f = modulus of foundation material

t = thickness of foundation

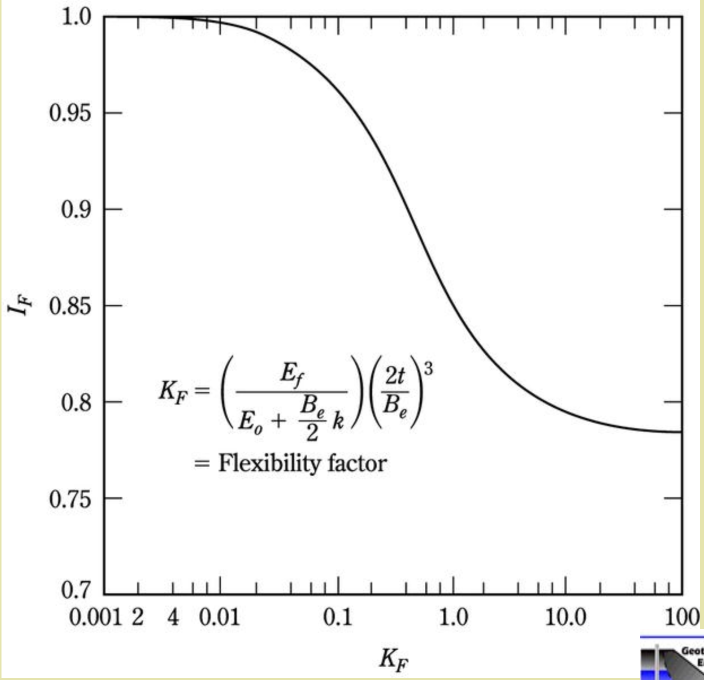


Figure 5.19 (p.256) Variation of rigidity correction factor I_F with flexibility factor k_F [Eq.(5.47)]

Immediate Settlement Analysis

3. **Improved Equation for Elastic Settlement (Mayne and Poulos, 1999) (cont'd.)**

   - Effect of embedment is being considered through I_E

     $I_E = f(\mu_s, D_f, B_e)$

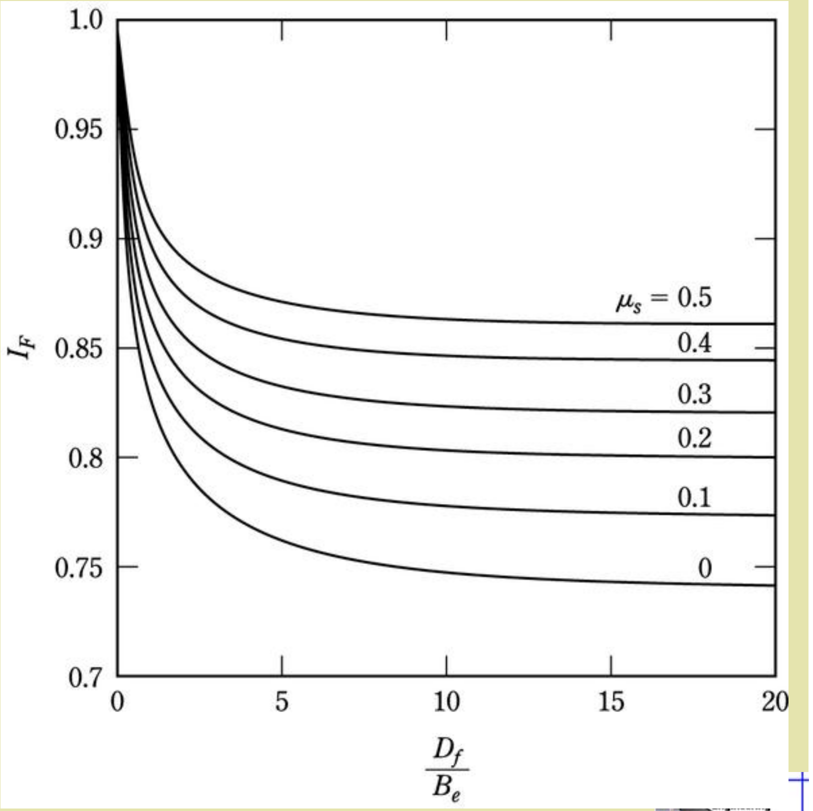


Figure 5.20 (p.256) Variation of embedment correction factor I_E with D_f/B_e [Eq.(5.48)]
Note: Figure in the text shows I_F instead of I_E.

Immediate Settlement Analysis

4. **Immediate (Elastic) Settlement of Sandy Soil – The Strain Influence Factor (Schmertmann and Hartman, 1978)**

(See Section 5.12, pp. 258-263)

The surface settlement

(i) $s_i = \int_{z=0}^{\infty} \varepsilon_z dz$

From the theory of elasticity, the distribution of vertical strain ε_z under a linear elastic half space subjected to a uniform distributed load over an area:

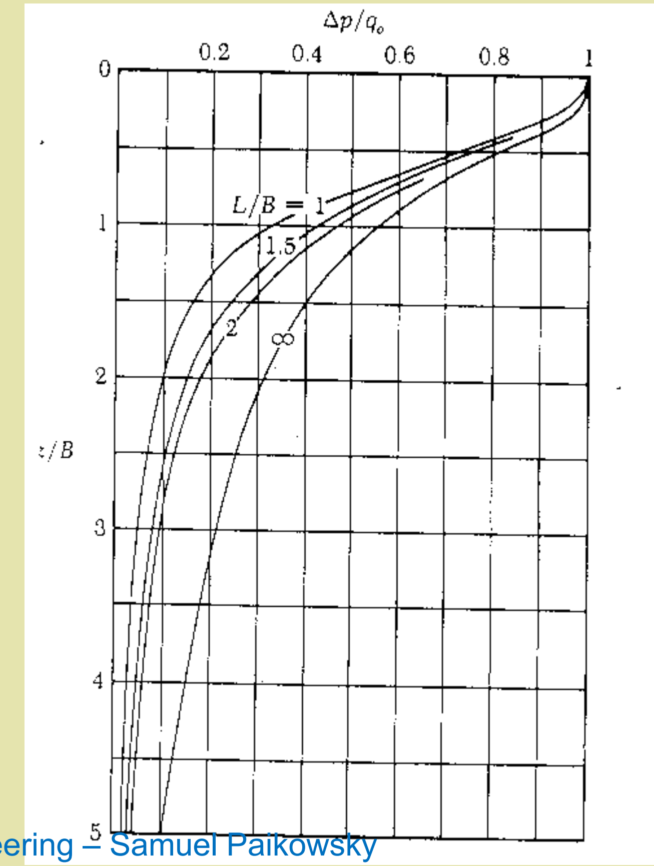
(ii) $\varepsilon_z = \frac{\Delta q}{E} I_z$

Δq = the contact load
E = modulus of elasticity - the elastic medium
I_z = strain influence factor = f (μ, point of interest)

Immediate Settlement Analysis

4. **Immediate (Elastic) Settlement of Sandy Soil – The Strain Influence Factor (Schmertmann and Hartman, 1978) (cont'd.)**

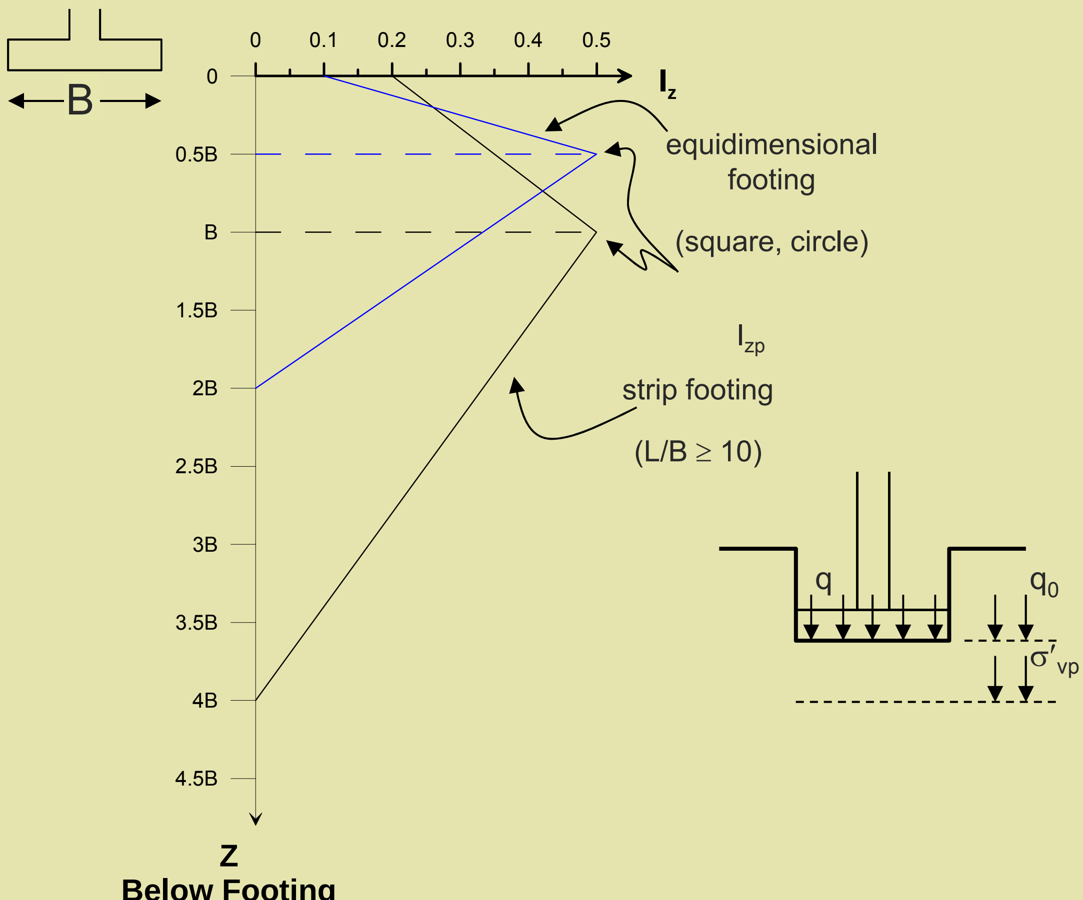
- From stress distribution (see Figure 3.41, p.12 of notes):
  influence of a square footing ≈ 2B
  influence of a strip footing ≈ 4B
  (both for $\frac{\Delta q}{q_{contact}} \approx 10\%$)



- From FEM and test results.
  The influence factor I_z:

Immediate Settlement Analysis

4. **Immediate (Elastic) Settlement of Sandy Soil – The Strain Influence Factor (Schmertmann and Hartman, 1978) (cont'd.)**



Immediate Settlement Analysis

4. **Immediate (Elastic) Settlement of Sandy Soil – The Strain Influence Factor (Schmertmann and Hartman, 1978) (cont'd.)**

substituting the above into Eq. (i).

For square $s_i = \Delta q \int_0^{2B} \frac{I_z}{E} dz$

Approximating the integral by summation and using the above simplified ε vs. D/B relations we get to equation 5.49 of the text.

$$S_e = C_1 C_2 \Delta q \sum_{i=1}^{n} \left(\frac{I_z}{E_s}\right) \Delta z_i$$

Δq = contact stress (net stress = stress at found – q_0)

$$c_1 = 1 - 0.5 \left[\frac{\sigma'_{vo}}{\Delta q}\right]$$

σ'_{vo} is calculated at the foundation depth

I_z = strain influence factor from the distribution

E_s = modulus in the middle of the layer

C_2 - (use 1.0) or C_2 = 1 + 0.2 log (10t)

Creep correction factor t = elapsed time in years, e.g. t = 5 years, C_2 = 1.34

Immediate Settlement Analysis

5. **The Preferable Iz Distribution for the Strain Influence Factor**

The distribution of I_z provided in p.28 of the notes is actually a simplified version proposed by Das (Figure 5.21, p.259 of the text). The more complete version of I_z distribution recommended by Schmertmann et al. (1978) is

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{\Delta q}{\sigma'_{vp}}}$$

Where σ'_{vp} is the effective vertical stress at the depth of I_{zp} (i.e. 0.5B and 1B below the foundation for axisymmetric and strip footings, respectively).

Immediate Settlement Analysis

6. **Immediate Settlement in Sandy Soils using Burland and Burbridge's (1985) Method**

(Section 5.13, pp.265-267)

$$\frac{S_e}{B_R} = \alpha_1 \alpha_2 \alpha_3 \left[\frac{1.25\left(L/B\right)}{0.25+\left(L/B\right)}\right]^2 \left(\frac{B}{B_R}\right)^{0.7} \left(\frac{q'}{p_a}\right) \qquad \text{(eq. 5.70)}$$

1. Determine N SPT with depth (eq. 5.67, 5.68)
2. Determine the depth of stress influence - z′ (eq. 5.69)
3. Determine α_1, α_2, α_3 for NC or OC sand (p.266)

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand

A rectangular foundation for a bridge pier is of the dimensions L=23m and B=2.6m, supported by a granular soil deposit. For simplicity it can be assumed that L/B ≈ 10 and, hence, it is a strip footing.

- Provided q_c with depth (next page)
- Loading = 178.54kPa, q = 31.39kPa (at D_f=2m)

Find the settlement of the foundation

(a-1) The Strain Influence Factor (as in the text)

$$C_1 = 1 - 0.5\frac{q}{\bar{q} - q} = 1 - 0.5\frac{31.39}{178.54 - 31.39} = 0.893$$

$$C_2 0.2 \log\left(\frac{t}{0.1}\right) \rightarrow$$

t = 5 years C_2 = 1.34

t = 10 years C_2 = 1.40

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

Using the attached Table for the calculation of Δz (see next page)

$$S_e = C_1 C_2 (\bar{q} - q) \sum \frac{I_z}{E_s} \Delta z = (0.893)(1.34)(178.54 - 31.39)(18.95 \times 10^{-5} m)$$

$S_e = 0.03336\text{m} \cong 33\text{mm}$

For t = 10 years → S_e = **34.5mm**

For the calculation of the strain in the individual layer and it's integration over the entire zone of influence, follow the influence chart (notes p.28) and the figure and calculation table below.

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

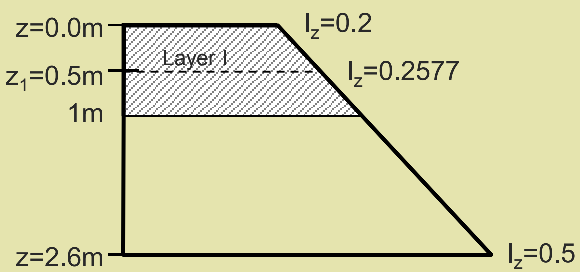
Example

$z = 0 \rightarrow I_z = 0.2$

$z = 1B = 2.6m \rightarrow I_z = 0.5$

$z_1 = 0.5m \rightarrow I_z = 0.2 + \frac{0.5-0.2}{2.6} \times 0.5 = 0.2577$

note: sublayer 1 has a thickness of 1m and we calculate the influence factor at the center of the layer.



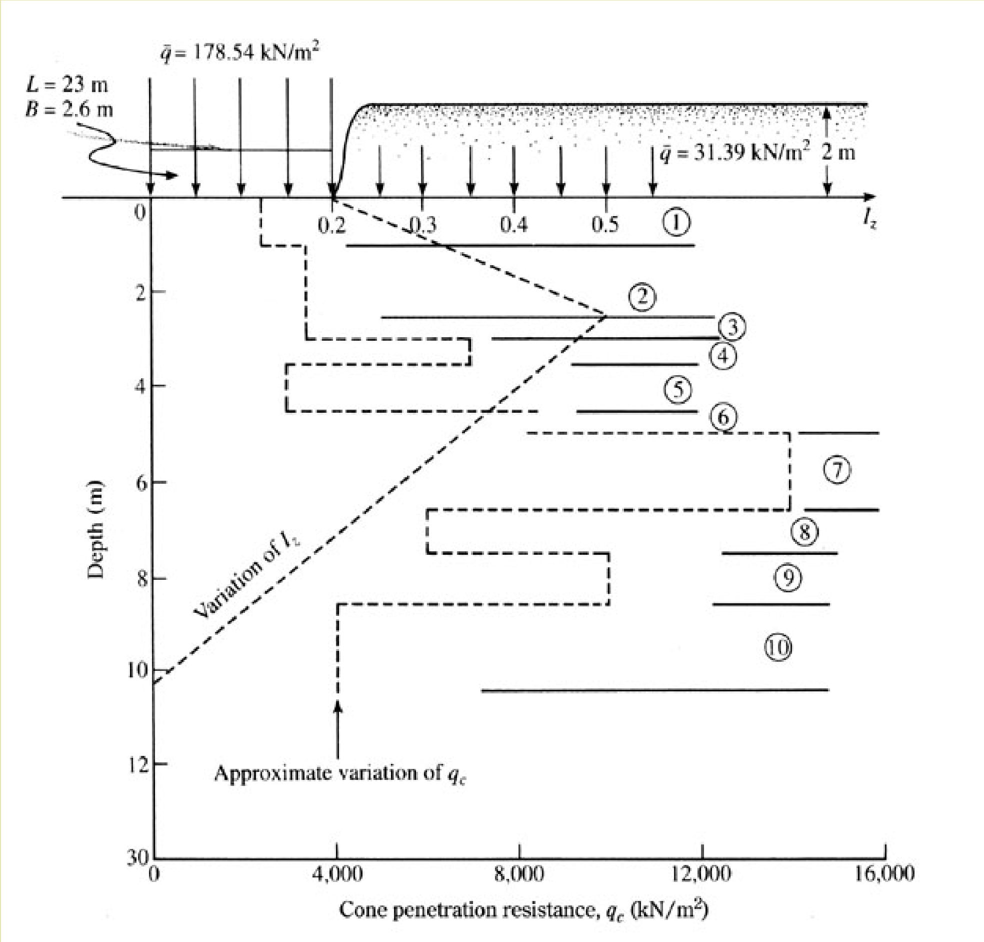
Layer	Δz (m)	q_c (kN/m²)	E_s[a] (kN/m²)	z to the center of the layer (m)	I_z at the center of the layer	$(I_z/E_s)\,\Delta z$ (m²/kN)
1	1	2,450	8,575	0.5	0.258	3.00×10^{-5}
2	1.6	3,430	12,005	1.8	0.408	5.43×10^{-5}
3	0.4	3,430	12,005	2.8	0.487	1.62×10^{-5}
4	0.5	6,870	24,045	3.25	0.458	0.95×10^{-5}
5	1.0	2,950	10,325	4.0	0.410	3.97×10^{-5}
6	0.5	8,340	29,190	4.75	0.362	0.62×10^{-5}
7	1.5	14,000	49,000	5.75	0.298	0.91×10^{-5}
8	1	6,000	21,000	7.0	0.247	1.17×10^{-5}
9	1	10,000	35,000	8.0	0.154	0.44×10^{-5}
10	1.9	4,000	14,000	9.45	0.062	0.84×10^{-5}
	Σ 10.4 m = 4B					$\Sigma\, 18.95 \times 10^{-5}$

[a] $E_s \approx 3.5 q_c$

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

Variation of I_z and q_c below the foundation



Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

Find the settlement of the foundation

(a-2) The Strain Influence Factor (Schmertmann et al., 1978))

$$I_{zp} = 0.5 + 0.1\sqrt{\frac{\Delta q}{\sigma'_{vp}}}$$

q = 31.39kPa → γ_t = 15.70kN/m^3

Δq = 178.54 – 31.39 = 147.15

σ'_{vp} @ 1B below the foundation = 31.39 + 2.6 (15.70) = 72.20kPa

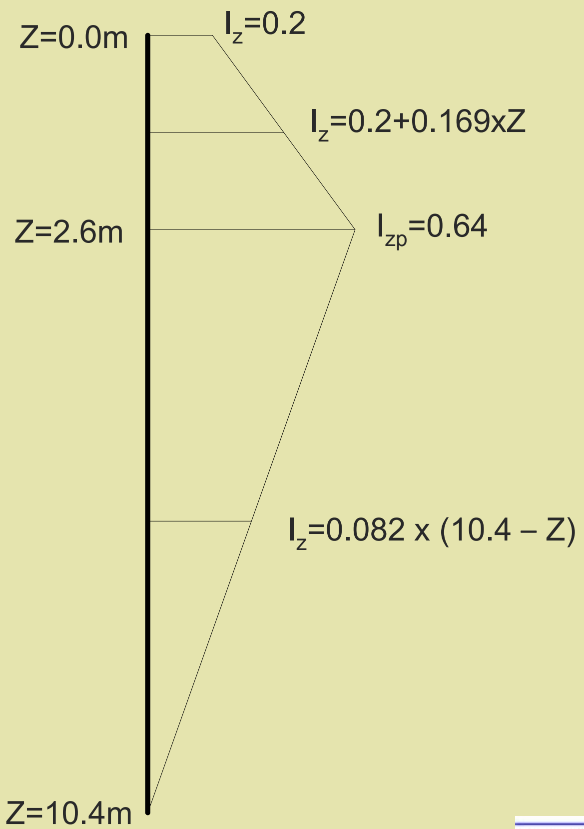
$$I_{zp} = 0.5 + 0.1\sqrt{\frac{147.15}{72.2}} = 0.50 + 0.14 = 0.64$$

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

This change will affect the table on p. 28 in the following way:

Layer	I_z	$(I_z/E_z)\Delta z$ $[(m^2/kN)x10^{-5}]$
1	0.285	3.31
2	0.505	6.72
3	0.624	2.08
4	0.587	1.22
5	0.525	5.08
6	0.464	0.79
7	0.382	1.17
8	0.279	1.32
9	0.197	0.56
10	0.078	1.06
		Σ 23.31x10^{-5}



$$S_e = C_1 C_2(\bar{q} - q)\sum \frac{I_z}{E_s}\Delta z$$

Using the I_{zp} S_e = 40.6mm

for t = 10 years, S_e = 42.4mm

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

Using the previously presented elastic solutions for comparison:

(b) The elastic settlement analysis presented in section 5.10

$$S_e = q_0(\alpha B')\frac{{\mu_s}^2}{E_s} I_s I_f \qquad \text{(eq. 5.33)}$$

B′ = 2.6/2 = 1.3m for center
B = 2.6m for corner
q_0 = 178.54kPa (stress applied to the foundation)

Strip footing, zone of influence ≈ 4B = 10.4m
From the problem figure ➔ $q_c \approx$ 4000kPa. Note the upper area is most important and the high resistance zone between depths 5 to 6.3m is deeper than 2B, so choosing 4,000kPa is on the safe side. Can also use weighted average (equation 5.34)

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

(b) The elastic settlement analysis presented in section 5.10 (cont'd.)

q_c ≈ 4,000kPa, general, use notes p.24-25:
E_s = $2.5q_c$ = 104,000kPa, matching the recommendation for a square footing
μ_s ≈ 0.3 (the material dense)

For settlement under the center:

α=4, m′=L/B=23/2.6 = 8.85, n′=H/(B/2)= (>30m)/(2.6/2) > 23

Table 5.8	m′ = 9	n′ = 12	F_1 = 0.828	F_2 = 0.095
	m′ = 9	n′ = 100	F_1 = 1.182	F_2 = 0.014

the difference between the values of m′=8 or m′=9 is negligible so using m′=9 is ok. For n′ one can interpolate. For accurate values one can follow equations 5.34 to 5.39.

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

(b) The elastic settlement analysis presented in section 5.10 (cont'd.)

interpolated values for n′=23 ➔ F_1=0.872, F_2=0.085

for exact calculations:

$$I_s = F_1 + \frac{1-2\mu_s}{1-\mu_s}F_2 = 0.872 + \frac{1-2(0.3)}{1-0.3}(0.085) \cong 0.921$$

As the sand layer extends deep below the footing H/B >> and F_2 is quite negligible.

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

(b) The elastic settlement analysis presented in section 5.10 (cont'd.)

For settlement under corner:

α=1, m′=L/B= 8.85,   n′=H/(B)= (>30m)/2.6 > 11.5

Tables 5.8 & 5.9

m′ = 9   n′ = 12 F_1 = 0.828 F_2 = 0.095

$$I_s = 0.828 + \frac{1-2(0.3)}{1-0.3}(0.095) \cong 0.882$$

D_f/B = 2/2.6 = 0.70,   L/B = 23/2.6 = 8.85

Table 5.10 ➔ μ_s = 0.3,   B/L = 0.2, D_f/B = 0.6 ➔ I_f = 0.85,

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

(b) The elastic settlement analysis presented in section 5.10 (cont'd.)

