

***Principles of Foundation Engineering 9th edition Das
Solutions Manual***

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Chapter 2

2.1 From Eq. (2.18),

$$\rho_d = \frac{G_s \rho_w}{1+e} = \frac{2450}{0.925} = \frac{2.80 \times 1000}{1+e}; e = 0.0571$$

From Eq. (2.6),

$$\text{Porosity, } n = \frac{e}{1+e} = \frac{0.0571}{1+0.0571} = \mathbf{0.054}$$

2.2 From Eq. (2.13), the dry density

$$\rho_d = \frac{\rho}{1+w} = \frac{2060}{1+0.153} = 1786.6 \text{ kg/m}^3$$

$$\text{From Eq. (2.18), } \rho_d = \frac{G_s \rho_w}{1+e}$$

$$e = \frac{G_s \rho_w}{\rho_d} - 1 = \frac{2.70 \times 1000}{1786.6} - 1 = 0.511$$

Once saturated, from Eq. (2.19),

$$\rho_{\text{sat}} = \frac{(G_s + e)}{(1+e)} \rho_w = \frac{(2.70 + 0.511)}{(1+0.511)} \times 1000 = \mathbf{2125.1 \text{ kg/m}^3}$$

2.3 Let's consider a 1-m² area in plan. The initial volume of this soil is

$V = 1 \times 0.5 = 0.5 \text{ m}^3$. Volume of the solids is V_s .

$$e = 0.9 = \frac{0.5 - V_s}{V_s}; V_s = 0.2632 \text{ m}^3$$

$$W_s = 0.2632 \times 2.68 \times 9.81 = 6.919 \text{ kN}$$

The new volume after compaction = $1 \times 0.455 = 0.455 \text{ m}^3$

$$\text{The dry unit weight, } \gamma_d = \frac{6.919}{0.455} = 15.21 \text{ kN/m}^3$$

$$\text{From Eq. (2.12), } \gamma_d = \frac{G_s \gamma_w}{1+e}$$

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.68 \times 9.81}{15.21} - 1 = \mathbf{0.729}$$

From Eq. (2.13), $\gamma = (15.21)(1 + 0.20) = \mathbf{18.25 \text{ kN/m}^3}$

2.4 At the compacted road base, the weight of solids,

$$W_s = 120,000 \times 19.5 = 2,340,000 \text{ kN}$$

At the borrow pit, the dry unit weight, $\gamma_d = \frac{\gamma}{1 + w} = \frac{17.5}{1 + 0.085} = 16.13 \text{ kN/m}^3$

$$\text{Volume of the pit, } V = \frac{2,340,000}{16.13} = \mathbf{145,080 \text{ m}^3}$$

The moisture content has to be increased from 8.5% (at the borrow pit) to 14.0% (at the road base). The quantity of water to add,

$$2,340,000 \times (0.14 - 0.085) = 128,700 \text{ kN}$$

$$\text{Volume of water to be added} = \frac{128,700}{9.81} = \mathbf{13,119.3 \text{ m}^3}$$

$$2.5 \quad \gamma_d = \frac{\gamma}{1 + w} = \frac{110.4}{1 + 0.105} = 99.9 \text{ lb/ft}^3$$

$$\gamma_d = \frac{G_s \gamma_w}{1 + e}; e = \frac{2.65 \times 62.4}{99.9} - 1 = 0.655$$

$$\text{From Eq. (2.23), } D_r = \frac{0.870 - 0.655}{0.870 - 0.515} \times 100 = \mathbf{60.6\%}$$

- 2.6 a. A-1-a c. A-3
b. A-1-b d. A-7-6

2.7 Soil A: % of gravel = 50, % of sand = 13, % of fines = 37
 $D_{10} = 0.035 \text{ mm}$, $D_{30} = 0.061 \text{ mm}$, $D_{60} = 9.8 \text{ mm} \rightarrow C_u = 280$; $C_c = 0.02$
 $LL = 58$, $PL = 34$, $PI = 24 \rightarrow$ plots below the A-line; hence, silt
 The soil can be described as **poorly (gap) graded sandy silty gravel with a group symbol of GM.**

Soil B: % of gravel = 24, % of sand = 69, % of fines = 7
 $D_{10} = 0.17$ mm, $D_{30} = 0.82$ mm, $D_{60} = 2.6$ mm $\rightarrow C_u = 15.3$; $C_c = 1.5$
 LL = 42, PL = 22, PI = 20 $\rightarrow$ plots above the A-line; hence, clay
 The soil can be described as **well graded clayey gravelly sand with a group symbol of SW-SM.**

Soil C: % of gravel = 1, % of sand = 99, % of fines = 0
 $D_{10} = 0.7$ mm, $D_{30} = 1.2$ mm, $D_{60} = 1.6$ mm $\rightarrow C_u = 2.3$; $C_c = 1.3$
 The soil can be described as **poorly (uniformly) graded sand with a group symbol of SP.**

Soil D: % of gravel = 0, % of sand = 12, % of fines = 88
 LL = 75, PL = 31, PI = 44 $\rightarrow$ plots above the A-line; hence, clay.
 The soil can be described as **sandy clay of high plasticity with a group symbol of CH.**

- 2.8 The head loss from the reservoir to the ditch, $\Delta h = 38 - 28 = 10.0$ m
 The length of the sand seam in the direction of the flow, $L = 200/\cos 10 = 203.1$ m
 The hydraulic gradient, $i = 10.0/203.1 = 0.0492$
 By Darcy's law [Eq. (2.35)], $v = (2.6 \times 10^{-5} \text{ m/s})(0.0492) = 0.128 \times 10^{-5} \text{ m/s}$
 The cross section of the sand seam through which the flow takes place is
 $1.0 \times 500.0 = 500.0 \text{ m}^2$
 The flow rate = $(0.128 \times 10^{-5} \text{ m/s})(500.0 \text{ m}^2) = 64.0 \times 10^{-5} \text{ m}^3/\text{s}$
 Volume of water flowing into the ditch per day:
 $= (64.0 \times 10^{-5} \text{ m}^3/\text{s})(24)(3600) = \mathbf{55.3 \text{ m}^3}$

- 2.9 For the flow net shown in Figure P2.9, $N_f = 3$ and $N_d = 10$
 Total head loss from right to left, $h_{\max} = 5.0$ m
 The flow rate is given by [Eq. (2.46)]

$$q = kh_{\max} \frac{N_f}{N_d} = (1.5 \times 10^{-5})(5.0) \left(\frac{3}{10} \right) = 2.25 \times 10^{-5} \text{ m}^3/\text{s/m length}$$

$$= (2.25 \times 10^{-5})(50.0)(24)(3600 \text{ m}^3/\text{day}) = \mathbf{97.2 \text{ m}^3/\text{day}}$$

- 2.10 On top of the soft clay layer (i.e., at 10 m depth), initially:

$$\sigma' = 1 \times 17.0 + 9(20 - 9.81) = 108.7 \text{ kN/m}^2$$

After the water table is lowered,

$$\sigma' = 3 \times 17.0 + 7(20 - 9.81) = 122.3 \text{ kN/m}^2$$

By lowering the water table, the effective stress has increased by
 $(122.3 - 108.7) = 13.6 \text{