- Settlement under the center (B′ = B/2, α = 4)

$$S_e = 178.54(4)(1.15)\frac{1-(0.3)^2}{10,000}(0.921)(0.85) = 0.0585m = 58mm$$

- Settlement under the corner (B′ = B, α = 1)

$$S_e = 178.54(1)(2.3)\frac{1-(0.3)^2}{10,000}(0.882)(0.85) = 0.0280m = 28mm$$

Average Settlement = **43mm**

Using eq. 5.41 settlement for flexible footing = (0.93)(43) = **40mm**

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

(c) The elastic settlement analysis presented in section 5.11

$$S_e = \frac{q_0 B_e I_G I_F I_E}{E_0}\left(1 - \mu_s{}^2\right) \quad \text{(eq. 5.46)}$$

$$B_e = \sqrt{\frac{4BL}{\pi}} = \sqrt{\frac{4(2.6)(23)}{\pi}} = 8.73m$$

$$\beta = \frac{E_0}{kB_e}$$

Using the given figure of q_c with depth, an approximation of q_c with depth can be made such that $q_c = q_0 + z(q/z)$ where $q_0 \approx 2200\text{kPa}$, $q/z \approx 6000/8 = 750\text{kPa/m}$

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

(c) The elastic settlement analysis presented in section 5.11 (cont'd.)

Using the ratio of $E_s/q_c = 2.5$ used before, this relationship translates to $E_0 = 5500$kPa and $k = E/z = 1875$kPa/m

$$\beta = \frac{5500}{(1875)(8.73)} 0.336$$

H/B_e = >10/8.73 > 1.15 no indication for a rigid layer and actually a less dense layer starts at ≈9m ($q_c \approx 4000$kPa)

Figure 5.18, $\beta \approx 0.34 \rightarrow$ $I_G \approx 0.35$ (note; H/B_e has almost no effect in that zone when greater than 1.0)

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

(c) The elastic settlement analysis presented in section 5.11 (cont'd.)

$$k_F = \frac{E_f}{E_0 + \frac{B_e}{2}} \left(\frac{zt}{B_e}\right)^3$$

Using $E_f = 15\text{x}10^6$kPa, t = 0.5m

$$k_F = \frac{15 \times 10^6}{5500 + \frac{8.73}{2} 1875} \left(\frac{2 \times 0.5}{8.73}\right)^3 = 1.65$$

$$I_F = \frac{\pi}{4} + \frac{1}{4.6 \times 10 k_F} = \frac{\pi}{4} + \frac{1}{4.6 \times 10 \times 1.65} = 0.80$$

$$I_E = 1 - \frac{1}{3.5 e^{(1.22\mu_s - 0.4)} \left({B_e}/{D_f} + 1.6\right)} = 1 - \frac{1}{3.5 e^{(1.22\mu_s - 0.4)} \left({8.73}/{2} + 1.6\right)} = 1 - \frac{1}{20.18} = 0.95$$

$$S_e = \frac{178.54 \times 8.73 \times 0.35 \times 0.80 \times 0.95}{5500}(1 - 0.3^2) = 0.0686m$$ = **69mm**

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

   (d) Burland and Burbridge's Method presented in Section 5.13, p.265

   1. Using $q_c \approx 4{,}000\text{kPa} = 41.8\text{tsf}$ and as $E_s \cong 7N$ and $E_s \cong 2q_c$ we can also say that: $N \approx q_c(\text{tsf})/3.5$
      $\therefore\ N \approx 12$

   2. The variation of q_c with depth suggests increase of q_c to a depth of ~6.5m (2.5B) and then decrease. We can assume that equation 5.69 is valid as the distance to the "soft" layer (z″) is beyond 2B.

$$\frac{z'}{B_R} = 1.4\left(\frac{B}{B_R}\right)^{0.75}$$

$B_R = 0.3\text{m}$

$B = 2.6\text{m}$

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

(d) Burland and Burbridge's Method presented in Section 5.13, p.265 (cont'd.)

3. Elastic Settlement (eq. 5.70)

$$S_e = B_R \alpha_1 \alpha_2 \alpha_3 \left[\frac{1.25\frac{L}{B}}{0.25+\frac{L}{B}}\right]^2 \left(\frac{B}{B_R}\right)^{0.7} \left(\frac{q'}{p_a}\right)$$

Assuming N.C. Sand:

$\alpha_1 = 0.14$, $\alpha_2 = \frac{1.71}{(12)^{1.4}} = 0.049$, $\alpha_3 = 1$

$$S_e = 0.3(0.14)(0.049)(1)\left[\frac{1.25\frac{23}{2.6}}{0.25+\frac{23}{2.6}}\right]^2 \left(\frac{2.6}{0.3}\right)^{0.7} \left(\frac{178.54}{100}\right)$$

$$S_e = 0.00206\left[\frac{11.06}{9.1}\right]^2 (8.67)^{0.7}(1.7854) = 0.025m = 25mm$$

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

(e) Summary and Conclusions

Method	Case	Settlement (mm)
Strain Influence Section 5.12, 5 years	I_z (Das)	33
	I_{zp} (Schmertmann et al.)	41
Elastic Section 5.10	Center	58
	Corner	28
	Average	40
Elastic Section 5.11		69
B & B Section 5.13		60

- The elastic solution (section 5.10) and the improved elastic equation (section 5.11) resulted with a similar settlement analysis under the center of the footing (58 and 69mm). This settlement is about twice that of the strain influence factor method as presented by Das (text) and B&B (section 5.13) (33 and 25mm, respectively).
- Averaging the elastic solution method result for the center and corner and evaluating "flexible" foundation resulted with a settlement similar to the strain influence factor as proposed by Schmertmann (40 vs. 41mm). The improved method considers the foundation stiffness.

Immediate Settlement Analysis

7. **Case History – Immediate Settlement in Sand (cont'd.)**

   (e) Summary and Conclusions (cont'd.)

- The elastic solutions of sections 5.10 and 5.11 are quite complex and take into considerations many factors compared to common past elastic methods.
- The major shortcoming of all the settlement analyses is the accuracy of the soil's parameters, in particular the soil's modulus and its variation with depth. As such, many of the refined factors (e.g. for the elastic solutions of sections 5.10 and 5.11) are of limited contribution in light of the soil parameter's accuracy.

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

(e) Summary and Conclusions (cont'd.)

- What to use?

  1) From a study conducted at UML Geotechnical Engineering Research Lab, the strain influence method using I_{zp} recommended by Schmertmann provided the best results with the mean ratio of load measured to load calculated for a given settlement being about 1.28 $\pm$ 0.77 (1 S.D.) for 231 settlement measurements on 53 foundations.

  2) Check as many methods as possible, make sure to examine the simple elastic method.

  3) Check ranges of solutions based on the possible range of the parameters (e.g. E_0).

Immediate Settlement Analysis

7. Case History – Immediate Settlement in Sand (cont'd.)

(e) Summary and Conclusions (cont'd.)

For example, in choosing q_c we could examine the variation between 3,500 to 6,000 and then the variation in the relationship between q_c and E_s between 2 to 3.5. The results would be:

E_{smin} = 2 x 3,500 = 7,000kPa
E_{smax} = 3.5 x 6,000 = 21,000kPa

As S_e of equation 5.33 is directly inverse to E_s, this range will result with:

S_{emin} = 27mm, S_{emax} = 81mm (compared to 57mm)

Immediate Settlement Analysis

8. **Immediate (Elastic) Settlement of Foundations on Saturated Clays:** (Junbu et al., 1956), section 5.9, p.243

$\mu = \nu_s = 0.5$ Flexible Footings

$$S_e = A_1 A_2 \frac{q_0 B}{E_s} \qquad \text{(eq. 5.30)}$$

A_1 = Shape factor and finite layer - $A_1 = f(H/B, L/B)$
A_2 = Depth factor - $A_2 = f(D_f/B)$

Note: H/B >>> deep layer the values become asymptotic
e.g. for L = B (square) and $H/B \geq 10$ $A_1 \approx 0.9$

Immediate Settlement Analysis

8. **Immediate (Elastic) Settlement of Foundations on Saturated Clays:** (Junbu et al., 1956), section 5.9, p.243 (cont'd.)

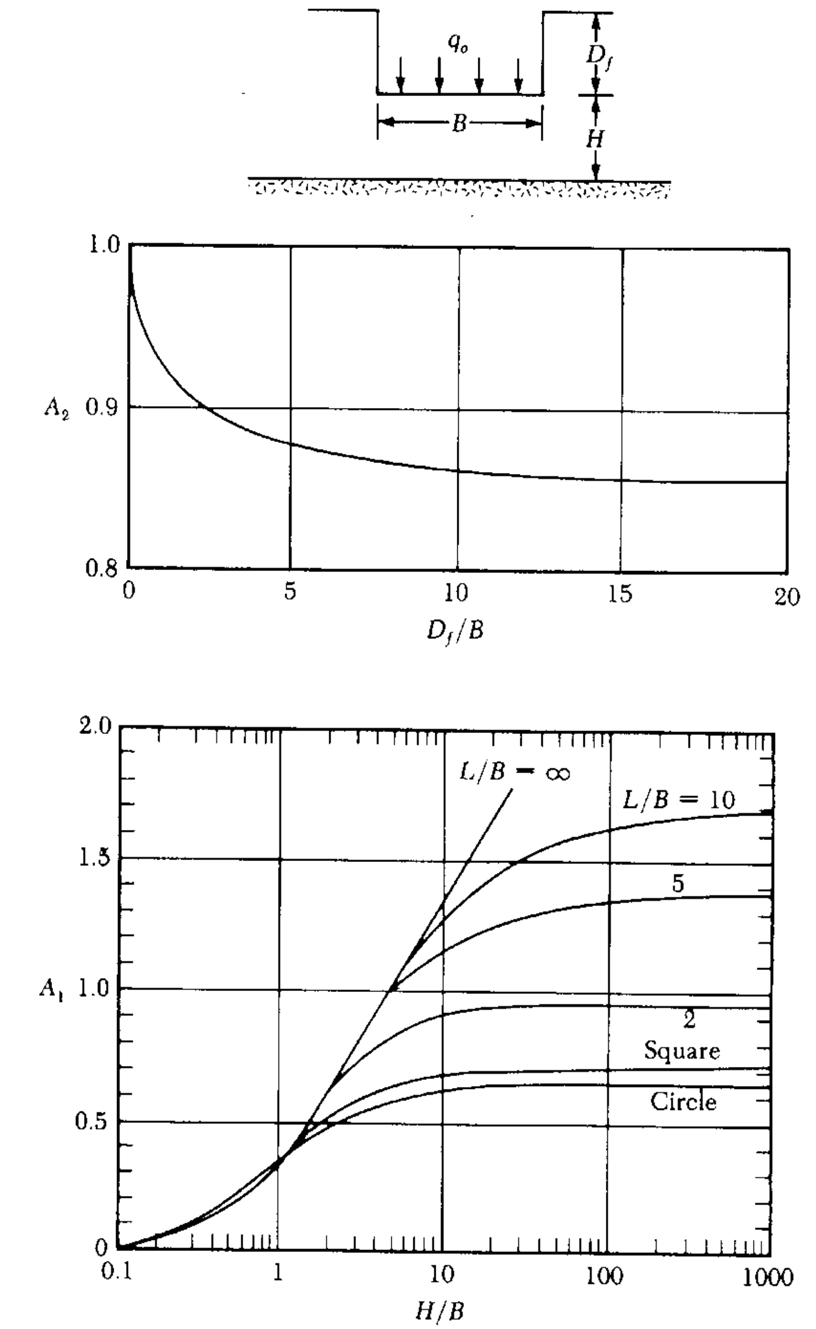


Figure 5.14 Values of A_1 and A_2 for elastic settlement calculation – Eq. (5.30) (after Christian and Carrier, 1978)

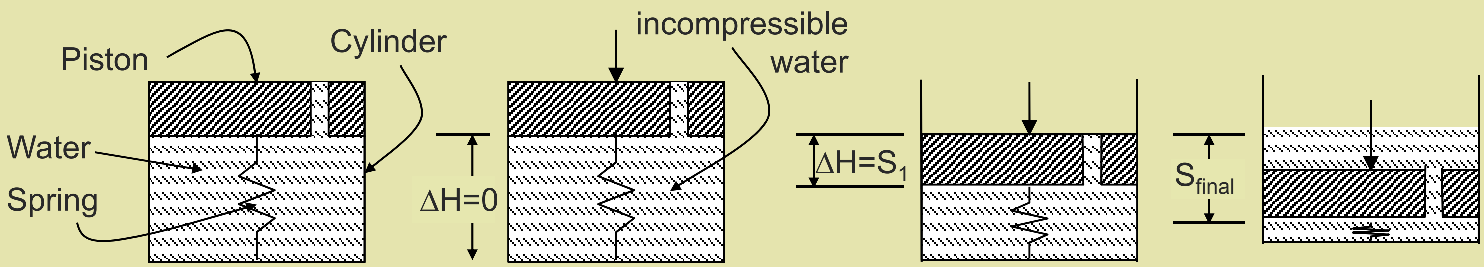
Consolidation Settlement - Long Term Settlement

Consolidation General, text Section 1.13 (pp. 32-37)

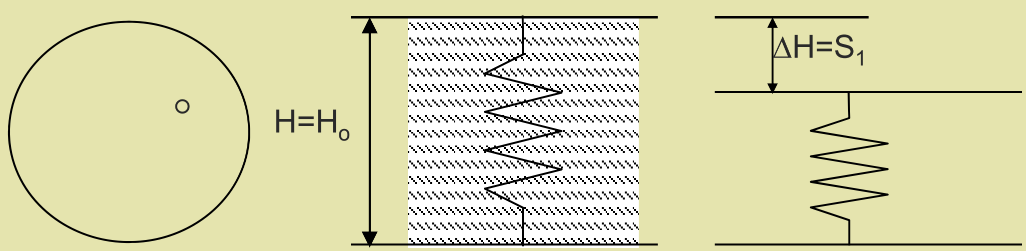
Consolidation Settlement for Foundations, text Sections 5.15 – 5.20 (pp. 273-285)

1. **Principle and Analogy**

model	$t = 0^+$	$t = t_1$	$t = \infty$



model	$t = 0^+$	$t = t_1$	$t = \infty$
$P_{spring} = 0$	$P_{spring} = 0$	$P_{spring} = \Delta H \times K_{spring}$	$P_{spring} = P$
$u = u_0 = 0$	$u_i = \frac{P}{A}$	$u = \frac{P - P_{spring}}{A}$	$u = u_0 = 0$



Consolidation Settlement - Long Term Settlement

1. **Principle and Analogy (cont'd.)**

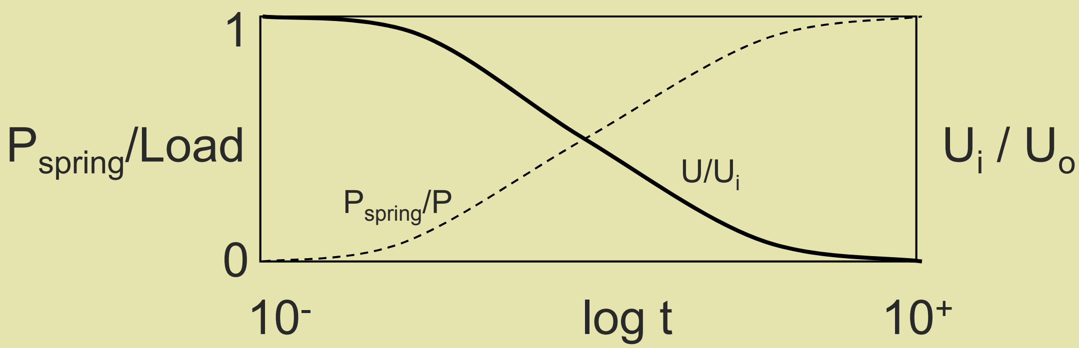
We relate only to changes, i.e. the initial condition of the stress in the soil (force in the spring) and the water are being considered as zero. The water pressure before the loading and at the final condition after the completion of the dissipation process is hydrostatic and is taken as zero, ($u_0 = u_{hydrostatic} = 0$). The force in the spring before the loading is equal to the weight of the piston (effective stresses in the soil) and is also considered as zero for the process, $P_{spring} = P_o =$ effective stress before loading= $P_{at\ rest}$. The initial condition of the process is full load in the water and zero load in the soil (spring), at the end of the process there is zero load in the water and full load in the soil.

Consolidation Settlement - Long Term Settlement

1. **Principle and Analogy (cont'd.)**

Analogy Summary

model	soil
water →	water
spring →	soil skeleton/effective stresses
piston →	foundation
hole size→	permeability
force P →	load on the foundation or at the relevant soil layer due to the foundation



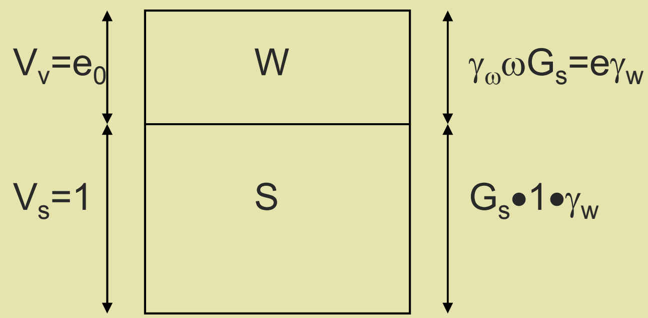
Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis

(a) Principle of Analysis

$$e = \frac{V_v}{V_s}$$

$$\omega = \frac{W_w}{W_s}$$



weight - volume relations saturated clay

initial soil volume = $V_o = 1 + e_o$

final soil volume = $V_f = 1 + e_o - \Delta e$

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(a) Principle of Analysis (cont'd.)

$\Delta V = V_o - V_f = \Delta e$

As area A = Constant: $V_o = H_o \times A$ and $V_f = H_f \times A$

$\Delta V = V_o - V_f = A(H_o - H_f) = A \times \Delta H$

$$\Delta H = \frac{\Delta V}{A}$$

for 1-D (note, we do not consider 3-D effects and assume pore pressure migration and volume change in one direction only).

$\varepsilon_v = \frac{\Delta H}{H_0} = \frac{\Delta V/A}{V_0/A} = \frac{\Delta V}{V_0}$, substituting for V, e relations $\varepsilon_v = \frac{\Delta V}{V_0} = \frac{\Delta e}{V_0} = \frac{\Delta e}{1+e_0}$

$$\boldsymbol{\Delta H = \varepsilon_v \times H_0 = \frac{\Delta e}{1+e_0} \times H_0}$$

Consolidation Settlement - Long Term Settlement

2. **Final Settlement Analysis (cont'd.)**

   (a) **Principle of Analysis (cont'd.)**

   Calculating Δe

   We need to know:

   i. Consolidation parameters c_c, c_r at a representative point(s) of the layer, based on odometer tests on undisturbed samples.

   ii. The additional stress at the same point(s) of the layer, based on elastic analysis.

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(b) Consolidation Test (1-D Test)

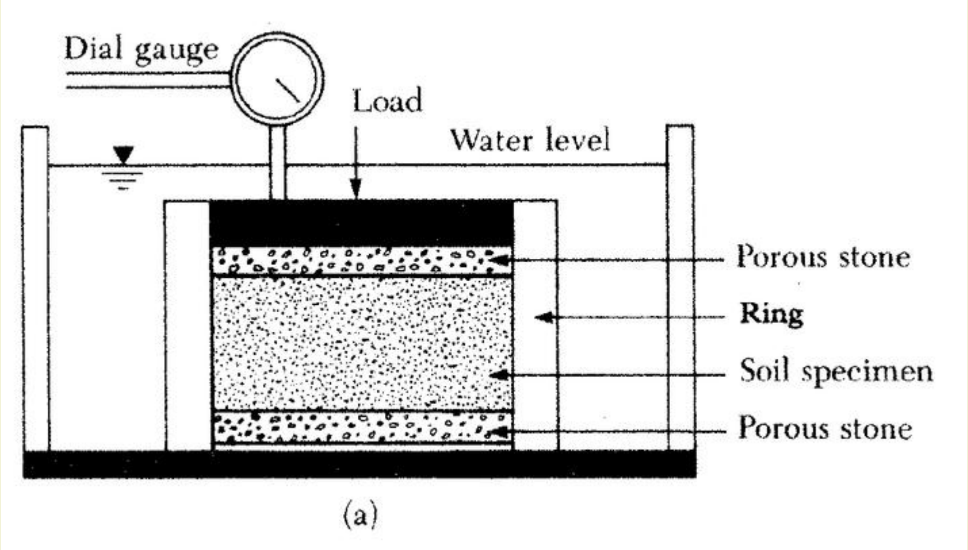


Figure 1.15a Schematic Diagram of consolidation test arrangement (p.33)

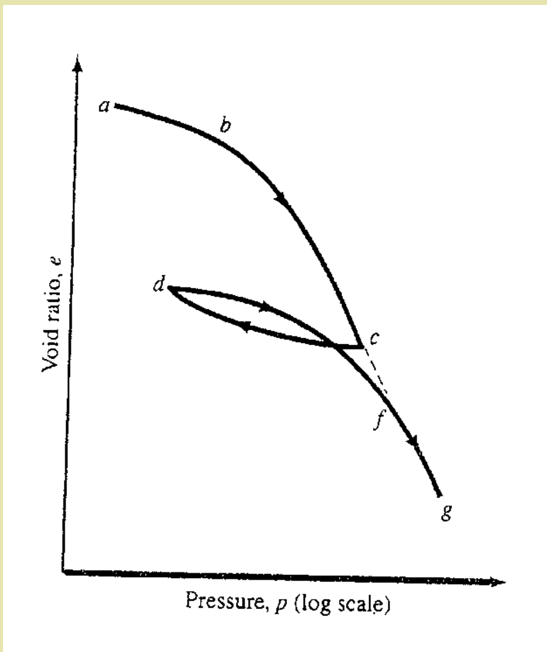
1. Oedometer = Consolidometer
2. Test Results

Consolidation Settlement - Long Term Settlement

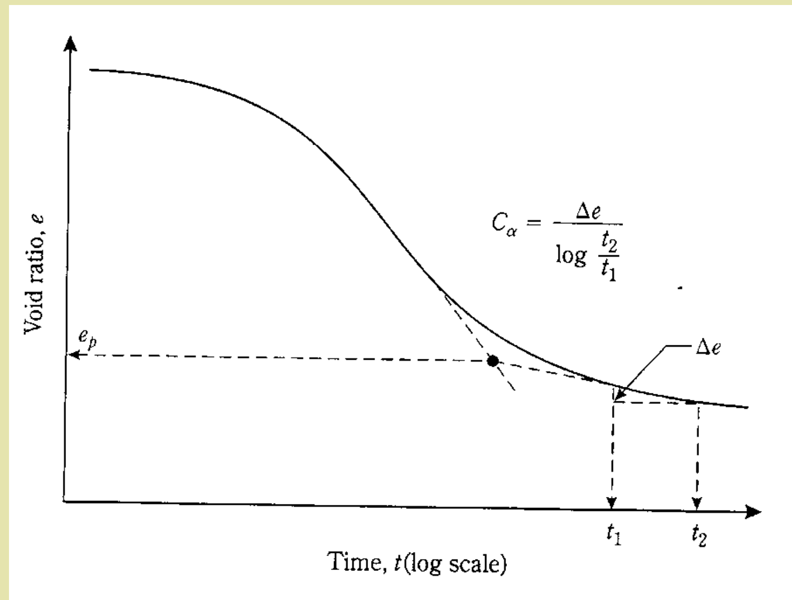
2. Final Settlement Analysis (cont'd.)

(b) Consolidation Test (1-D Test) (cont'd.)

a) final settlement with load after 24 hours



b) settlement with time under a certain load



$e = \frac{V_v}{V_s}$ e << → V_v << → denser material

γ_d >> $\gamma_d = \frac{W_s}{V}$ (V <<)

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(c) Obtaining Parameters from Test Results

analysis of e-log p results.

1st Stage - Casagrande's procedure to find max. past pressure. (see Figures 1.15 to 1.17, text pp.33 to 37, respectively)

1. find the max. curvature.
   - use a constant distance and look for the max. normal.
   - draw tangent to the curve at that point.
2. draw horizontal line through that point and divide the angle.
3. extend (if doesn't exist) the e-log p line to $e = 0.42e_o$
4. extend the tangent to the curve and find its point of intersection with the bisector of stage 2. → P_c' = max. past pressure.

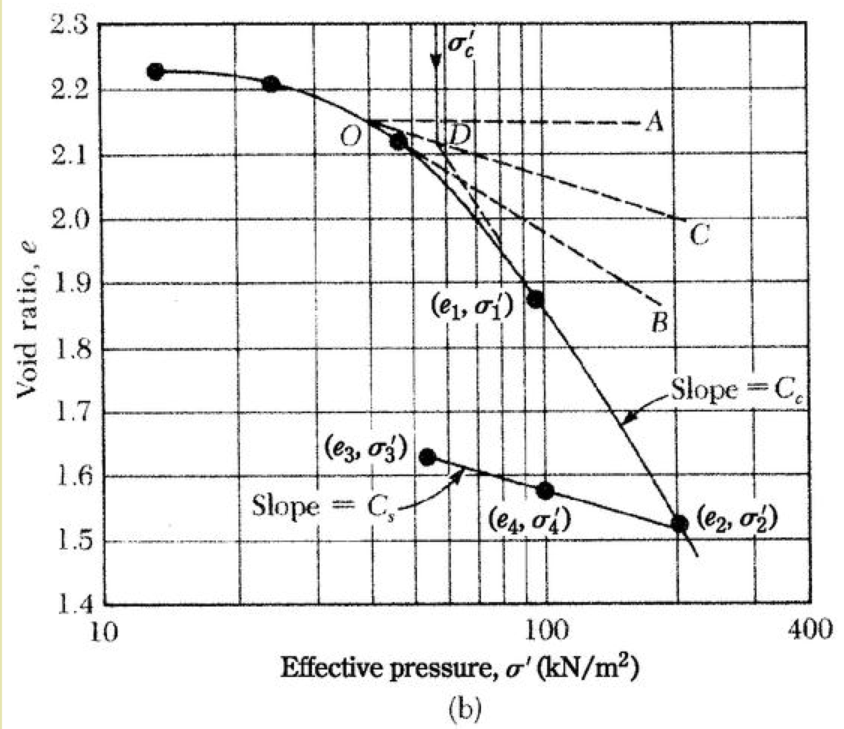


Figure 1.15 (b) e-log σ′ curve for a soft clay from East St. Louis, Illinois (note: at the end of consolidation, σ = σ′

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

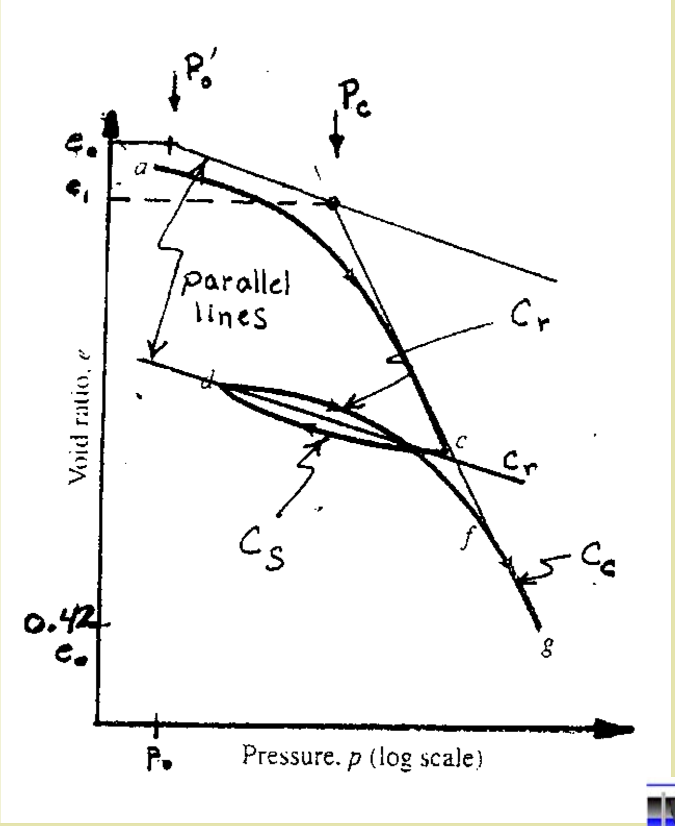
(c) Obtaining Parameters from Test Results (cont'd.)

$$OCR = \frac{p_c'}{p_0'}$$

analysis of e-log p results.

2nd Stage - Reconstructing the full e-log p′ (undisturbed) curve (Schmertmann's Method, See Figures 1.16 and 1.17, pp.35,37)

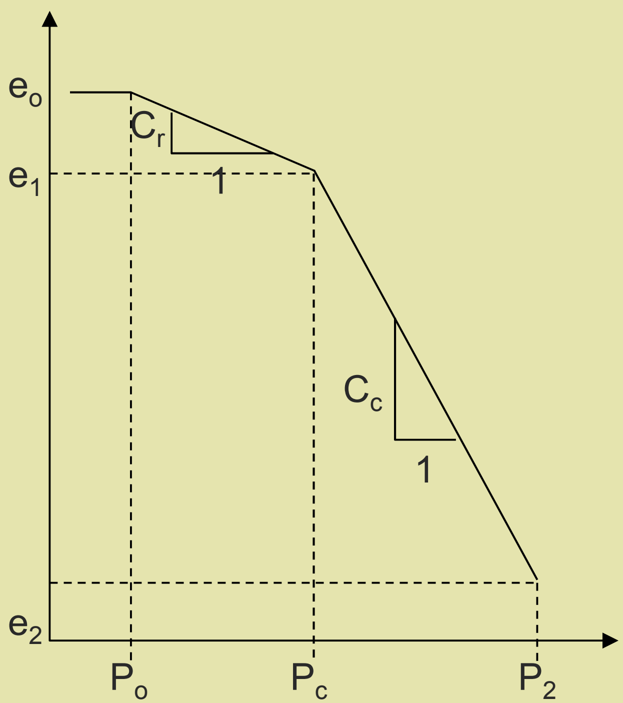
1. find the point e_o, p'_o
   $e_o = \omega_n \times G_s$ $p'_o = \gamma' z$.
2. find the avg. recompression curve and pass a parallel line through point 1.
3. find point p_c' & e
4. connect the above point to
   $e = 0.42 e_o$



Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(c) Obtaining Parameters from Test Results (cont'd.)



Compression index (or ratio)

$$C_c = \frac{\Delta e}{\log\left(p_2/p_1\right)} = \frac{e_1 - e_2}{\log\left(p_2/p_c\right)}$$

Recompression index (or ratio)

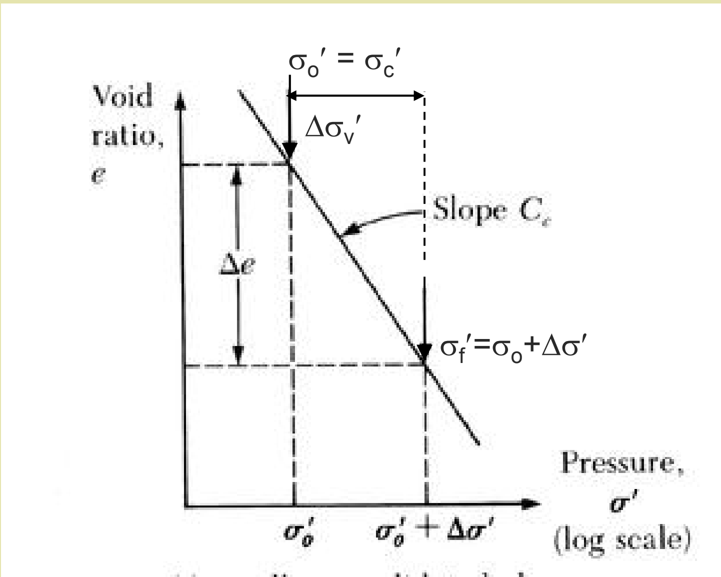
$$C_r = \frac{\Delta e}{\log\left(p_c/p_0\right)} = \frac{e_0 - e_1}{\log\left(p_c/p_0\right)}$$

- See p.35-37 of the text for C_s & C_c values.
- natural clay $C_c \approx 0.09(LL - 10)$ where LL is in (%) (eq.1.50)
- B.B.C $C_c = 0.35$ $C_s = 0.07$

Consolidation Settlement - Long Term Settlement

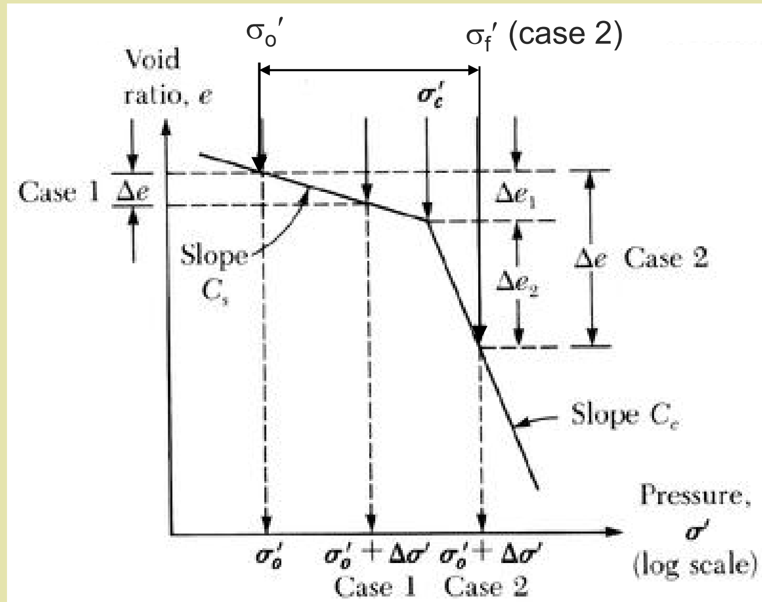
2. **Final Settlement Analysis (cont'd.)**

(d) **Final Settlement Analysis**



Normally consolidated clay

(b)



Overconsolidated clay

(c)

$$\Delta e = C_c \log \frac{\sigma'_0 + \Delta\sigma'}{\sigma'_0}$$

$$\Delta e = \underbrace{C_s \log \frac{\sigma'_c}{\sigma'_0}}_{\Delta e_1} + \underbrace{C_c \log \frac{\sigma'_0 + \Delta\sigma}{\sigma'_c}}_{\Delta e_2}$$

(for $\sigma'_0 + \Delta\sigma' > \sigma'_c$)

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(d) Final Settlement Analysis (cont'd.)

Solution:

1. Subdivide layers according to stratification and stress variation
2. In the center of each layer calculate $\sigma'_{vo}(s'_o)$ and $\Delta\sigma'$
3. Calculate for each layer Δe_i

$$H = \sum_{i=1}^{n} H_i \frac{\Delta e_i}{1 + e_0}$$

replace p_c by $\sigma_v'_{max}$ and p_o by $\sigma_v'_o$
The average increase of the pressure on a layer ($\Delta\sigma' = \Delta s'_{av}$) can be approximated using the text; eq. 5.84 (p.274)

$$\Delta\,\sigma'_{av} = \frac{1}{6}(\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b)$$

↑ top ↑ middle ↑ bottom

Consolidation Settlement - Long Term Settlement

2. **Final Settlement Analysis (cont'd.)**

   (d) Final Settlement Analysis (cont'd.)

   Skempton - Bjerrum Modification for Consolidation Settlement
   Section 5.16 p. 275 - 279

   The developed equations are based on 1-D consolidation in which the increase of pore pressure = increase of stresses due to the applied load. Practically we don't have 1-D loading in most cases and hence different horizontal and vertical stresses.

   $\Delta u = \sigma_c + A[\sigma_1 - \sigma_c]$

   A = Skempton's pore pressure parameter

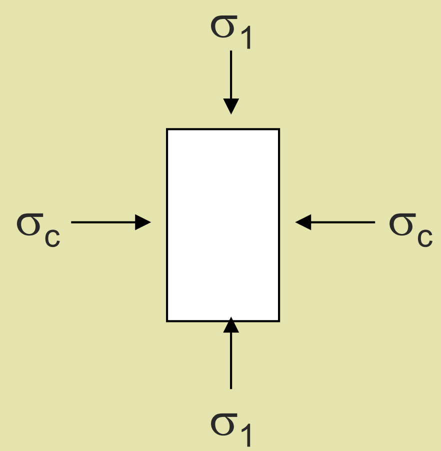
Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(d) Final Settlement Analysis (cont'd.)

For example: Triaxial Test

N.C.	OCR = 1	0.5<A<1
	OCR < 4	0.25<A<0.5
	OCR ≈ 5	0
	OCR > 6	-0.5<A<0



considering the partial pore pressure build up, we can modify our calculations:

1) calculate the consolidation settlement the same way as was shown earlier
2) determine pore water pressure parameter → lab test or see the table on p. 52 in the text
3) H_c/B = consolidation depth / foundation width
4) use Fig. 5.31, p.276, (A & H_c/B) → settlement ratio (<1) (Note circular or continuous)
5) $S_c = S_{c\ calc} \times$ Settlement Ratio

Note: Table 5.14, p.277 provides the settlement ratio as a function of B/H_c and OCR based on Leonards (1976) field work. It is an alternative to Figure 5.31 as A = f(OCR), (see above) for which an equivalent circular foundation can be calculated (e.g.)

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(d) Final Settlement Analysis (cont'd.)

From Das, Figure 5.31 and Table 5.14

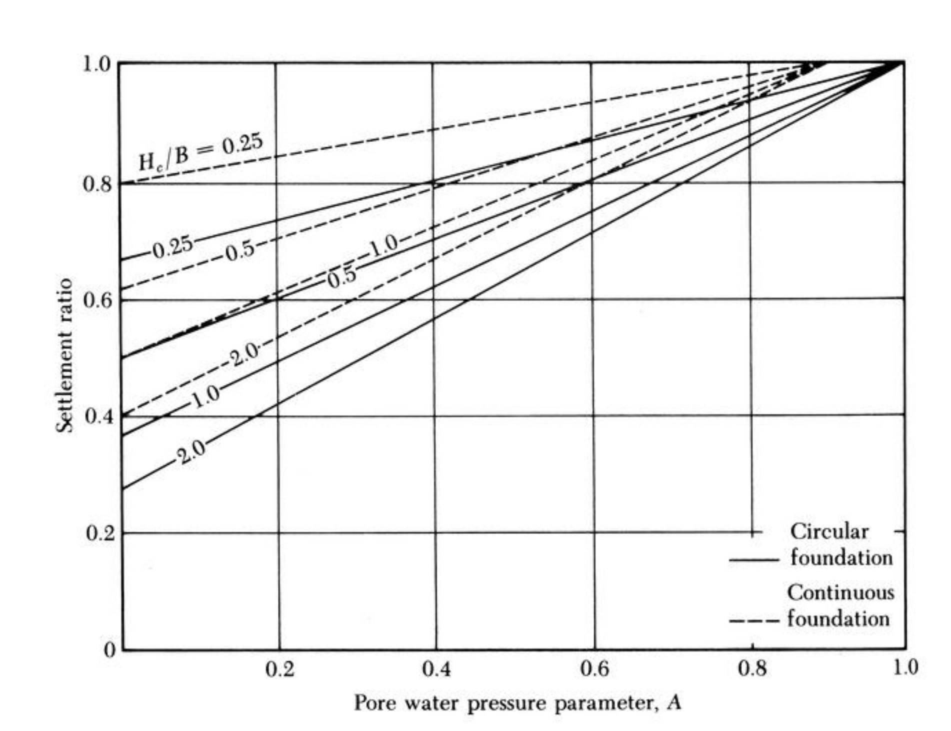


Figure 5.31 Settlement ratios for circular (K_{cir}) and continuous (K_{str}) foundations

Table 5.14 Variation of $K_{cr(OC)}$ with OCR and B/H_c

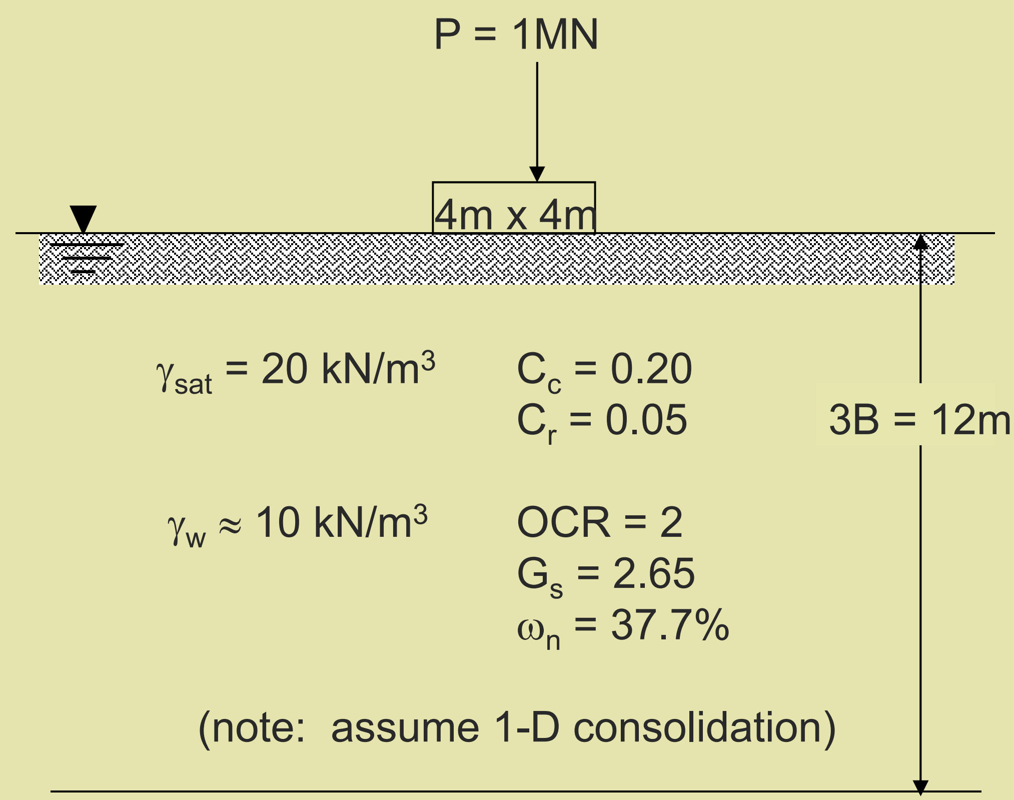
OCR	$K_{cr(OC)}$		
	$B/H_c = 4.0$	$B/H_c = 1.0$	**$B/H_c = 0.2$**
1	1	1	**1**
2	0.986	0.957	**0.929**
3	0.972	0.914	**0.842**
4	0.964	0.871	**0.771**
5	0.950	0.829	**0.707**
6	0.943	0.800	**0.643**
7	0.929	0.757	**0.586**
8	0.914	0.729	**0.529**
9	0.900	0.700	**0.493**
10	0.886	0.671	**0.457**
11	0.871	0.643	**0.429**
12	0.864	0.629	**0.414**
13	0.857	0.614	**0.400**
14	0.850	0.607	**0.386**
15	0.843	0.600	**0.371**
16	**0.843**	**0.600**	**0.357**

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(e) EXAMPLE – Final Consolidation Settlement

Calculate the final settlement of the footing shown in the figure below. Note, OCR = 2 for all depths. Give the final settlement with and without Skempton & Bjerrum Modification.



Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(e) EXAMPLE – Final Consolidation Settlement (cont'd.)

P=1MN, B=4mx4m, q_0 = 1000/16=62.5kPa

	z (m)	z/B	$\Delta q/q_o$	Δq	P_o' (kPa)	P_c' (kPa)	$P_o' + \Delta q = P_f'$	Δe	$\frac{\Delta e}{1+e_0} \times \Delta H$
Layer I	1	(0.25) +	0.90	56.3	10	20	66.3	0.1188	0.1188
----------	2	----------							
Layer II	3	(0.75) +	0.50	31.3	30	60	61.3	0.0165	0.0165
----------	4	----------							
Layer III	6	(1.50) +	0.16	10.0	60	120	70.0	0.003	0.006
----------	8	----------							
Layer IV	10	(2.5) +	0.07	4.4	100	200	104.4	0.001	0.002
----------	12	----------							

$\sum$ = 0.1433m

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(e) EXAMPLE – Final Consolidation Settlement (cont'd.)

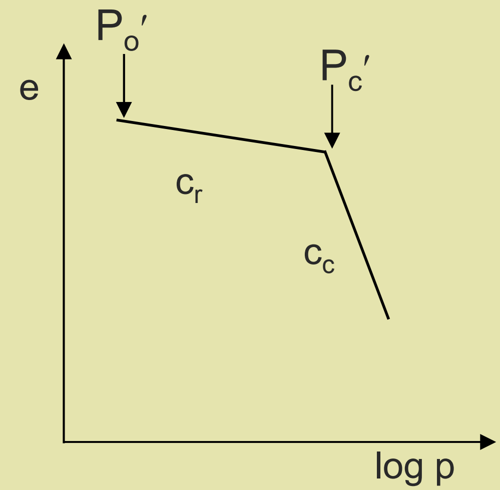
1) From Figure 3.41, Notes p. 12
   $\rightarrow$ influence depth {10% $\rightarrow$ 2B, $\cong$ 5% $\rightarrow$ 3B} = 12 m.
2) Subdivide the influence zone into 4 sublayers 2 of 2m in the upper zone (major stress concentration) and 2 of 4 m below.
3) Calculate for the center of each layer: Δ_q, P_o', P_c', P_f'
4) $e_o = \omega_n \cdot G_s = 1.0$
5) Calculate Δe for each layer:

$$\Delta e_1 = c_r \log \frac{20}{10} + C_c \log \frac{66}{20} = 0.1188$$

$$\Delta e_2 = c_r \log \frac{60}{30} + C_c \log \frac{61}{60} = 0.0165$$

$$\Delta e_3 = c_r \log \frac{70}{60} = 0.003$$

$$\Delta e_4 = c_r \log \frac{104}{100} = 0.001$$



Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(e) EXAMPLE – Final Consolidation Settlement (cont'd.)

For the evaluation of the increased stress, use general Charts of Stress distribution beneath a rectangular and strip footings

Use Figure 3.41 (p.12 of notes)

$\rightarrow \Delta P / q_0$ vs. z / B under the center of a rectangular footing

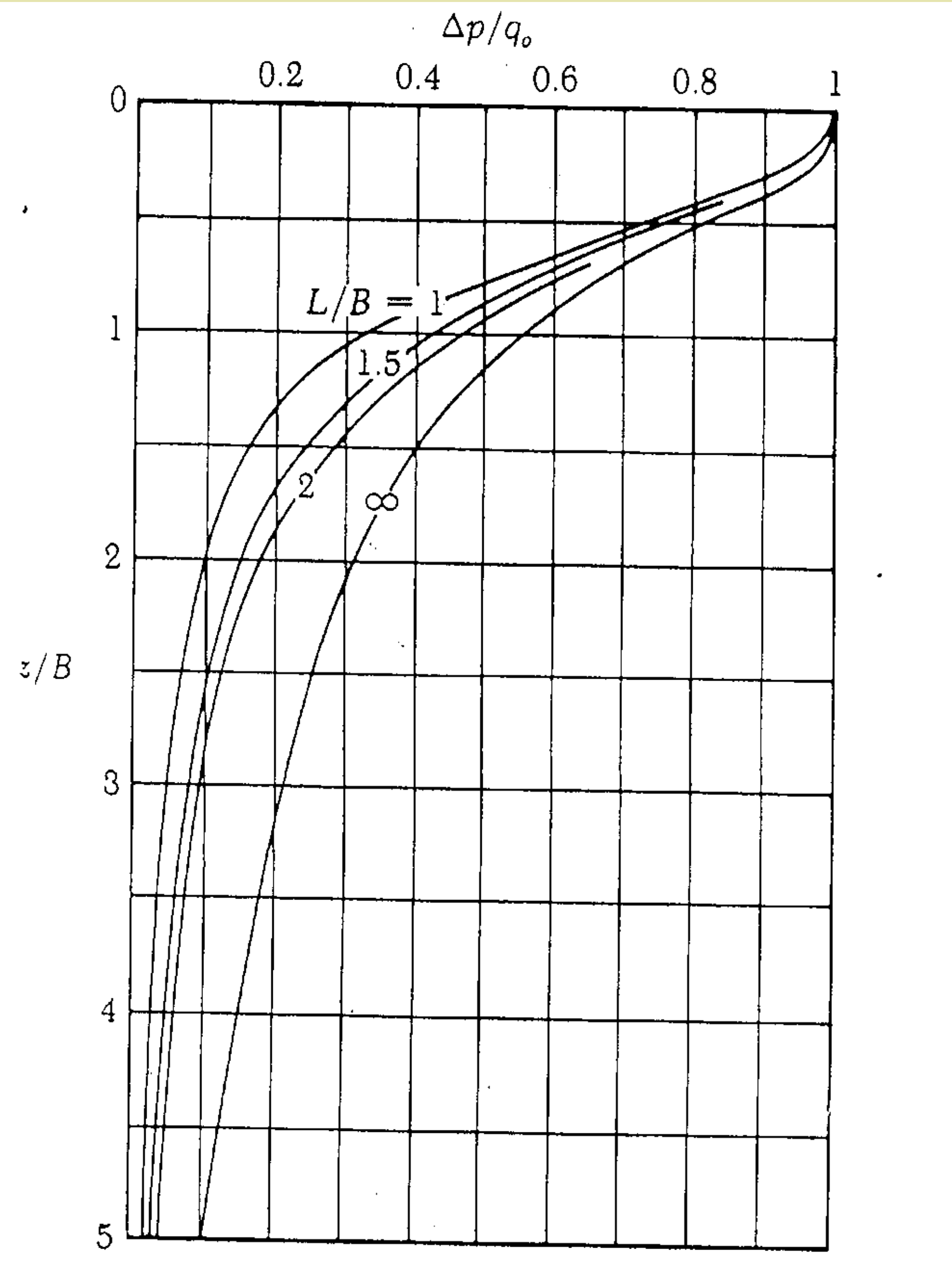
(use $L / B = 1$)

Consolidation Settlement - Long Term Settlement

2. <u>Final Settlement Analysis</u> (cont'd.)

(e) <u>EXAMPLE – Final Consolidation Settlement</u> (cont'd.)

Stress Increase in a Soil Mass Caused by Foundation Load



Das "Principle of Foundation Engineering", 3rd Edition

Figure 3.41 Increase of stress under the center of a flexible loaded rectangular area

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(e) EXAMPLE – Final Consolidation Settlement (cont'd.)

6) The final settlement, not using the table:

$$\Delta H = \sum \Delta H_i \frac{\Delta e_i}{1+e_0} = 2m \times \frac{0.1188}{1+1} + 2m \times \frac{0.0165}{1+1} + 4m \times \frac{0.003}{1+1} + 4m \times \frac{0.001}{1+1} = 0.14m$$
$$= 14cm$$

note: upper 2m contributes ≈ 85% of the total settlement

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(e) EXAMPLE – Final Consolidation Settlement (cont'd.)

6) The final settlement, not using the table: (cont'd.)

Skempton - Bjerrum Modification

Use Figure 5.31, p. 276

$A \cong 0.4$ $Hc/B >>> 2$ Settlement ratio < 0.57

$Sc < 0.57 \times 14 = 8cm$ $Sc < 8cm$

- **Check solution when using equation 5.84 and the average stress increase:**

$$\Delta\sigma'_{av} = \frac{1}{6}(\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b)$$

Like before, assume a layer of 3B = 12m

$\Delta\sigma'_t = q_o = \frac{1000kN}{16} = 62.5$ kPa $\Delta\sigma'_m$ (@6m = 1.5B) $\cong 0.16q_o$

$\Delta\sigma'_b$ (@12m = 3B) $\cong 0.04q_o$

$\Delta\sigma'_{av} = 1/6\ (1 + 4 \times 0.16 + 0.04)q_o = 1/6 \times 1.68 \times 62.5 = 0.28 \times 62.5 = 17.5$ kPa

$\Delta\sigma'_{av} = 17.5$ kPa

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(e) EXAMPLE – Final Consolidation Settlement (cont'd.)

6) The final settlement, not using the table: (cont'd.)

$Z = 6m$, $Z/B = 1.5$, $\frac{\Delta q}{q_0} = 0.28$ $\Delta q = 17.5$ kPa

$P_o' = 60kPa$, $P_c' = 120kPa$ $P_f' = 77.5kPa$

$\Delta e = C_r \log \frac{77.5}{60} = 0.05 \times 0.111 = 0.0056$

$\frac{\Delta e}{1+e_0} \times \Delta H = \frac{0.0056}{1+1} \times 12m = 0.033m = 3.33$ cm

Why is there so much difference?

As OCR does not change with depth, the influence of the additional stresses decrease very rapidly and hence the concept of the "average point" layer does not work well in this case. The additional stresses at the representative point remain below the maximum past pressure and hence large strains do not develop. The use of equation 5.84 is more effective with a layer of a final thickness.

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(f) Terzachi's 1-D Consolidation Equation

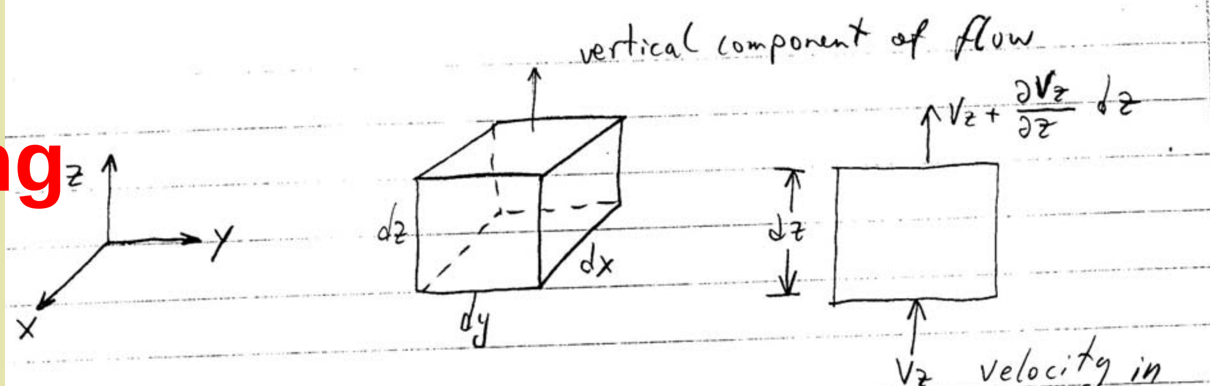
Terzaghi used the known diffusion theory (e.g. heat flow) and applied it to consolidation.

1) The soil is homogenous and fully saturated
2) The solid and the water are incompressible
3) Darcy's Law governs the flow of water out of the pores
4) Drainage and compression are one dimensional
5) The strains are calculated using the small strain theory, i.e. load increments produce small strains

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(f) Terzachi's 1-D Consolidation Equation (cont'd.)



① continuity. the volume change is the difference between what comes in and goes out.

$$\Delta V_z = \left[V_z - \left(V_z + \frac{\partial V_z}{\partial z} dz\right)\right] dx\, dy$$

② Darcy

$$V_z = K_z \cdot i_z = K_z \frac{\partial h}{\partial z} \qquad K = \text{coeff. of permeability}$$

---

① $\Delta V_z = -\frac{\partial V_z}{\partial z} dx\, dy\, dz$

subst (II) into (I) $\quad \Delta V_z = -\frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right) dx\, dy\, dz$

(a) $\Delta V_z = -K_z \frac{\partial^2 h}{\partial z^2} dx\, dy\, dz$

The volume of the water in the element:

$$V_w = \frac{S \cdot e}{1+e_o} \cdot V_T = \frac{S \cdot e}{1+e_o} dx\, dy\, dz$$

$$\Delta V_z = \frac{\partial V_w}{\partial t} = \frac{\partial}{\partial t}\left(\frac{S \cdot e}{1+e_o} dx\, dy\, dz\right) \quad \text{(b)} \quad \text{rate of change of water volume.}$$

$$V_s = \frac{V_T}{1+e_o} = \frac{dx\, dy\, dz}{1+e_o} \qquad \text{Volume of solid in the element} = \text{constant.}$$

Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

(f) Terzachi's 1-D Consolidation Equation (cont'd.)

5/13

$$\Delta V_z = \underbrace{-k_z \frac{\partial^2 h}{\partial z^2}\, dx\,dy\,dz}_{(a)} = \underbrace{\frac{1}{1+e_0}\frac{\partial}{\partial t}(s\cdot e)\, dx\,dy\,dz}_{(b)}$$

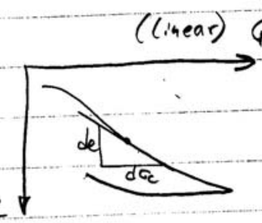
$$-k_z \frac{\partial^2 h}{\partial z^2} = \frac{1}{1+e_0}\frac{\partial}{\partial t}(s\cdot e) \qquad (c)$$

r.h.s

$$\frac{\partial}{\partial t}(s\cdot e) = \left(e\frac{\partial s}{\partial t} + s\frac{\partial e}{\partial t}\right)$$

$$\frac{\partial s}{\partial t} = 0 \quad \text{full saturation all the time.} \quad S=1$$

$$\frac{\partial}{\partial t}(s\cdot e) = s\frac{\partial e}{\partial t} = \frac{\partial e}{\partial t}$$



change of variables

$$\frac{\partial e}{\partial t} = \frac{\partial e}{\partial \sigma_v'}\frac{\partial \sigma_v'}{\partial t} = a_v \frac{\partial \sigma_v'}{\partial t}$$

a_v = coeff. of compressibility from e vs. σ' (linear) relations $a_v = -\frac{de}{d\sigma'} \neq const. \approx \frac{\Delta e}{\Delta \sigma'}$

$k_z = k_x = k_y = k$ assuming isotropic permeability

subst. in eq. (c)

$$-k\frac{\partial^2 h}{\partial z^2} = \frac{1}{1+e_0} a_v \frac{\partial \sigma_v'}{\partial t}$$

rearrange:

$$\frac{k(1+e_0)}{a_v}\frac{\partial^2 h}{\partial z^2} = -\frac{\partial \sigma_v'}{\partial t}$$

convinient change in h:

$$h = h_e + h_p = h_e + \frac{u}{\gamma_w} = h_e + \frac{1}{\gamma_w}(u_{ss} + u_e)$$

u_s = (mostly hydrostatic) = steady state

u_e = excess pore pressure

$$\frac{\partial^2 h}{\partial z^2} = \frac{\partial^2}{\partial z^2}\left[h_e + \frac{1}{\gamma_w}(u_{ss} + u_e)\right]$$

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Consolidation Settlement - Long Term Settlement

2. **Final Settlement Analysis (cont'd.)**

   (f) Terzachi's 1-D Consolidation Equation (cont'd.)

$$\frac{\partial^2 h_e}{\partial z^2} = 0$$

u_{ss} varies linearly so $\frac{\partial u_{ss}}{\partial z} = \text{const}$ & $\frac{\partial^2 u_{ss}}{\partial z^2} = 0$

hence:

$$\frac{k(1+e_0)}{a_v} \frac{\partial^2 u_e}{\partial z^2} \frac{1}{\gamma_w} - \frac{\partial \sigma_v'}{\partial t}$$

$$\frac{k(1+e_0)}{\gamma_w a_v} = C_v$$ = coefficient of consolidation. $[L^2/T]$ (m^2/sec)

$$C_v \frac{\partial^2 u_e}{\partial z^2} = -\frac{\partial \sigma_v'}{\partial t}$$

$$\sigma_v' = \sigma_t - u = \sigma_t - (u_{ss} + u_e)$$

$$\frac{\partial \sigma_t}{\partial t} = 0 \qquad \frac{\partial u_{ss}}{\partial t} = 0$$

$$-\frac{\partial \sigma_v'}{\partial t} = \frac{\partial u_e}{\partial t}$$

$$\boxed{C_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t}}$$ 1-D consolidation

C_v is a diffusion constant usually obtained directly from the consolidation test. It is actually not a constant but only due to our simplifications seeing a_v, k and e_0 constants, it's becomes one.

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Consolidation Settlement - Long Term Settlement

2. Final Settlement Analysis (cont'd.)

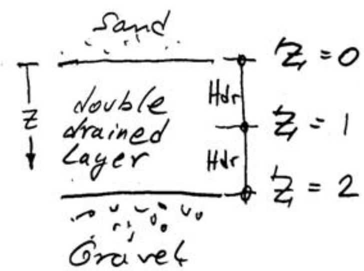
(f) Terzachi's 1-D Consolidation Equation (cont'd.)

(g) Time rate consolidation - chart solution.

$$C_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t} \qquad C_v = \frac{k(1+e_0)}{\gamma_w a_v}$$

Using simple, uniform inital excess pore pressure distribution we introduce nonedimensional variables

$$Z = \frac{z}{H_{dr}} \qquad T = \frac{C_v \cdot t}{H_{dr}^2}$$



The consolidation equation then becomes:

$$\frac{\partial^2 u_e}{\partial Z^2} = \frac{\partial u_e}{\partial T}$$

The solution of that equation has to satisfy the following Boundary Conditions (B.C.)

at $t = 0$ $\quad T = 0 \quad u_e = u_i \quad 0 \leq Z \leq 2$

at all t $\quad u_e = 0$ for $Z = 0$ & $Z = 2$

The solution is (Taylor 1948)

$$u_e = \sum_{m=0}^{m=\infty} \frac{2u_i}{M} (\sin M Z) e^{-M^2 T}$$

$M = \frac{\pi}{2}(2m+1)$ $\qquad$ m = dummy variable taking on values 1, 2, 3, . . .

We can show the solution on a graph where we present the Consolidation Ratio $U_z = 1 - \frac{u_e}{u_i} = f(Z, T)$

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Consolidation Settlement - Long Term Settlement

3. **Time Rate Consolidation** (sections 1.15 and 1.16 in the text, pp.38-47)

   (a) Outline of Analysis

   The consolidation equation is based on homogeneous completely saturated clay-water system where the compressibility of the water and soil grains is negligible and the flow is in one direction only, the direction of compression.

   Utilizing Darci's Law and a mass conservation equation ➔ rate of outflow - rate of inflow = rate of volume change; leads to a second order differential equation

$$C_v \frac{\partial^2 u_e}{\partial z^2} = \frac{\partial u_e}{\partial t} - \frac{\partial \sigma_v}{\partial t}$$

   u_e = excess pore pressure
   σ_v = vertical effective stress

   Practically, we use either numerical solution or the following two relationships related to two types of problems:

Consolidation Settlement - Long Term Settlement

3. <u>Time Rate Consolidation</u> (cont'd.)

(a) <u>Outline of Analysis</u> (cont'd.)

<u>Problem 1</u>: Time and Average Consolidation

<u>Equation</u> 1) $$t_i = \frac{T_v {H_{dr}}^2}{C_v}$$

t_i - The time for which we want to find the average consolidation settlement. See Fig. 1.21 (p.42) in the text, and the tables on p.56-58 in the notes.

T_v = time factor → T = f (U_{avg})

(L) H_{dr} = the layer thickness of drainage path.

$\left(\frac{L}{t}\right)$ C_v = coeff. of consolidation = $\frac{k}{\gamma_w m_v}$

m_v = coeff. Of volume comp. = $\frac{a_v}{1+e_0}$

av = coeff. Of compression = $\frac{\Delta e}{\Delta p}$

Consolidation Settlement - Long Term Settlement

3. Time Rate Consolidation (cont'd.)

(a) Outline of Analysis (cont'd.)

Problem 1: Time and Average Consolidation

Equation 2) $$U_{avg} = \frac{S_t}{S_\infty} = \frac{Settlement\ of\ the\ layer\ at\ time\ t}{Final\ settlement\ due\ to\ primary\ consolidation}$$

For initial constant pore pressure with depth

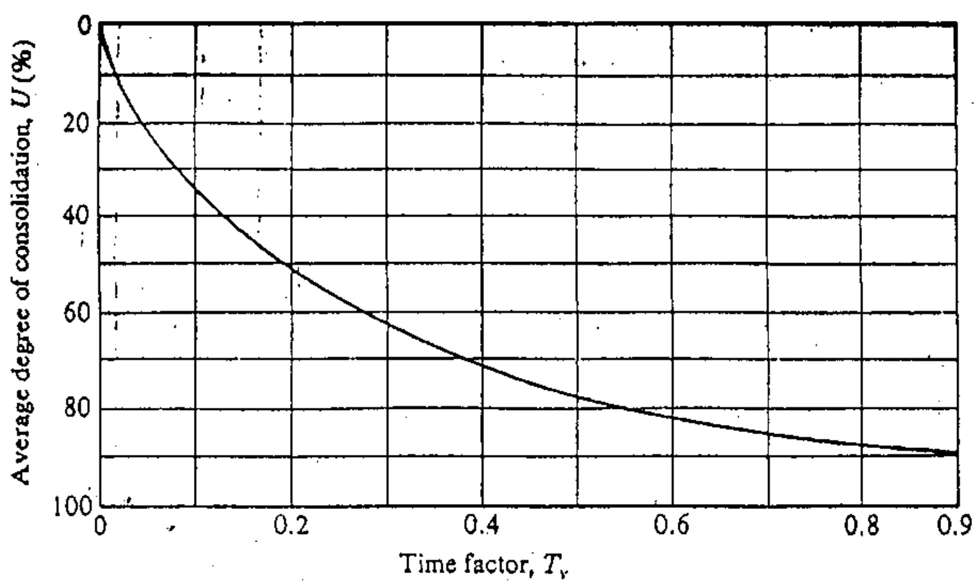


Figure 7.25 Variation of average degree of consolidation with time factor, T_v (u_0 constant with depth)

Table 7.3 Variation of Time Factor with Degree of Consolidation* (p. 42)

Degree of consolidation U%	Time factor, T_v
0	0
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848
100	∞

*u_0 is constant with depth

Consolidation Settlement - Long Term Settlement

3. **Time Rate Consolidation** (cont'd.)

(a) Outline of Analysis (cont'd.)

Problem 2: Time related to a consolidation at a specific point

Equation 3) Degree of consolidation at a point $U_{zt} = 1 - \frac{u_{z,t}}{u_{z,0}}$

Pore pressure at a point (distance z, time t) $U_{z,t} = \gamma_w \times hw_{z,t}$

For initial linear distribution of Δui the following distribution of pore pressures with depths and time is provided

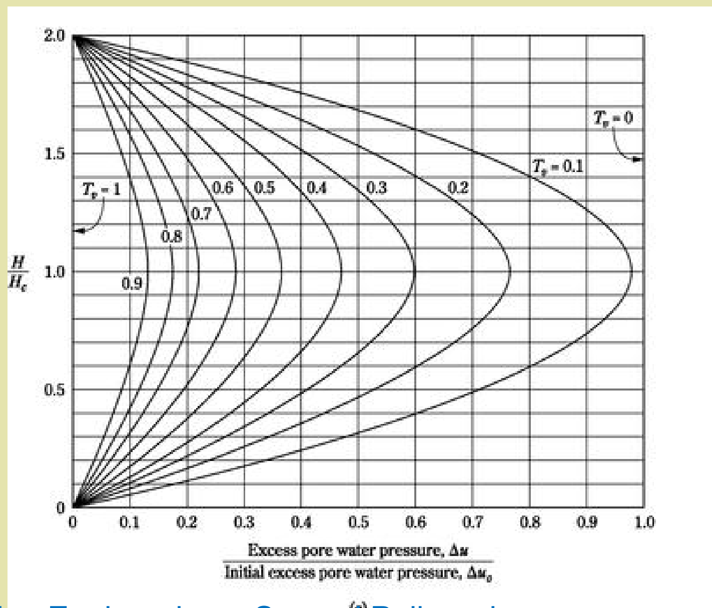
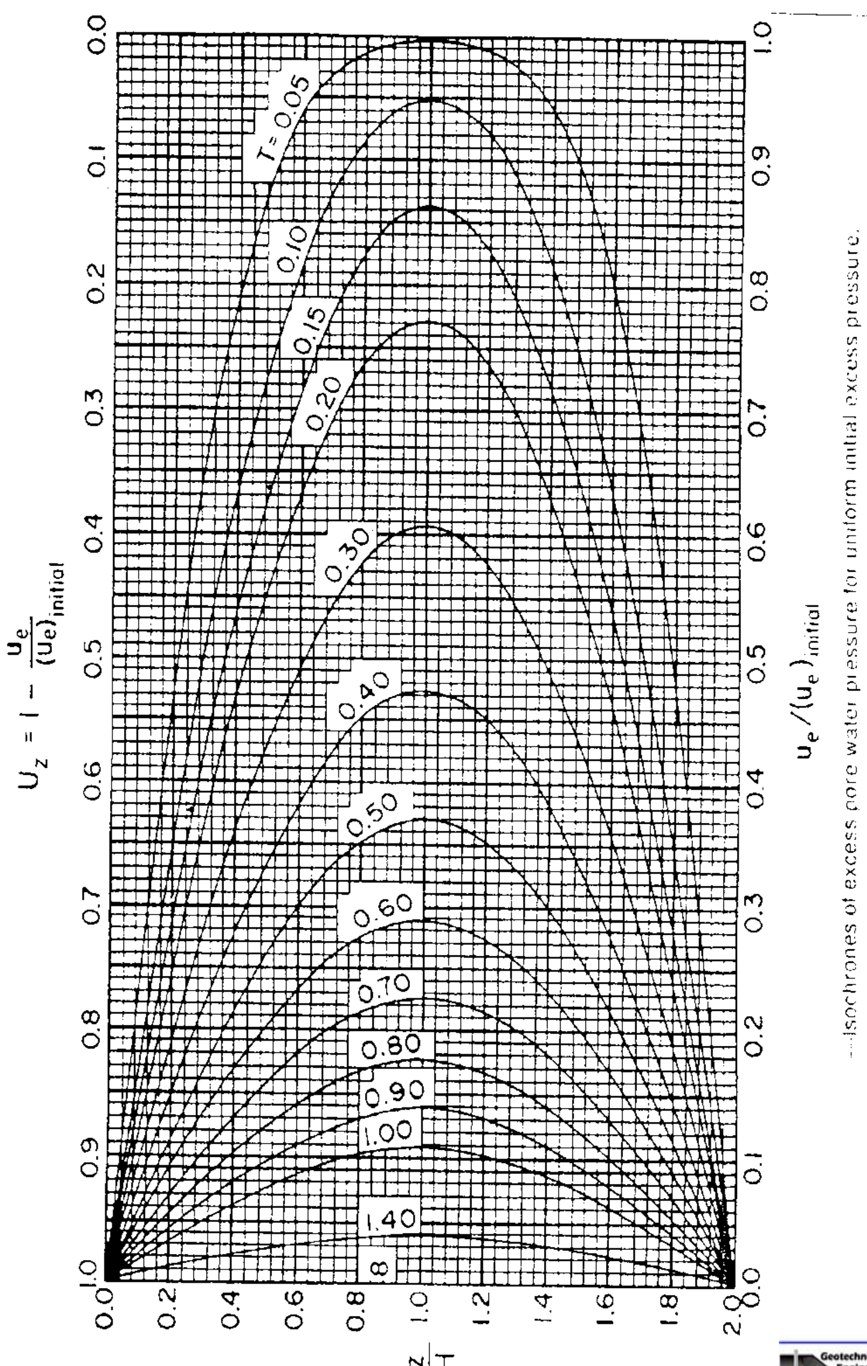


Fig. 1.20 (c)
Plot of $\Delta u/\Delta u_o$ with T_v and H/H_c (p.39)

Consolidation Settlement - Long Term Settlement



Consolidation
Settleme
Term Set

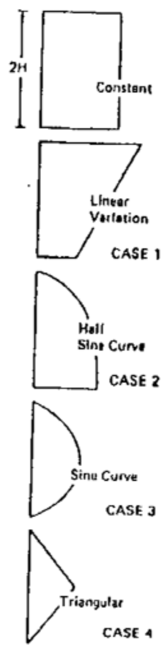
Table 1

One-Dimensional Consolidation Theory: Solutions for Four Cases of Initial Excess Pore Water Pressure Distribution in Double-Drained Stratum.

Average Degree of Consolidation for Various Values of T

T	Average Degree of Consolidation, U (%) Case 1	Case 2	Case 3	Case 4
0.004	7.14	6.49	0.98	0.80
0.008	10.09	8.62	1.95	1.60
0.012	12.36	10.49	2.92	2.40
0.020	15.96	13.67	4.81	4.00
0.028	18.88	16.38	6.67	5.60
0.036	21.40	18.76	8.50	7.20
0.048	24.72	21.96	11.17	9.60
0.060	27.64	24.81	13.76	11.99
0.072	30.28	27.43	16.28	14.36
0.083	32.51	29.67	18.52	16.51
0.100	35.68	32.88	21.87	19.77
0.125	39.89	36.54	26.54	24.42
0.150	43.70	41.12	30.93	28.86
0.175	47.18	44.73	35.07	33.06
0.200	50.41	48.09	38.95	37.04
0.250	56.22	54.17	46.03	44.32
0.300	61.32	59.50	52.30	50.78
0.350	65.82	64.21	57.83	56.49
0.400	69.79	68.36	62.73	61.54
0.500	76.40	76.28	70.88	69.95
0.600	81.56	80.69	77.25	76.52
0.700	85.59	84.91	82.22	81.65
0.800	88.74	88.21	86.11	85.66
0.900	91.20	90.79	89.15	88.80
1.000	93.13	92.80	91.52	91.25
1.500	98.00	97.90	97.53	97.45
2.000	99.42	99.39	99.28	99.26

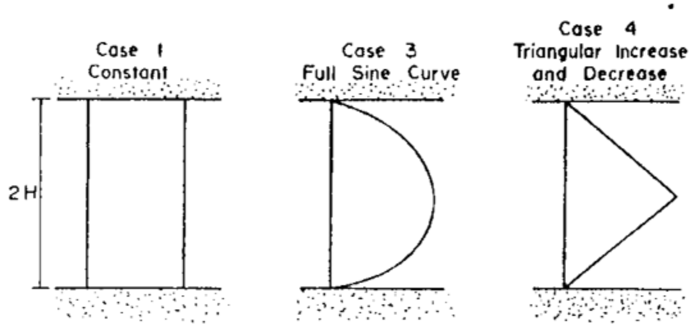
Distributions of Initial Excess Pore Water Pressure



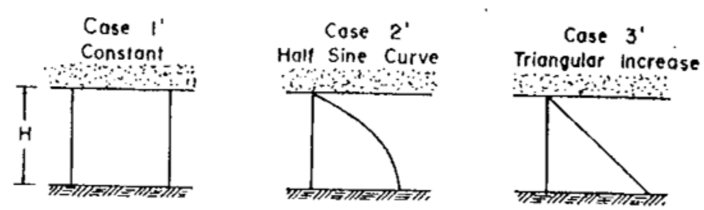
One-Dimensional Consolidation Theory: Time Factor for Various Average Degrees of Consolidation Double-Drained Stratum

U (%)	Time Factor T Case 1	Case 2	Case 3	Case 4
0	0	0	0	0
5	0.0020	0.0030	0.0208	0.0250
10	.0078	.0111	.0427	.0500
15	.0177	.0238	.0659	.0753
20	.0314	.0405	.0904	.101
25	.0491	.0608	.117	.128
30	.0707	.0847	.145	.157
35	.0962	.112	.175	.187
40	.126	.143	.207	.220
45	.159	.177	.242	.255
50	.197	.215	.281	.294
55	.239	.257	.324	.336
60	.286	.305	.371	.384
65	.342	.359	.425	.438
70	.403	.422	.488	.501
75	.477	.495	.562	.575
80	.567	.586	.652	.665
85	.684	.702	.769	.782
90	0.848	0.867	0.933	0.946
95	1.129	1.148	1.214	1.227
100	∞	∞	∞	∞

see Table 1 for initial excess pore pressure distribution



a) - Double - Drained Stratum



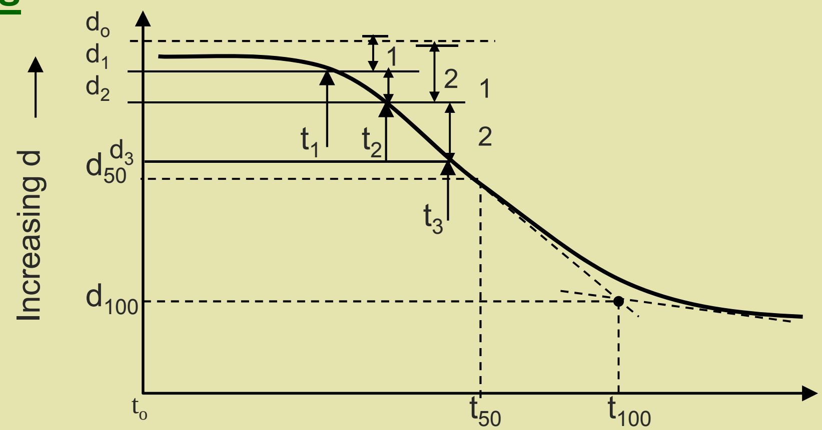
b) - Single - Drained Stratum

Fig. 7.8 – Initial excess pore water pressure distribution for double-drained and single-drained strata for which Table 1 is applicable.

Consolidation Settlement - Long Term Settlement

3. Time Rate Consolidation (cont'd.)

(b) Obtaining Parameters from the Analysis of e-log t Consolidation Test Results



1. find d_o - 0 consolidation time t = 0
   set time t_1, $t_2 = 4t_1$, $t_3 = 4t_2$
   find corresponding d_1, d_2, d_3
   offset $d_1 - d_2$ above d_1 and $d_2 - d_3$ above d_2
2. find d_{100} - 100% consolidation
   referring to primary consolidation (not secondary).
3. find d_{50} and the associated t_{50}

Consolidation Settlement - Long Term Settlement

3. **Time Rate Consolidation** (cont'd.)

   (b) **Obtaining Parameters from the Analysis of e-log t Consolidation Test Results (cont'd.)**

Coefficient of consolidation

$$C_v = \frac{T_i {H_{dr}}^2}{t_i}$$

T_i = time factor (equation 1.75, p.41 of text)

H_{dr} = drainage path = ½ sample

t_i = time for i% consolidation

Using 50% consolidation and case I

$$C_v = \frac{0.197 {H_{dr}}^2}{t_{50}}$$

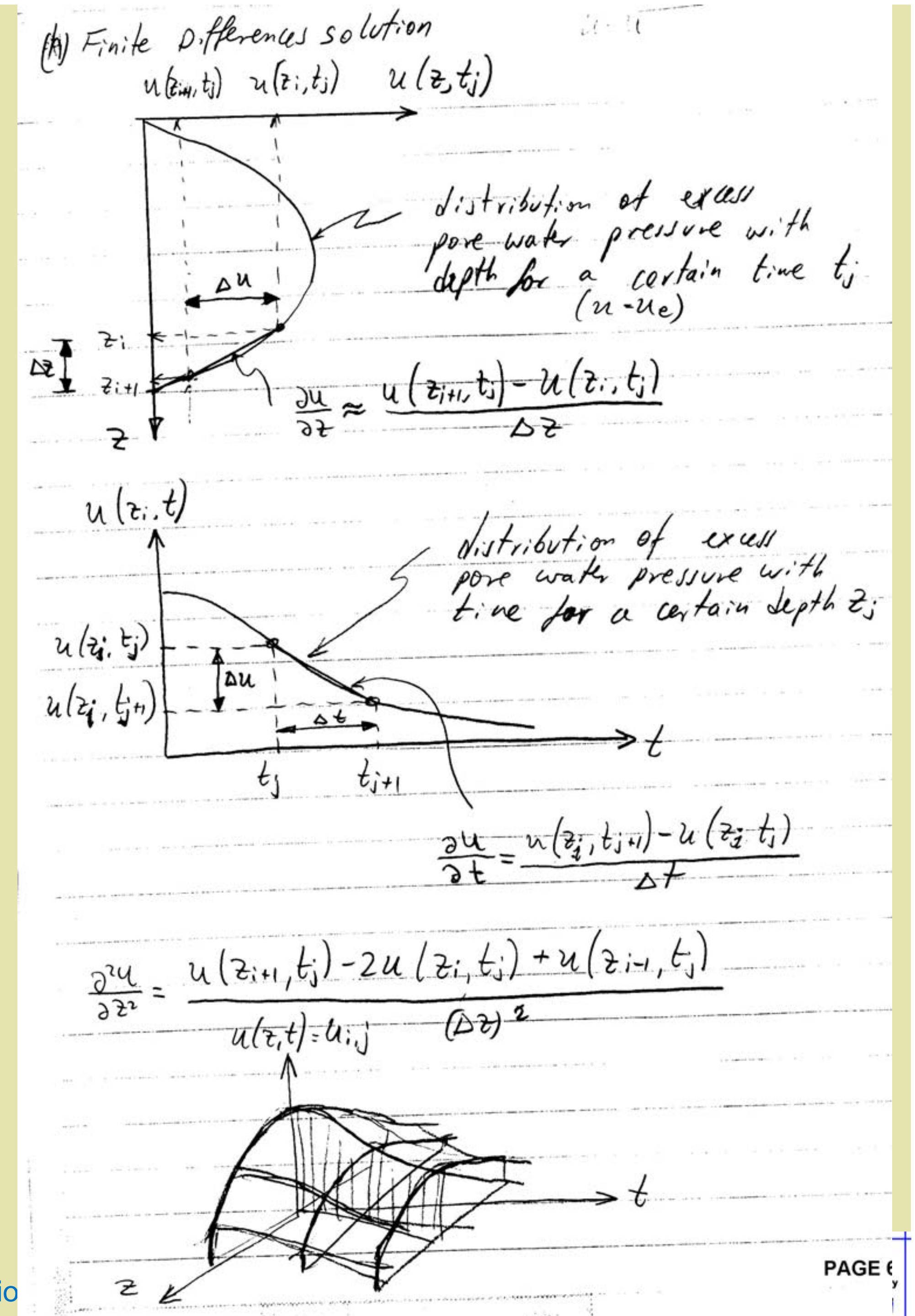
T for Uavg = 50%

and linear initial distribution

Consolidation Settlement - Long Term Settlement

3. <u>Time Rate Consolidation</u> (cont'd.)

(b) **<u>Obtaining Parameters from the Analysis of e-log t Consolidation Test Results</u> (cont'd.)**



Consolidation Settlement - Long Term Settlement

3. **Time Rate Consolidation** (cont'd.)

(b) **Obtaining Parameters from the Analysis of e-log t Consolidation Test Results (cont'd.)**

Coefficient of consolidation

For simplicity we can write $u(z_{iH}, t_j) = u_{i+1,j}$

$$C_v = \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t}$$

Substitute

$$C_v \frac{\left(u_{i+1,j} - 2u_{i,j} + u_{i-1,j}\right)}{\Delta z^2} = \frac{\left(u_{i,j+1} - u_{i,j}\right)}{\Delta t}$$

Which can easily be solved by a computer. For simplicity we can rewrite the above equation as:

$$u_{i+1,j} = \alpha u_{i+1,j} + (1 - 2\alpha) u_{i,j} + \alpha u_{i-1,j}$$

For which:

$$\alpha = \frac{C_v \cdot \Delta t}{(\Delta z)^2} \leq 0.5$$

Consolidation Settlement - Long Term Settlement

3. **Time Rate Consolidation** (cont'd.)

   (b) **Obtaining Parameters from the Analysis of e-log t Consolidation Test Results (cont'd.)**

Coefficient of consolidation

For $\alpha = 0.5$ we get:

$$u_{i,j+1} = \frac{1}{2}\left(u_{i-1,j} + u_{i+1,j}\right)$$

This form allows for hand calculations

e.g. For i=2, j=3 $u_{2,4} = ½ (u_{1,3} + u_{3,3})$

Consolidation Settlement - Long Term Settlement

3. Time Rate Consolidation (cont'd.)

(b) Obtaining Parameters from the Analysis of e-log t Consolidation Test Results (cont'd.)

Example

Find $u(z,t)$ using the simplified finite differences solution for double drainage and rectangular initial pore pressure distribution

$n = 10$ no. of sublayers

$C_v = 10^{-5}$ m²/min.

$\Delta\sigma_v' = 5.0$ t/m²

$H = 25$ m

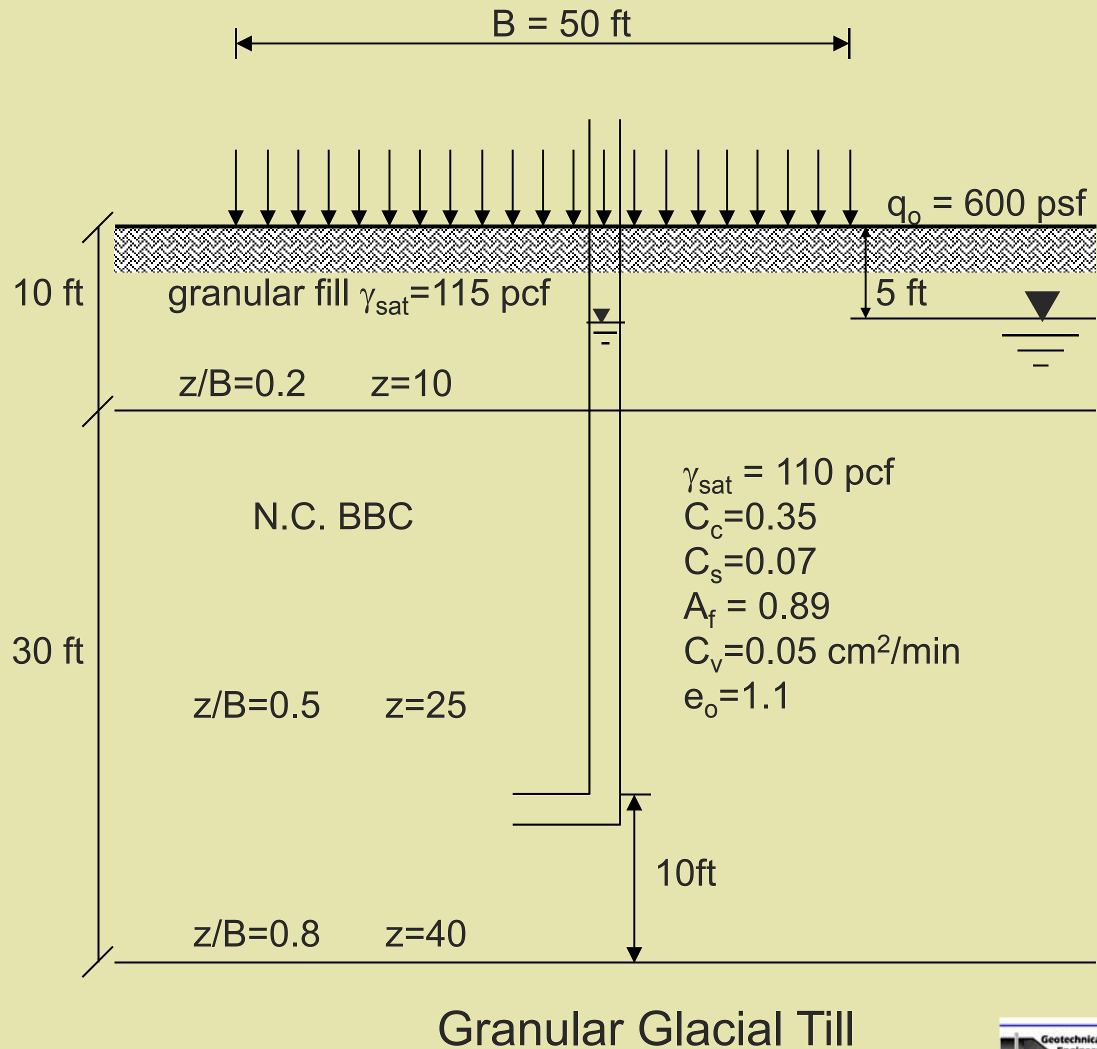
$$\Delta t = \frac{\alpha(\Delta z)^2}{C_v} = \frac{0.5 \times 2.5^2}{10^{-5}} = 3.125 \cdot 10^5 \text{ minutes} = 217 \text{ days}$$

i		0	1	2	3	4	5	6	7	8	9	10
z_i meter מטר		0	2.5	5.0	7.5	10.0	12.5	15.0	17.5	20.0	22.5	25.0
time זמן j	t_j ימים days	u (z,t) הלחץ הנקבובי pore pressure										
	0⁻	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00
0	0	2.50	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	5.00	2.50
1	217	0	3.75	5.00	5.00	5.00	5.00	5.00	5.00	5.00	3.75	0
2	434	0	2.50	4.37	5.00	5.00	5.00	5.00	5.00	4.37	2.50	0
3	651	0	2.18	3.75	4.69	5.00	5.00	5.00	4.69	3.75	2.90	0
4	868	0	1.87	3.44	4.37	4.84	5.00	4.84	4.37	3.44	1.88	0
5	1085	0	1.72	3.12	4.14	4.69	4.84					0
6	1302	0	1.56	2.93	3.91	4.49	4.69					0
7	1519	0	1.46	2.73	3.71	4.30	4.49					0
8	1736	0	1.38	2.59	3.51	4.10	4.30					0
9	1953	0	1.29	2.44	3.34	3.90	4.10					0
10	2170	0	1.20	2.32	3.17	3.72	3.90					0

Consolidation Settlement - Long Term Settlement

4. Consolidation Example

The construction of a new runway in Logan Airport requires the pre-loading of the runway with approximately 0.3 tsf. The simplified geometry of the problem is as outlined below, with the runway length being 1 mile.



Consolidation Settlement - Long Term Settlement

4. Consolidation Example (cont'd.)

1) Calculate the final settlement.

Assuming a strip footing and checking the stress distribution under the center of the footing using Fig. 3.41 (p. 12 of the notes)

Location	z (ft)	z/B	$\Delta q / q_o$	Δq (psf)
Top of Clay	10	0.2	~0.98	588
Middle of Clay	25	0.5	~0.82	492
Bottom of Clay	40	0.8	~0.60	360

Using the average method

$$\Delta\sigma'_{av} = \frac{1}{6}(\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b) = \frac{1}{6}(588 + 4 \times 492 + 360) = \underline{\mathbf{486psf}}$$

Consolidation Settlement - Long Term Settlement

4. Consolidation Example (cont'd.)

1) Calculate the final settlement (cont'd.)

The average number agrees well with the additional stress found for the center of the layer, (492psf).

Assuming that the center of the layer represents the entire layer for a uniform stress distribution. At 25 ft:

$p_o' = \sigma_v' = 115 \times 5 + (115 - 62.4) \times 5 + (110 - 62.4) \times 15$
$= 575 + 263 + 714 = 1552\text{psf}$
$p_f' = p_o' + \Delta q = 1552 + 486 = 2038\text{ psf}$
$\Delta e = C_c \log (p_f'/p_o') = 0.35 \log (2038/1552) = 0.0414$

$$s = \Delta H = H\left(\frac{\Delta e}{1 + e_0}\right) = 30ft \times 12inch \times \left(\frac{0.0414}{1 + 1.1}\right) = 7.1inch$$

Consolidation Settlement - Long Term Settlement

4. Consolidation Example (cont'd.)

2) Assuming that the excess pore water pressure is uniform with depth and equal to the pressure at the representative point, find:

(a) The consolidation settlement after 1 year

- Find the time factor:

$$t_i = \frac{T_v {H_{dr}}^2}{C_v} \qquad T_v = \frac{t_i C_v}{{H_{dr}}^2}$$

C_v = 0.05 cm^2/min = 0.00775 in^2/min
H_{dr} = H/2 = 30 ft / 2 = 15 ft
T_v = 12 × 30 × 24 × 60 × 0.00775 / (15 x 12)^2 = 0.124

- Find the average consolidation for the time factor.

For a uniform distribution you can use equation 1.74 (p.41) of the text or the chart or tables provided in the notes.

Consolidation Settlement - Long Term Settlement

4. Consolidation Example (cont'd.)

2) Assuming that the excess pore water pressure is uniform with depth and equal to the pressure at the representative point, find:

(a) The consolidation settlement after 1 year

- Find the average consolidation for the time factor.

Using the table in the class notes (p.56 & p.58)

T = 0.125 → Case I - uniform or linear initial excess pore pressure distribution. → U = 39.89 % = 40%

$$U_{avg} = \frac{S_t}{S_\infty} \qquad S_t = U_{avg} \times S_\infty$$

$S_t = 0.40 \times 7.1 =$ 2.84 inch

Consolidation Settlement - Long Term Settlement

4. Consolidation Example (cont'd.)

2) Assuming that the excess pore water pressure is uniform with depth and equal to the pressure at the representative point, find: (cont'd.)

(b) What is the pore pressure 10 ft. above the till 1 year after the loading?

From above; t = 12 months, T = 0.124
$2\ Hd_r = 30$ ft
$z / H_{dr} = 20/15 = 1.33$ (z is measured from the top of the clay layer)

Using the isochrones with T = 0.124 and z/H = 1.33
We get $u_e / u_i \approx 0.8$
$u_e = 0.8 \times 486 = 389$ psf

Consolidation Settlement - Long Term Settlement

4. Consolidation Example (cont'd.)

2) Assuming that the excess pore water pressure is uniform with depth and equal to the pressure at the representative point, find: (cont'd.)

   (c) What will be the height of a water column in a piezometer located 10 ft above the till: (i) immediately after loading and (ii) one year after the loading?

(i) u_i = 486 psf $h_i = u/\gamma_w = 486/62.4 = 7.79$ft.

(ii) u_e = 389 psf $h = u/\gamma_w = 389 / 62.4 = 6.20$ ft

The water level will be 2.79 ft. above ground and 1.2 ft above the ground level immediately after loading and one year after the loading, respectively.

Consolidation Settlement - Long Term Settlement

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THE TWENTY-SEVENTH TERZAGHI LECTURE

Presented at the American Society of Civil Engineers

1991 Annual Convention

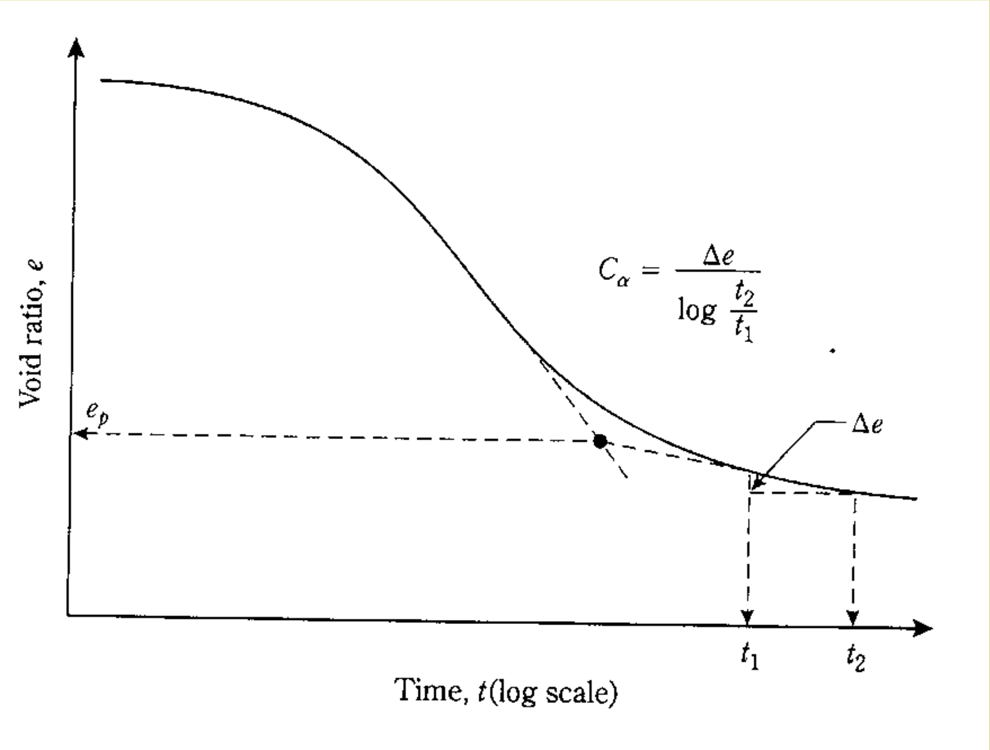
October 22, 1991

J. MICHAEL DUNCAN

Consolidation Settlement - Long Term Settlement

5. **Secondary Consolidation (Compression) Settlement**

Figure 5.33 (p.279)
(a) Variation of e with log t under a given load increment, and definition of secondary compression index.



Consolidation Settlement - Long Term Settlement

5. **Secondary Consolidation (Compression) Settlement (cont'd.)**

Following the full dissipation of the excess pore pressure, (primary consolidation) more settlement takes place, termed secondary compression or secondary consolidation. This settlement under constant effective stresses is analogous to creep in other materials. The secondary consolidation is relatively small in regular clays but can be dominant in organic soils, in particular peat.

$$C_\alpha = \frac{\Delta e}{\log\left(t_2/t_1\right)}$$

[relate to any 2 points on the secondary compression curve]

Magnitude of secondary consolidation:

$$S_{c(s)} = \frac{\Delta e}{1+e_0} H_c$$

where: $\Delta e = C_\alpha \log\left(t_2/t_1\right)$

[relate to the time of interest]

Clays $C_\alpha / cc \approx 0.045 \pm 0.01$

Peats $C_\alpha / cc \approx 0.075 \pm 0.01$

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ENGINEERING PROPERTIES OF CRANBERRY BOG PEAT

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ABSTRACT

Peat is an organic complex soil, well known for its high compressibility and low stability. Peat forms naturally by the incomplete decomposition of plant and animal constituents under anaerobic conditions at low temperatures. A relocation of state highway No. 44 in Carver, Massachusetts requires the construction of sheet pile walls, fills and embankments through cranberry bogs and ponds containing deep peat deposits. The engineering properties of Carver peat in Southern Massachusetts (south of Boston) were investigated via laboratory testing including standard index tests, fiber content, engineering classification, consolidated undrained triaxial tests, and oedometer tests. The tests were carried out on vertically and horizontally oriented undisturbed samples. Unlike inorganic clays, the secondary compression of peat is of great significance as it dominates its deformation and takes place over a long period of time. The presented test program examines the deformation properties of the peat and the ratio of the coefficient of secondary compression (C_α) to compression index (C_c). The data are compared to those reported in the literature. The obtained engineering properties were found to be overall within the range reported for other peat types. The peat structure and fiber orientation seem to affect the properties. The time for primary consolidation for horizontally oriented samples decreases due to an increase in the horizontal permeability and the time of secondary compression increases due to compression mostly normal to the fibers' orientation.

1. INTRODUCTION

1.1 Background

US Route 44 spans east west across southeastern Massachusetts into Rhode Island. The Massachusetts Highway Department (MHD) is relocating Route 44 under project no. 113100. Parts of the new highway alignment spans across ponds and cranberry bogs in the town of Carver, located about 40 miles southeast of Boston. The proposed roadway is a four lane divided highway with a typical median width of 60 feet. Environmental concerns dictated that sheet piles need to be placed at the ponds and bogs roadway sections, in order to excavate and replace the underlaying organic soils and construct the embankments and roadway. The design of sheet piles supported by organic soils raises the difficulties of assigning engineering parameters to peat. These difficulties prevail whenever other engineering alternatives are considered. The objective of the presented work is to assess the engineering properties of the peat found along the proposed highway, and which is currently supporting the sheet piling. The investigated properties are to be utilized in the analysis of the supporting sheet piles and compared with the wall performance during construction as monitored by instrumentation.

1.2 Subsurface Conditions

Extensive subsurface investigation shows that the soil type and density is relatively consistent throughout the project and the wetland areas. The soil profile consists primarily of fibrous peat within a fine to coarse sand layer. The thickness of the Peat deposits range from 0 to 10.7 m (35 feet) and the ground

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water table is near the ground surface. Figure 1 depicts a longitudinal section for a 427m (1,400ft.) long segment of the constructed route (a portion of the project) including the pond location from which the tested Peat samples were obtained (around station no. 150). The presented section in Figure 1 includes 10 borings and numerous probes outlining the contour of the Peat layer.

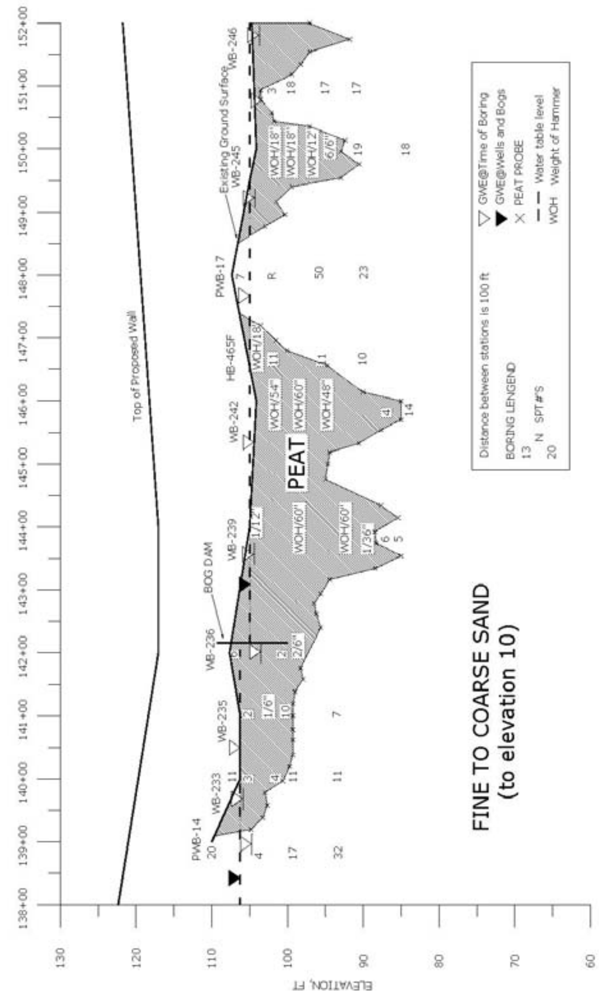

Fig.1 Typical subsurface conditions in the wetland area along a section of Route 44 between stations 138 and 152.

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1.3 Peat/Bog-Overview

Peat is a material consisting of organic residues formed through the decomposition of plant and animal constituents under aerobic and anaerobic conditions associated with low temperatures and geological effects such as glacial ice. Common names for accumulation of organic soils include bog, fen, moor, muck, and muskeg. Cranberry is a Native American wetland fruit, which grows in places, called in Massachusetts, a Bog. Natural bogs evolved from organic deposits accumulation in kettle holes created by glaciers. Peat exhibits poor strength and undergoes large deformations over a long period of time. As a result, Peat and organic soils are characterized as being among the worst kinds of foundation material associated with low bearing capacity, high compressibility and long-term settlement. In most cases, the majority of the settlement in peat results from creep at a constant vertical effective stress (secondary compression) accounting for more than 60% of the total settlement. Among geotechnical materials, peat has the highest values of the ratio between coefficient of secondary compression-to-compression index; $C_\alpha / C_c = 0.06 \pm 0.1$ whereas for comparison, granular materials may display the lowest values of $C_\alpha / C_c = 0.02$ (Mesri et. al. 1997). Due to the high water content and the plant matter structure, Peat deposits accumulate at high initial void ratio (e) varying typically from 5 to 16 depending on the water content. Peat particles are light because of the lower specific gravity of the organic matter, resulting with a typical natural unit weight ranging from 9.1 to 11.6 kN/m^3. When the organic matter decomposes, it turns into a sort of glue called humus, which is strong enough to bind several smaller particles together, making them into larger multi-particles, which can alter the behavior of the soil.

1.4 Design Consideration

Often a site is chosen for construction irrespectively of its geotechnical suitability but for its location; such is the case for route 44 relocation project. Due to unpredictable long-term settlement of organic soils, construction over such soils is usually impractical without a complete replacement or some sort of soil treatment. Many methods exist to improve sites with underlaying soft organic soils including, surcharging techniques to expedite the consolidation process, displacement method of placing fill directly on top of the deposit (which then by its weight, sinks and displaces the weak soil) or the use of geosynthetic products to either bridge over limited areas or to generate more evenly distributed settlement. For deep deposits, pile foundations may be employed to transfer loads through the organic soils to a firm lower layer or other methods of similar principles utilizing columns of gravel or cement and fill layers with or without synthetic material to bridge between them. Two challenges exist in the Route 44 project under the sheet pile construction requirement, one is the construction of the sheet pile itself having the peat as a reactive material, and the other is the treatment of the peat between the sheet piles. Embankments, walls, service roads and the highway are planned to be built in the area between the sheet piles. Due to long term maintenance concerns and the cost of alternative solutions, excavation and soil replacement were chosen for these areas. For the sheet piles themselves, no alternatives exist and hence their construction required the development of lateral loads in the peat. This study presents therefore experimental findings for the engineering qualities of the peat when loaded both; vertically and horizontally.

2. EXPERMENTAL PROGRAM AND BASIC PROPERTIES

2.1 Planned Testing and Sampling

Table 1 presents a summary of the laboratory study detailing the type and number of tests planned and executed thus far. Issues considered included the knowledge of basic soil properties, index parameters, strength and deformation of vertically and horizontally oriented samples as well as the effect of the sample size on the obtained results. Due to the size of the fibers and roots in the tested peat the size of the common soil test samples relative to the fiber size became a concern. This factor along with the need for testing horizontally oriented samples and relatively shallow peat deposits in the bog, lead to a direct sampling from the surface utilizing large size samplers. Two square steel tubes dimensioned 15.24 x 15.24 cm (6x6 inch) and 25.4 x 25.4 cm (10x10 inch); both 6.35 mm (0.25 inch) thick and 1.83 m (6 feet) long, were used for sampling. The samplers were pushed into the peat from the surface at a location in which the water was at about the ground surface. A retainer (catcher) that was constructed for the sampler was found to be unnecessary as when pulled out, a full sample size was retained. The smaller and the larger size samples were designated as block (1) and block (2) respectively.

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2.2 Carver Peat Characteristics

The obtained peat, termed Carver peat is classified by the different Peat classifications in the following way; Fibrous according to the plasticity chart for peat suggested by Casagrande (1966), Fibric according to Lynn et al. (1974) and Hemic, high ash, moderately acidic, and highly absorbent according to ASTM D-4427. The color of carver peat is dark brown to brownish-orange, it has strong odor and contains small woody elements. The Humification degree of Carver peat is H_3 to H_4 using Von Post's Humification Scale, (ASTM D5715). The principal characteristics of the Carver peat are summarized in Table 2.

3. CONSOLIDATION TESTS

3.1 General Details

Three vertically oriented samples and four horizontally oriented samples were tested in oedometer cells with the details outlined in Table 3. The effective overburden pressure for the sampled peat (mid point) was approximately 1.2 kPa with effective preconsoildation pressure of approximately 9 kPa, and a resulting over consolidation ratio of about 7.5. Sample preparation of peat is more difficult than that of the typical inorganic soils due to the presence of fibers, the high initial water content and voids ratio. To minimize sample disturbance the samples were trimmed using a very sharp razor knife, and special care was taken in its placement. The porous stones were fully saturated before the test and filter papers were used to margin the biodegradation and decomposition of the samples. This is necessary considering the long period of time required for the consolidation tests in which each applied increment was sustained for about 10,000 minutes (1 week). A thin film of Silicone grease was applied to the cell wall in order to minimize the side friction. The consolidation tests were carried out at approximately constant temperature of 22 ± 4 °C.

Table 1. Testing program of Carver peat

Test Type	No. Of Planned Tests		No. Of Performed Tests		Comments
(Bulk Unit Weight)	2		2		One test for each block
(Specific Gravity)	2		2		
(Organic Content)	2		2		
pH	1		1		
(Liquid Limit), (Plastic Limit)	2		2		
	Vertical Samples		Horizontal Samples		
	Planned	Performed	Planned	Performed	
Consolidation Test	4	3	4	4	Different aspect ratios
Consolidated Undrained Triaxial Test	4	1	4	0	
Direct Shear Test	4	0	4	0	

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Table 2. Carver peat soil properties

Property	Unit	Block (1)	Block (2)	Reference
γ (Bulk Unit Weight)	kN/m^3	10.44	10.10	ASTM D4531
ω_0 (Water Content)	Percent	780.0 - 946.0	759.0 - 816.0	ASTM D2216
G_s (Specific Gravity)	-	1.48	1.52	ASTM D854
O_c (Organic Content)	Percent	60.0	77.0	ASTM D2974
pH	-	4.50	4.50	ASTM D2976
LL (Liquid Limit)	Percent	580.0	600.0	ASTM D4318
PL* (Plastic Limit)	Percent	375.0	400.0	ASTM D4318
Fibers Content	Percent	40.0	52.0	ASTM D1997

* Rolling the sample to 4.5mm instead of 3 mm due of the presence of fibers

3.2 The Compression and Rebound Index

Figures 2 and 3 present the stress strain relations in the form of void ratio vs. consolidation pressure, (e-log σ) for samples oriented vertically and horizontally, respectively. Table 3 summarizes the compression index (C_c) and the rebound index (C_s) values for the different tests. The information presented in Table 3 show that for Block 1, the C_c values for samples oriented in the vertical and horizontal directions are 5.2 and 4.1, respectively resulting in a horizontal to vertical compression index ratio (C_{ch} / C_{cv}) of 0.78. For Block 2, the C_c values for the vertical and horizontal samples are 3.9 and 2.7 respectively, resulting in a ratio of 0.70. Subjected to the limited number of tests, these values indicate that the sample orientation and the block from which the samples were obtained, affected the obtained results while the oedometer size and it's aspect ratio had no effect. As both peat samples were retrieved at the same location, the obtained compression index values may reflect the large variation in the peat or alternatively suggest that the peat in the blocks were influenced by the sampler's size such that the peat in the small sampler was compressed more during sampling than the peat in the large sampler. The compression index values found for Carver peat is compared in Figure 4 with other values presented by Mesri (1973) and Terzaghi et al. (1996). The relationship in Figure 4 suggest that the compression index of Carver peat agrees well with the other values attributed to peat with the exception of the horizontally oriented samples having lower compression index as discusses above. The rebound index values agree with the range reported by Mesri et al. (1997) of C_s between 0.3 to 0.9.

Table 3. Oedometer test details and compression and rebound index results

Test No.	e_o	ω_0 (%)	C_c	C_s	Block No.	Sample Orientation[1]	Oedometer Diameter (cm) /Aspect Ratio[2]
1	12.41	837	3.40	0.47	2	V	11.28/2.89
2	12.00	800	4.30	0.43	2	V	11.28/2.89
4	14.00	935	5.18	0.90	1	V	11.28/2.89
5	11.54	760	2.67	0.34	2	H	11.28/2.89
6	11.54	770	4.03	0.34	1	H	11.28/2.89
7	11.54	772	2.72	0.36	2	H	7.00/4.40
8	11.54	775	4.07	---	1	H	7.00/4.40

[1] V,H–Vertically and Horizontally samples, respectively [2] Ratio of original sample height to diameter

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3.3 Time – Settlement Relationship

The formulation of the consolidation process for fully saturated soils, (Terzaghi, 1923) assumes that the soil particles and water are incompressible and deformation takes place due to expulsion of water from the pores under the influence of hydrodynamic effects upon loading. This compression process, (termed primarily consolidation) assumes relationship between effective stresses to void ratio and should cease therefore when the dissipation of the excess pore-water pressure is completed. In fine-grained soils the compression continues after the dissipation of pore water pressure is completed and takes place under a constant effective stress in what is termed secondary compression or creep. Due to the high permeability of peat, the primary consolidation is relatively short but the secondary compression takes place over a lengthy period of time and hence is of great significance. The secondary compression has been attributed to the plastic deformation of the highly viscous adsorbed double layer and continuous adjustment and arrangement of soil constituents after they have been distributed during the primary consolidation, (Dhowian, 1978). Accordingly, the primary consolidation method of settlement analysis developed by Terzaghi seems to be inappropriate to address the secondary compression. Many investigators have assumed and used different relationships and models to describe the secondary compression. Gibson and Lo (1961) identified three types of secondary compression curves relating to the relationship between settlement and time on a logarithm scale; *type 1* shows a gradual decrease in the rate of secondary compression until ultimate settlement is finally reached; *type 2* exhibits a proportional relationship between secondary compression and logarithm of time for a significantly long period of time before reaching the final settlement, and *type 3* shows a proportional relationship to a certain point at which an acceleration of the rate of secondary compression takes place, believed to be the result of bond breakage of between particles. The compression-log time curve of type 3 materials consists of four components of strain; instantaneous strain which takes place immediately after load application, primary strain which lasts in most cases for several minutes, secondary strain which has a constant rate with log time and lasts for a considerable period of time and tertiary strain which is a higher rate secondary strain. This phenomenon is believed to be due to the breakage of bonds between particles and a curved transition zone usually exists from the secondary to the tertiary zones.

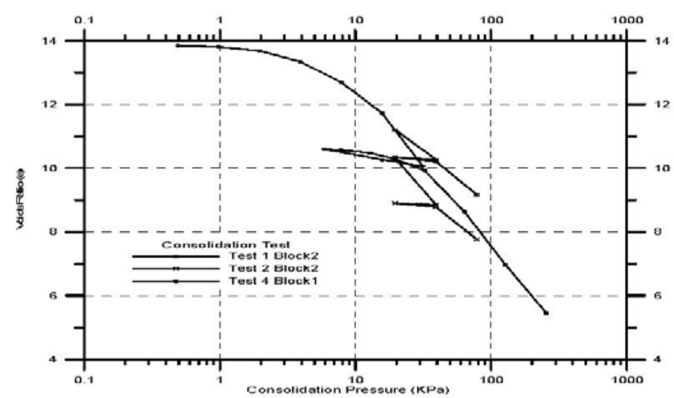

Fig. 2. Void ratio versus consolidation pressure for samples oriented vertically

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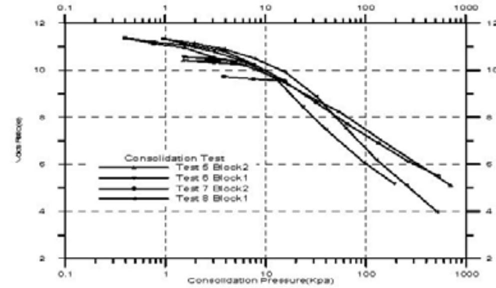

Fig. 3. Void ratio versus consolidation pressure for samples oriented horizontally

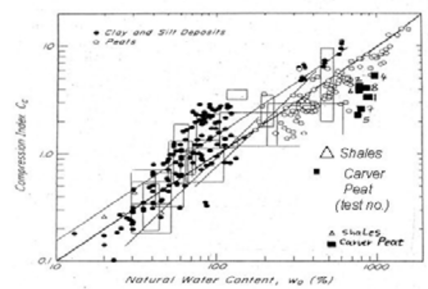

Fig. 4. Empirical correlation between compression index and in situ water content for clay, silt, shales and peats including the current study (based on Mesri, 1973 and Terzaghi et al. 1996).

3.4 Secondary and Tertiary Compression

Obtained relations and the related parameters

Figures 5 and 6 describe some of the relationships between the void ratio and time for the vertically oriented samples under different loads. The obtained relations show that Carver peat behavior is in agreement with the aforementioned type 3 curves, exhibiting an accelerated rate of secondary compression. Dhowian and Edil (1980) and Mesri and Choi (1985) suggested that secondary compression begins after the primary compression ends; these hypothesis was adopted in this research study finding the related parameters in the following way:

(i) t_p - the time at the end of primary consolidation, (EOP) employing Taylor's square root method (Taylor, 1942).

(ii) C_α - the coefficient of secondary compression, defining the tangentional slope (δ_e/δ_t).

(iii) t_k – the designated time for the end of secondary compression and the beginning of the tertiary compression, defined by the interception of the tangents to the curves in the secondary and tertiary zones.

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(iv) C_k -the coefficient of tertiary compression, defining the tangentional slope after the transitional zone between the secondary and the tertiary compression, (Edil and Dhowian 1979; Dhowian and Edil 1980).

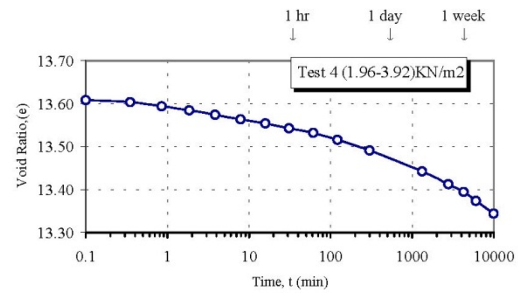

Fig. 5. Typical settlement vs. time relationship (e - log t) for test 4 under low stress level

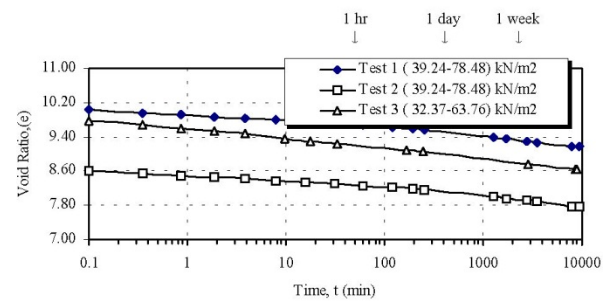

Fig. 6. Typical settlement vs. time relationship (e - log t) for the vertically oriented samples under similar stress levels

The time for secondary and tertiary compression

Figures 7a,b present the time in which the primary compression is completed and the secondary compression starts, (t_p) versus the consolidation pressure for vertically and horizontally oriented samples, respectively. In all cases, the primary consolidation takes place within 3 minutes, and for the horizontally oriented samples, the time is about one half of the time required to complete the primary consolidation in the vertically oriented samples. Figures 8a,b present the time in which the secondary compression is completed and the tertiary compression starts, (t_k) versus the consolidation pressure for vertically and horizontally oriented samples, respectively. The time of the secondary compression is measured in hundreds to thousands minutes with distinctive peak(s) at particular stress levels. Overall, the time required for secondary compression is longer in the horizontally oriented samples compared with the time required for the vertically oriented samples under the same consolidation pressure.

It seems that the behavior observed in figures 7 and 8 is associated with the structure of the peat and it's deposition process, having the majority of the fibers oriented horizontally. Such structure results with permeability in the horizontal direction being higher than in the vertical direction, and hence the time for the primary consolidation being shorter. In contrast, the structure in the vertical direction is more easily

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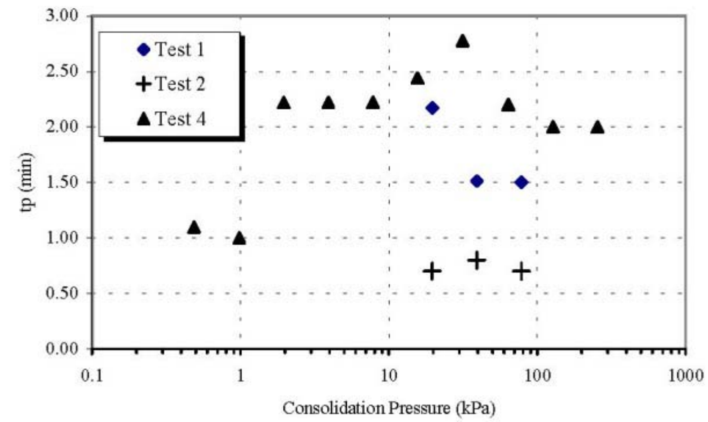
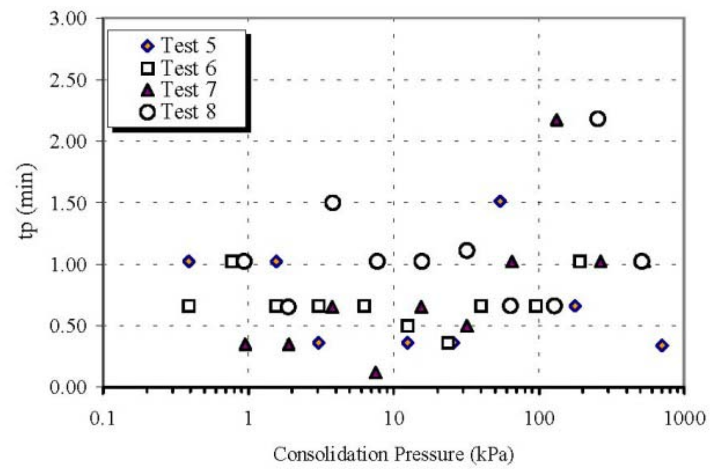
compressed (fibers in parallel) than in the horizontal direction, resulting with a shorter time for a secondary compression in the vertically oriented samples.



(a)



(b)

Fig. 7. The time to the end of primary consolidation and beginning of the secondary compression versus consolidation pressure for vertically (a) and horizontally (b) oriented samples

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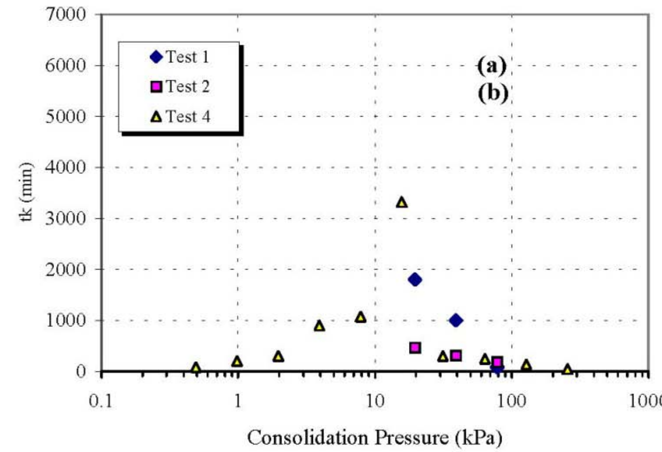
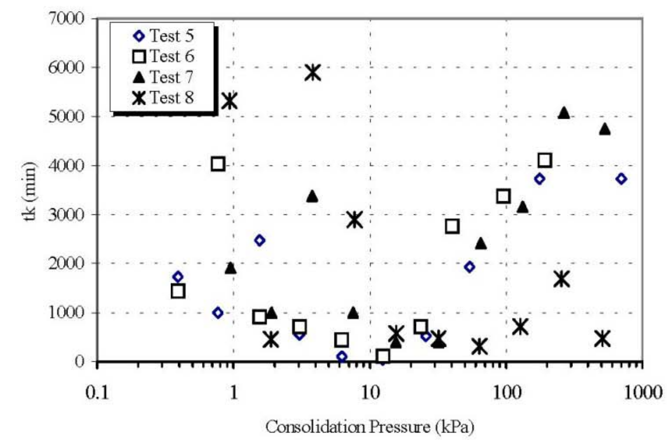


(a)



(b)

Fig. 8. The time to the end of secondary consolidation and beginning of the tertiary compression (tk) versus consolidation pressure for vertically (a) and horizontally (b) oriented samples

Coefficients of secondary and tertiary compression of vertically oriented samples

Figures 9 and 10 present the values of the coefficient of the secondary compression (C_α) and tertiary compression (C_k) versus the consolidation pressure for test no.4, respectively. Beyond a pressure of about 1 kPa, approximately a linear increase exists between the stress and the value of the coefficient of seco··dary compression, (on a log stress axis). Variations of the values of the coefficient of tertiary com··ession exist with the increase of the consolidation stresses. Figures 11 and 12 present the values of the coefficient of the secondary compression (C_α) and tertiary compression (C_k) versus the consolidation pressure in the range of 10 to 100 kPa, respectively. The data in Figure 11 suggests that the values of C_α

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are about (0.15 ± 0.08). The data in figure 12 suggests a decrease in the coefficient of tertiary compression with the increase of the consolidation pressure. Dhowian (1978) describes similar trends for Portage peat. The secondary compression of six consolidation tests resulted with an average coefficient of secondary compression $C_\alpha = 0.15$ (30 data points). The coefficient of tertiary compression reported by Dhowian decreased with the increase of the consolidation pressure from approximately 0.48 for 20 kPa to 0.38 for 60 kPa.

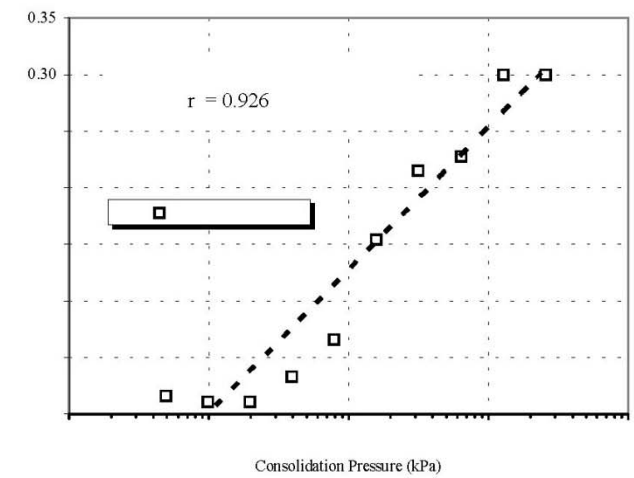

Fig. 9. C_α versus consolidation pressure for test 4

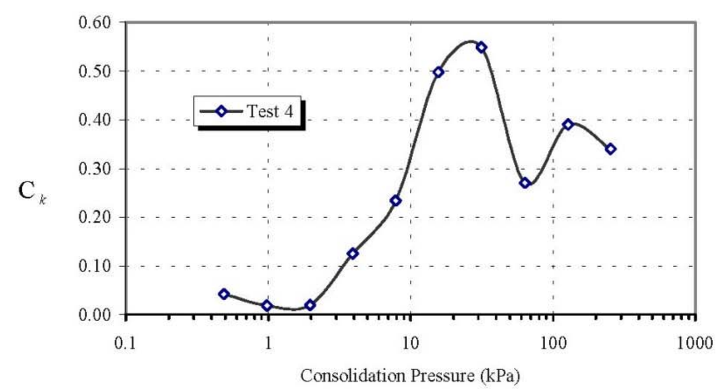

Fig. 10. C_k versus consolidation pressure for test 4

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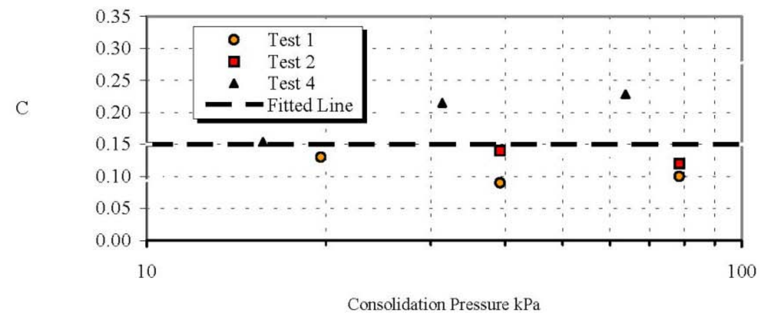

Fig. 11. C_α for the consolidation pressure range of 10 -100 kN/m^2 for the vertically oriented samples

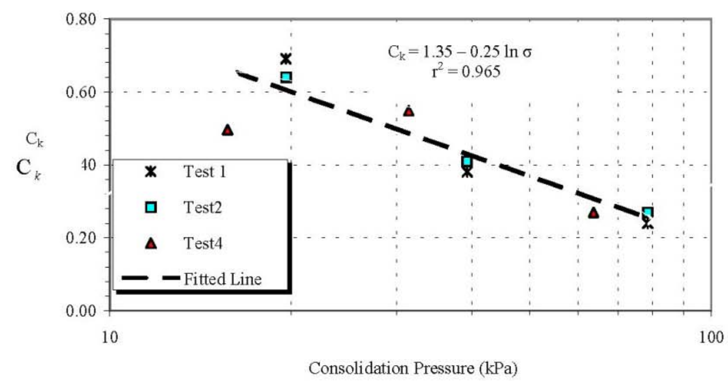

Fig. 12. C_k versus consolidation pressure in the range of 10 -100 kN/m2 for vertically oriented samples

Coefficients of secondary and tertiary compression of horizontally oriented samples

Figures 13 and 14 present the values of the coefficient of secondary compression (C_α) and tertiary compression (C_k) versus the consolidation pressure for the consolidation tests of the horizontally oriented, samples, respectively. The coefficient of secondary compression show an approximate constant value of 0.02 to the pressure of 5.0 kN/m^2 from which a linear increase is observed up to a pressure of about 100 kPa beyond which the data is scattered. The coefficient of secondary compression C_α has the average value of (0.03) within the consolidation pressure range of 0.10 to 10.0 kPa and a range of values between 0.05 to 0.15 for the stress levels of 10 to 100 kPa. These values are about two third of the values observed for the vertically loaded samples under the same pressure range (Fig. 11). Mesri (1973) reported on conflicting relationships that have been proposed regarding the coefficient of secondary compression.

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Newland and Allely (1960) indicate C_α independent of consolidation pressure. Wahls, (1962) indicates C_α decreases with pressure. Ladd and Preston, (1965) indicate C_α increases slightly with consolidation pressure. In this paper, C_α may be assumed to be constant within some levels of stresses, but generally C_α increases with consolidation pressure for both horizontally and vertically oriented samples.
The data in figure 14 suggests a gradual consistent increase in the coefficient of tertiary compression with the increase of the consolidation pressure. This trend is opposite to that observed for the vertically loaded samples (Fig. 12).

As tertiary compression is an accelerated rate of the secondary compression, a ratio between the two may be both feasible and practical. Figure 15 presents this ratio (C_k/C_α) for all the tests. While the horizontally oriented samples show a larger scatter (open symbols) the ratio remains limited in magnitude for most consolidation pressures, resulting in $C_k/C_\alpha = 3.4 \pm 1.8$ (± 1SD, 22 points) for the consolidation pressures between 10 to 100 kPa.

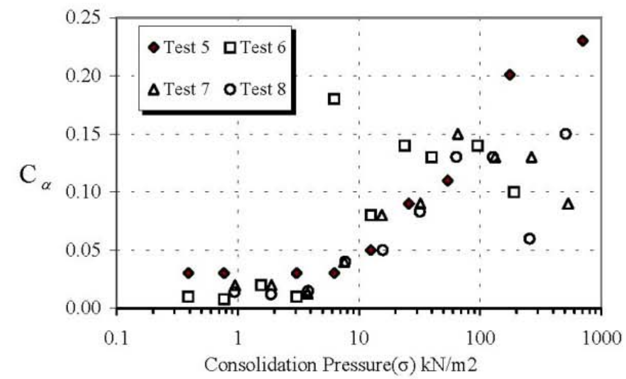

Fig. 13. C_α versus consolidation pressure for horizontally oriented samples

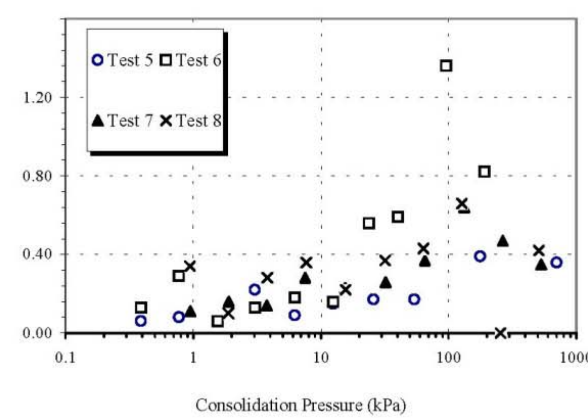

Fig. 14. C_k versus consolidation pressure for horizontally oriented samples

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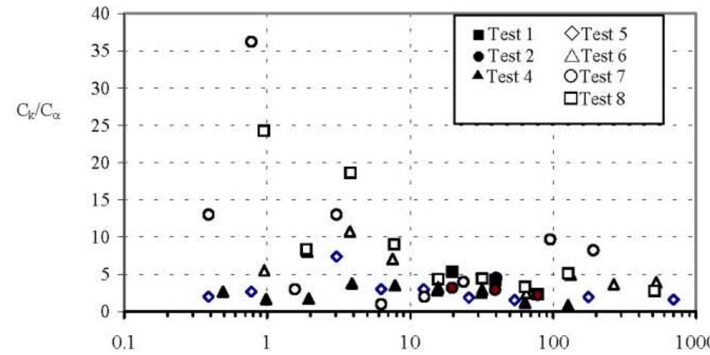

Fig. 15. The ratio between the tertiary and secondary compression indices (C_k/C_α) versus consolidation pressure for vertically and horizontally oriented samples.

3.5 The Relationship between The Primary And The Secondary Compression Indices (C_α/C_c)

Mesri and Godlewski, (1977) suggested that for natural soils, there seems to be a unique relationship between C_α and C_c that holds good at any effective pressure, void ratio, and time during secondary compression. Fox et.al. (1992), reported that the ratio C_α/C_c is not constant because C_α increases with time under constant effective stress. Very often tertiary compression is also seen following secondary compression. Figure 16 shows the variation of the ratio C_α/C_c with the consolidation pressure for test 4. It can be seen that the ratio C_α/C_c ranges from 0.0026 to 0.058 and is not constant. Table 4 summarizes the range of values for the ratio C_α/C_c found in the different tests, referring to all stresses tested and to a range between 10 to 100kPa. When referring to a limited range of stresses (mostly beyond 10 kPa) the ratio of C_α/C_c seem to remain in a relatively small range for all practical proposes. This range did not defer much between the vertically and the horizontally oriented samples.

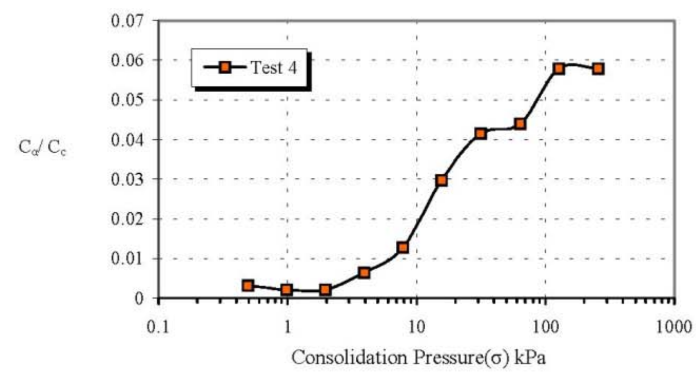

Fig. 16. Values of C_α/C_c versus consolidation pressure

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Table 4. Values of C_α / C_c for the various tests

Test No.	C_α / C_c	C_α / C_c $10 < \sigma < 100$kPa
1	0.026-0.038	0.026-0.038
2	0.028-0.047	0.028-0.047
4	0.0026-0.058	0.030-0.044
5	0.0075-0.086	0.019-0.041
6	0.0020-0.035	0.020-0.035
7	0.0074-0.055	0.029-0.047
8	0.0034-0.037	0.012-0.032

4. CONSOLIDATED UNDRAINED TRIAXIAL TESTS

Isotropically consolidated undrained triaxial compression tests were performed on samples obtained from a depth of 1.80 m. The undisturbed triaxial specimens were approximately 7.0 cm in diameter, and 15.25 cm in height with an aspect ratio of 2.17. The specimens were taken from larger block samples and carefully trimmed to size using a razor knife. The porous stones were fully deaired and saturated with water. The drainage lines were flushed with water to eliminate air bubbles. Full saturation of the samples are essential in order obtain reliable pore pressure readings. The soft peat samples obtained from the field were essentially saturated. However the triaxial specimens enclosed in the membrane were flushed with deaired water under a low hydraulic gradient to remove any trapped air bubbles. The saturated samples yielded B values higher than 0.998.

Deviator stress versus axial strain for triaxial tests performed on vertically oriented peat samples are shown in Figure 17a. Results of excess pore-water pressure versus axial strain are shown in Figure 17b. The tests were performed at four different confining pressures (0.1 psi, 5 psi, 10 psi, and 20 psi). Apparently, the higher the consolidation stress the higher the strength. The following effective stress shear strength parameters were obtained from the Mohr Coulomb failure envelope presented in Figure 18: $\bar{\phi} = 12°$, $\bar{c} = 12$ kN/m^2. These preliminary triaxial test results differ from those reported by Edil and Wang (2000), that suggest higher friction angles and negligible cohesion for normally consolidated peats. Future testing of Carver peat will further address this issue.

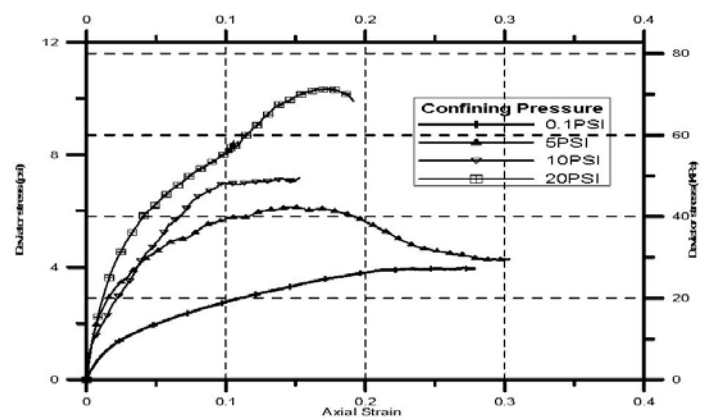

Fig. 17. (a) Axial strain versus deviator stress for the peat samples

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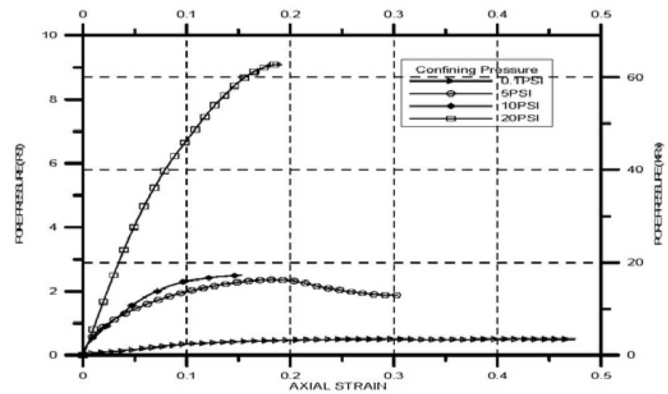

Fig. 17. (b) Excess pore pressure versus axial strain

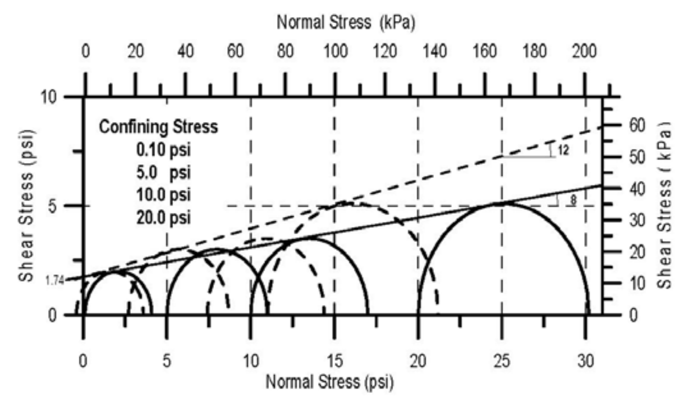

Fig. 18. Shear strength parameters from triaxial tests on vertically oriented peat specimens

5. SUMMARY AND CONCLUSIONS

Peat samples from Carver, Massachusetts, were tested to characterize their engineering properties. Carver peat is fibrous; over consolidated and was tested on vertically and horizontally oriented samples. Conventional long duration oedometer tests showed that primary consolidation was very rapid, (especially for the horizontally oriented samples) and creep effect counted for the majority of the compression. The different long-term behavior curve related to the heterogeneity of the peat, the different fibrous content and the orientation of the loading relative to the orientation of the peat deposition.

The compression index for the vertically oriented samples was between 3.4 to 5.2. These values are within the range observed for other vertically loaded peat types at similar natural water contents. The compression index for the horizontally oriented samples was lower, at the approximate ratio of $C_{ch}/C_{cv} \approx 0.75$.

The time for primary consolidation for horizontally oriented samples is shorter compared to that in the vertically oriented samples. The time of secondary compression is longer in the horizontally oriented samples (while scattered was overall significantly longer) than that in the vertically oriented samples

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under the same consolidation stresses. These observations seem to be explained through the peat structure and fiber orientation such that the permeability increases along the fibers and the compressibility increases normal to the fibers' orientation. Further research is required and will be carried out to examine these observations.

The coefficient of secondary compression (C_α) increases with the consolidation pressure once exceeding a threshold stress level between the overburden pressure and the precompression pressure. A coefficient of secondary compression of $C_\alpha = 0.15$ was found for the consolidation pressure in the range of 10 to 100 kPa. The coefficient of tertiary compression (C_k) decreased with the increase of the consolidation pressure for the vertically oriented samples and increased for the horizontally oriented samples. The trends and absolute values of the vertically oriented samples matched those reported in the literature for other peat types.

The ratio between the primary and secondary compression indices C_α / C_c is not constant as C_α varies with the consolidation pressure. This ratio seems to remain, however, within a relatively limited range of 0.03 $\pm$ 0.01 for stresses between 10 to 100 kPa, regardless of the orientation of the sample. The ratio between the tertiary to the secondary compression indices (C_k/C_α) was found to be within the range of 3.4 ± 1.8 for both; vertically and horizontally oriented samples within a limited zone of consolidation pressure between 10 to 100 kPa.

Isotropically consolidated undrained triaxial compression tests were performed on Carver peat, showing that the peat has apparent cohesion of 12.0 kN/m^2 at 45 % fibers content, undrained angle of friction of 8°, and a drained angle of friction of 12°. These initial tests were performed without backpressure; future planned tests will be performed using backpressure, and the results will be closely compared to those available for other peat types.

ACKNOWLEDGMENT

This project was carried out at the Geotechnical Engineering Research Laboratory of the University of Massachusetts at Lowell and sponsored by Geosciences Testing and Research Inc. (GTR) of N. Chelmsford MA. contracted through Northeast Pile and P.A Landers (Contractors) to the Massachusetts Highway Department, (MHD). The collaboration and support of Mssrs. Nabil Hourani, Peter Connors and Helmut Ernst of the Geotechnical and Construction sections of the MHD is appreciated. Mssrs. Todd Elliot and Terry Edwards of P.A Landers assisted in the sampling and other relevant field activities.

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BIOGRAPHY

Samuel G. Paikowsky holds a B.S. and a MSc. from the Technion, Israel and a Sc.D. in Geotechnical Engineering from MIT. He is a Professor in the Department of Civil and Environmental Engineering at the University of Massachusetts at Lowell and a Principal in Geosciences Testing and Research (GTR) of N. Chelmsford MA. His basic research relevant to granular material behavior includes original mechanical models, dedicated laboratory and field experimental apparatuses and ideal testing systems utilizing photoelasticity, image analysis, and tactile sensor technology. His applicative research and consulting relates to foundations design and construction addressing various issues like time dependent pile capacity, dynamic analyses, static-cyclic testing, multiple deployment model piles, Load and Resistance Factor Design (LRFD) and innovative load-testing systems

Dr. Pradeep U. Kurup is an Associate Professor in the Department of Civil and Environmental Engineering at the University of Massachusetts Lowell. He has vast expertise in advanced experimental techniques and in analytical modeling. He has done extensive research in the areas of site characterization and monitoring, application of novel sensing technologies to geotechnical and geo-environmental engineering, calibration chamber testing, soil-structure interaction, "seeing-ahead techniques" for trenchless technologies, and artificial neural network modeling. He has published his research contributions in several journals and conferences proceedings. Dr. Kurup is an active member in several professional societies, and is also a registered Professional Engineer.

Assem Elsayed holds a BSc. in Civil Engineering from Alexandria University, Egypt in 1992. His engineering experience started in Alexandria and Cairo, Egypt, when he was working for several consultant offices in the field of structural and geotechnical engineering. After 4 years of working in Egypt, he joined Arab Construction Inc. in Saudi Arabia, where he supervised the foundation and structural work of power plants in Dhahran. In fall 2001 he has become a graduate research assistant at the University of Massachusetts Lowell during pursuing his master degree. Assem is the 1992 recipient of excellence award by Alexandria University for the best senior project in Highways and Airports design. He presented "The Engineering Properties of Peat" at the Northeast Geo-technical Graduate Research Symposium, Amherst Massachusetts 2002.

Consolidation Settlement - Long Term Settlement

5. **Secondary Consolidation (Compression) Settlement (cont'd.)**

Example

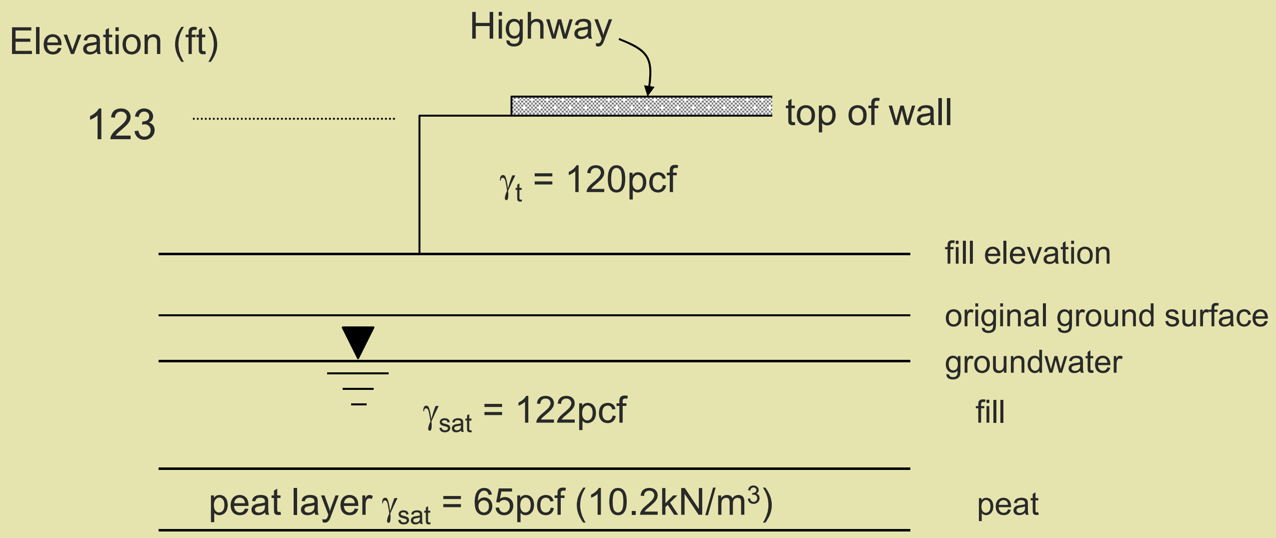
Excavation and replacement of the organic soils was carried out between the sheet piles in Rt. 44 relocation project. Due to various reasons, a monitoring program has detected a remnant peat layer, 4ft thick as shown in the figure. Using the expected loads due to the fill and the MSE (Mechanically Stabilized Earth) Walls, estimate the settlement of the peat:

(a) During primary consolidation, and
(b) During secondary consolidation over a 30 year period.

Consolidation Settlement - Long Term Settlement

5. Secondary Consolidation (Compression) Settlement (cont'd.)

Example (cont'd.)



Peat Parameters:
Based on Table 2 of the paper, $\gamma_{sat} = 10.2 kN/m^3 = 65 pcf$
Based on Tables 3 and 4 for vertically loaded samples,
$e_0 \approx 13 \quad C_c \approx 4.3 \quad C_s \approx 0.68 \quad C_\alpha/C_c \approx 0.036 \rightarrow C_\alpha \approx 0.15$

(see Figure 11)

Consolidation Settlement - Long Term Settlement

5. Secondary Consolidation (Compression) Settlement (cont'd.)

Example (cont'd.)

Assuming a 2-D problem and a peat cross-section before the excavation,

σ'_{vo} = (110-107) × 65 + (107-98)(65-62.4) = 218.4psf

$\Delta\sigma'_v$ = (123-114) × 120 + (114-107) × 120 + (107-100)(122-62.4) + (100-98) × (65-62.4)
= 2342.4psf

$$S_c = \frac{C_c}{1+e_0} log\left(\frac{\sigma'_f}{\sigma'_0}\right) H_0 = \frac{4.3}{1+13} log\left(\frac{2342.4+218.4}{218.4}\right) 4 = (0.307)(1.07)4$$

$$= 1.31ft = 15.75inch$$

$$S_{c(s)} = \frac{C_\alpha}{1+e_0} log\left(\frac{t}{t_p}\right) H_0$$

Consolidation Settlement - Long Term Settlement

5. Secondary Consolidation (Compression) Settlement (cont'd.)

Example (cont'd.)

Evaluation of t_p – end of primary consolidation
From the consolidation test result,
$t_p \approx$ 2min (Figure 7a, and section 3.4.2 of the paper)

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

As C_v and T_v are the same for the sample and the field material:

$$\frac{t_{p\ field}}{t_{p\ lab}} = \frac{{H_{dr\ field}}^2}{{H_{dr\ lab}}^2} = \left(\frac{H_{dr\ field}}{H_{dr\ lab}}\right)^2$$

$H_{dr\ lab}$ = 2.89/2 = 1.45inch $\qquad$ $H_{dr\ field}$ = 2ft = 24 inch $\qquad$ (see table 3)

$t_{p\ field} \cong$ 2min x $(24/1.45)^2$ = 548min $\cong$ 9.1hours

$$S_c = \frac{0.15}{1+13} log\left(\frac{(30)(365)(24)}{9.1}\right)4 = (0.011)(4.46)4 = 0.20ft = 2.3inch$$

Consolidation Settlement - Long Term Settlement

5. **Secondary Consolidation (Compression) Settlement (cont'd.)**

Conclusions:

1. A relatively thin layer of peat, 4ft thick, will undergo a settlement of 18 inches, 38%, of its thickness.

2. Most of the settlement will occur within a very short period of time, theoretically within 9 hours, practically within a few weeks.

3. The secondary settlement, which is significant, will continue over a 30-year period and may become a continuous source of problem for the road maintenance.

Consolidation Settlement - Long Term Settlement

5. **Secondary Consolidation (Compression) Settlement (cont'd.)**

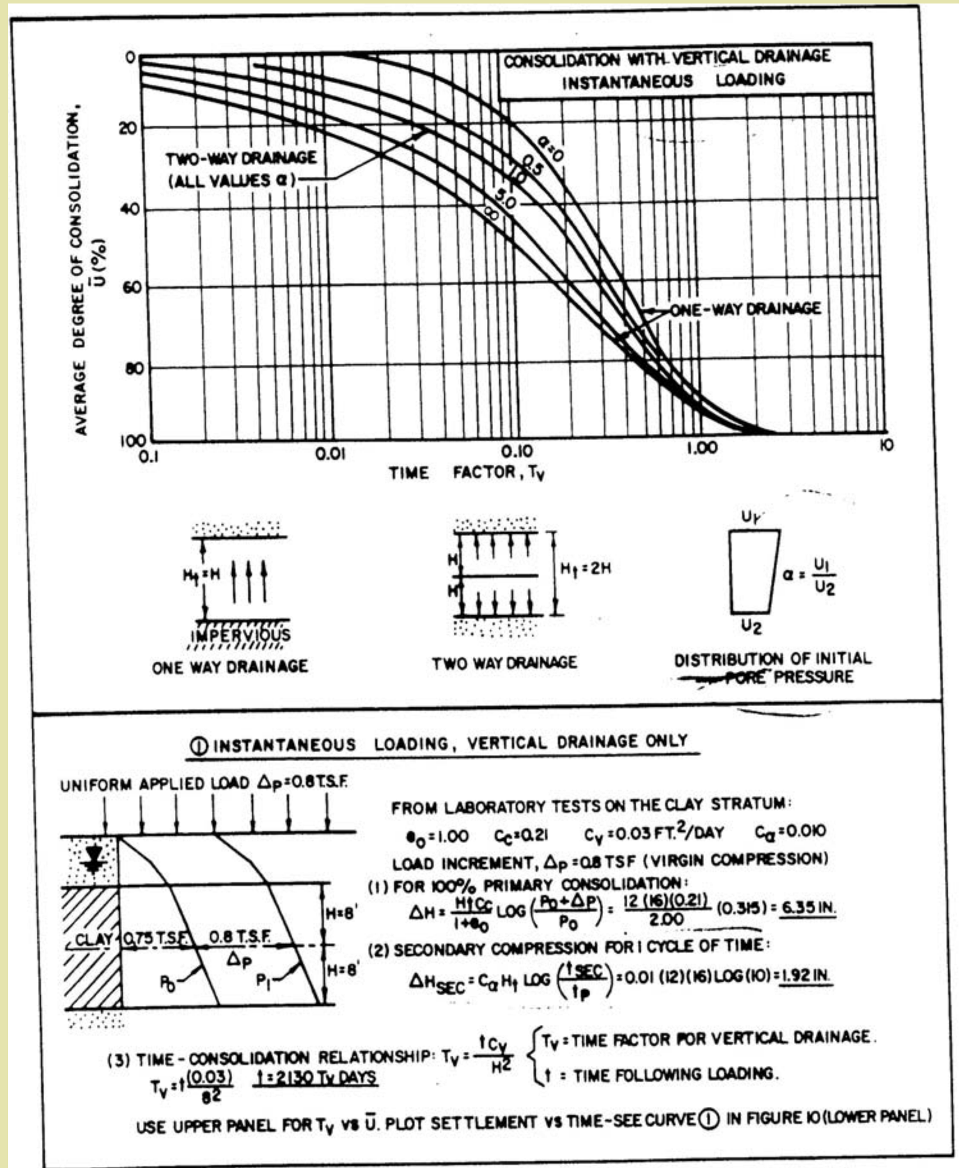


FIGURE 9
Time Rate of Consolidation for Vertical Drainage
Due to Instantaneous Loading

NAVFAC Manual

7.1-227

Consolidation Settlement - Long Term Settlement

5. Secondary Consolidation (Compression) Settlement (cont'd.)

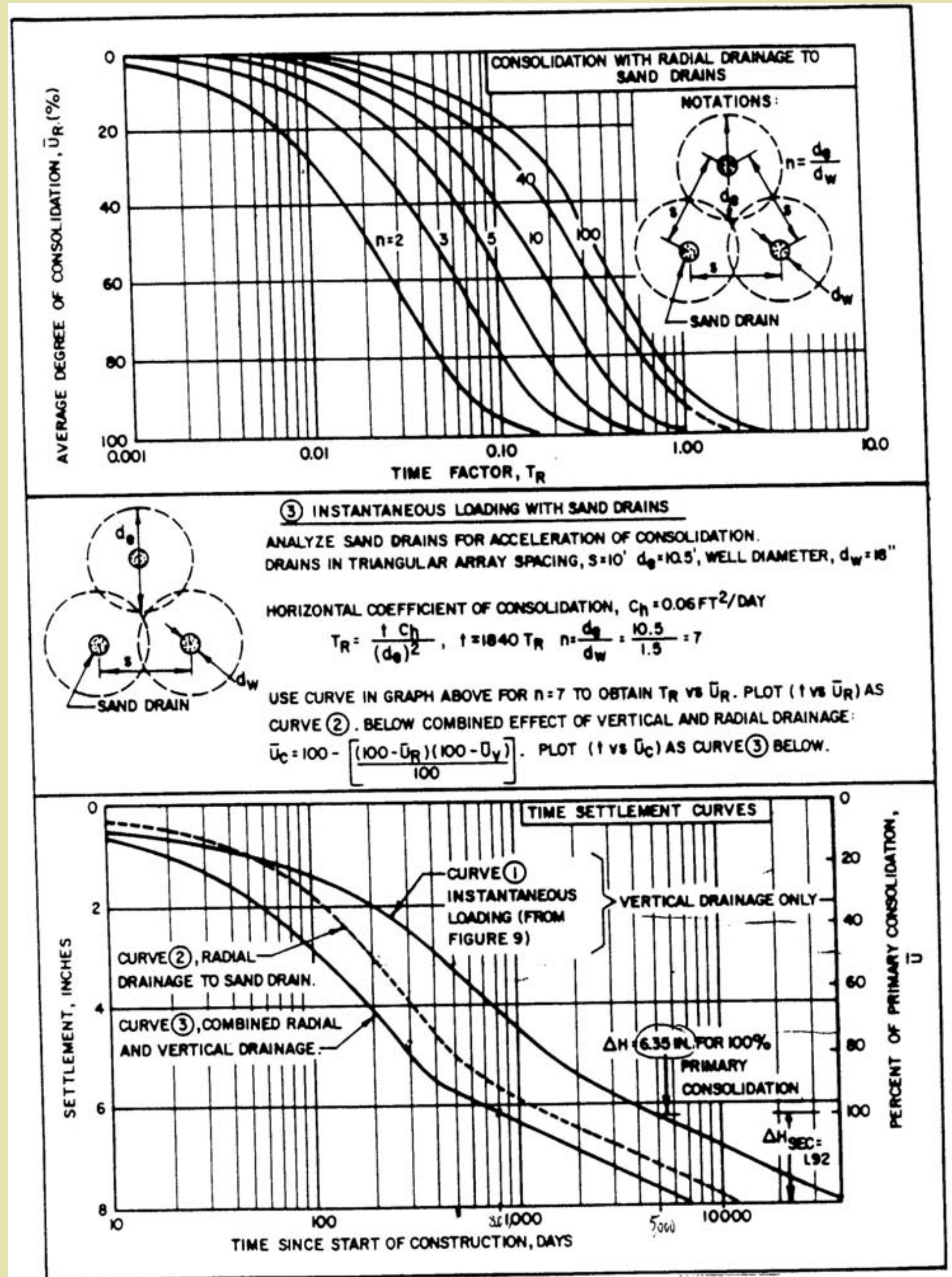


FIGURE 10
Vertical Sand Drains and Settlement Time Rate
7.1-228

Consolidation Settlement - Long Term Settlement

5. Secondary Consolidation (Compression) Settlement (cont'd.)

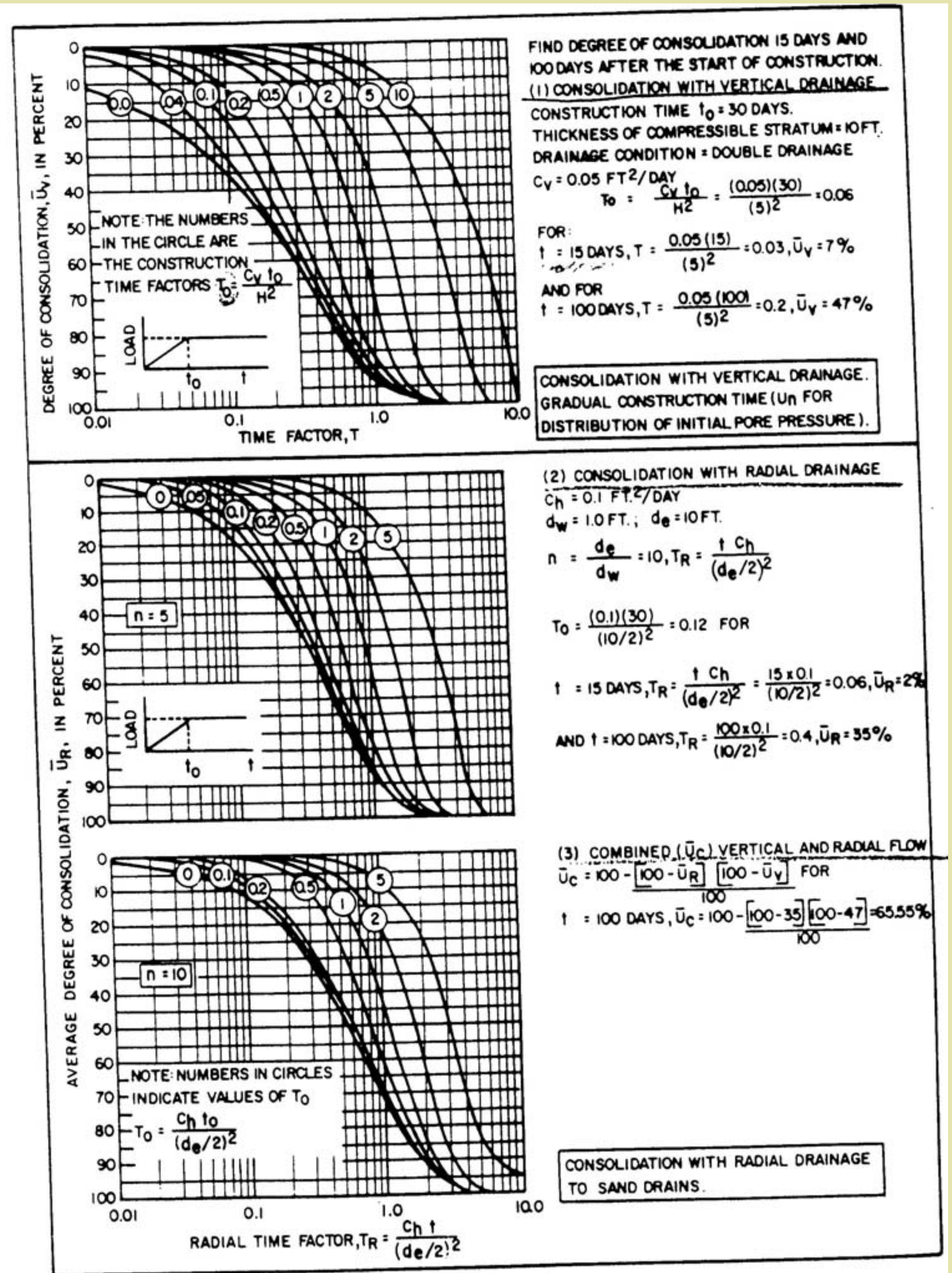


FIGURE 13
Time Rate of Consolidation for Gradual Load Application
7.1-232

Additional Topics

1. **Allowable Bearing Pressure in Sand Based on Settlement Consideration** (Section 5.13, pp. 263-267)

- Using an empirical correlation between N SPT and allowable bearing pressure which is associated with a standard maximum settlement of 1 inch and a maximum differential settlement of ¾ inch.
- Relevant Equations (modified based on the above)

SI Units

$$q_{net} = 19.16 \times N \times F_d \times \left(\frac{S_e}{25.4}\right)$$ **B ≤ 1.22m** (eq. 5.63)

[kPa]

$$q_{net} = 11.98 \times N \times F_d \times \left(\frac{S_e}{25.4}\right) \times \left(\frac{3.28B+1}{3.28B}\right)^2$$ **B > 1.22m** (eq. 5.63)

q_{net} (q_{all}-γD_f) is the allowable stress, N = N corrected

depth factor

$$F_d = 1 + \frac{1}{3} \qquad \frac{Df}{B} \leq 1.33$$

S_e = tolerable settlement in mm

Additional Topics

1. **Allowable Bearing Pressure in Sand Based on Settlement Consideration** (cont'd.)

English Units

- The same equations in English units:

$$q_{net} = \left(\frac{N}{2.5}\right) \times F_d \times S_e \qquad \underline{\mathbf{B \leq 4ft}} \qquad \text{(eq. 5.59)}$$

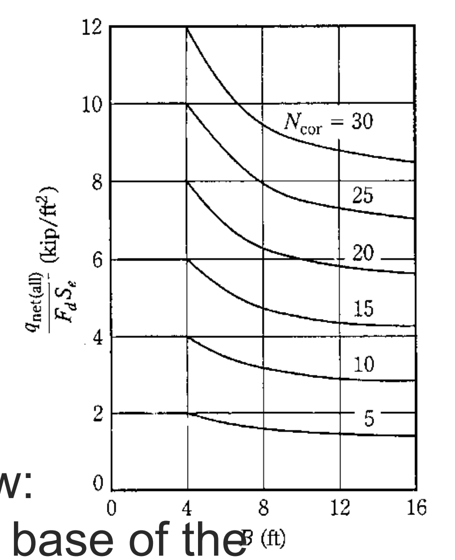
q_{net} [kips/ft^2] S_e [inches]

$$q_{net} = \frac{N}{4}\left(\frac{B+1}{B}\right)^2 \times F_d \times S_e \qquad \underline{\mathbf{B > 4ft}} \qquad \text{(eq. 5.60)}$$

Additional Topics

1. **Allowable Bearing Pressure in Sand Based on Settlement Consideration** (cont'd.)

The following figure is based on equations 5.59 and 5.60: q_{net} over the depth factor vs. foundation width for different $N_{corrected}$ SPT.



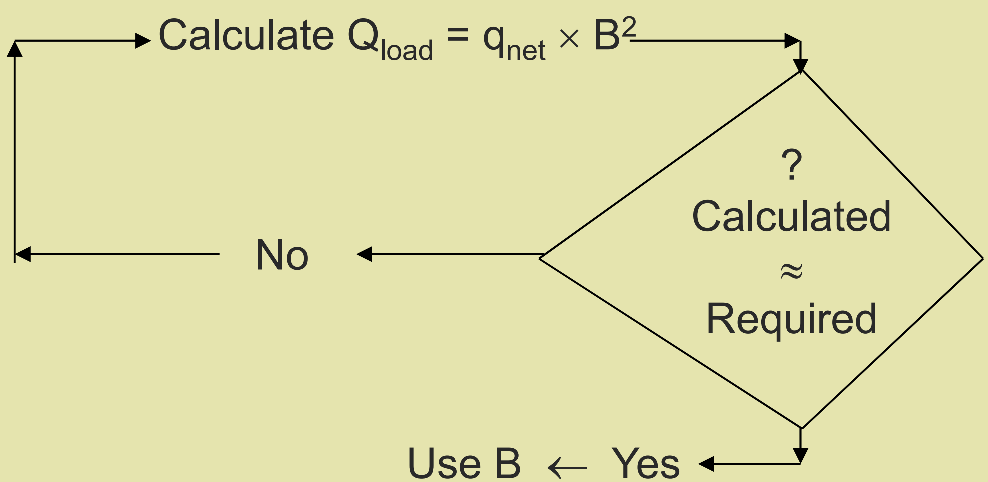
Find B to satisfy a given Q_{load} following the procedure below:
Correct NSPT with depth for approximately 2-3B below the base of the foundation (use approximated B).
Choose a representative $N_{corrected}$ value
Assume B → Calculate F_d → Calculate q_{net} using B&N or find from the above figure $q_{net}/(F_d \times S_e)$
Use iterations:

Additional Topics

1. **Allowable Bearing Pressure in Sand Based on Settlement Consideration** (cont'd.)

Find B to satisfy a given Q_{load} following the procedure below:

1. Correct NSPT with depth for approximately 2-3B below the base of the foundation (use approximated B).
2. Choose a representative $N_{corrected}$ value
3. Assume B $\rightarrow$ Calculate F_d $\rightarrow$ Calculate q_{net} using B&N or find from the above figure $q_{net}/(F_d \times S_e)$
4. Use iterations:



Additional Topics

▼ EXAMPLE 4.9

A shallow square foundation for a column is to be constructed. It must carry a net vertical load of 1000 kN. The foundation soil is sand. The standard penetration numbers obtained from field exploration are given in Figure 4.34. Assume that the depth of the foundation will be 1.5 m and the tolerable settlement is 25.4 mm. Determine the size of the foundation.

Solution The field standard penetration numbers need to be corrected by using the Liao and Whitman relationship (Table 2.4). This is done in the following table:

Depth (m)	Field value of N_F	σ'_v (kN/m²)	Corrected N_{cor}^{a}
2	3	31.4	7
4	7	62.8	9
6	12	94.2	12
8	12	125.6	11
10	16	157.0	13
12	13	188.4	9
14	12	206.4	8
16	14	224.36	9
18	18	242.34	11

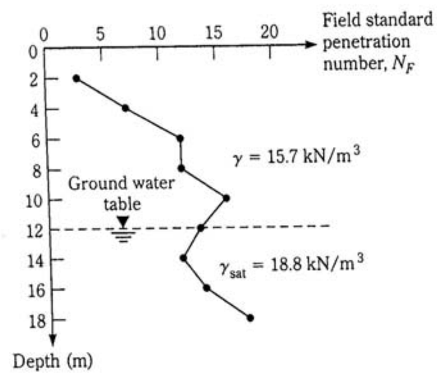
[a] Rounded off

From the table, it appears that a corrected average N_{cor} value of about 10 would be appropriate. Using Eq. (4.53)

$$q_{net(all)} = 11.98 N_{cor} \left(\frac{3.28B + 1}{3.28B}\right)^2 F_d \left(\frac{S_e}{25.4}\right)$$

Allowable S_e = 25.4 mm and N_{cor} = 10, so

$$q_{net(all)} = 119.8 \left(\frac{3.28B + 1}{3.28B}\right)^2 F_d$$



▼ FIGURE 4.34

Additional Topics

for N=10 Se=25.4mm

The following table can now be prepared for trial calculations:

B (m)	F_d[a]	$q_{net(all)}$ (kN/m²)	$Q_o = q_{net(all)} \times B^2$ (kN)
2	1.248	197.24	788.96
2.25	1.22	187.19	947.65
2.3	1.215	185.46	981.1
2.4	1.206	182.29	1050.0
2.5	1.198	179.45	1121.56

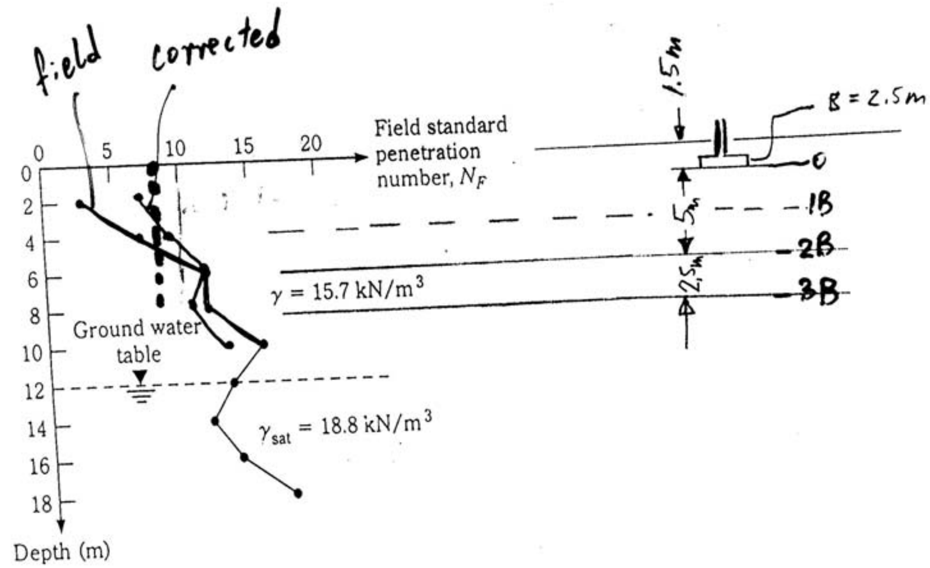
[a] D_f = 1.5 m

790 < 1000 kN
950 < 1000

Because Q_o required is 1000 kN, B will be approximately equal to 2.4 m. ▲

The column load = net load + $\bar{q} \times A$

$$Q_{column} = 1050\ kN + 15.7 \times 1.5 \times 2.4^2 = 1050 + 136 = \underline{1186\ kN}$$



▼ FIGURE 4.34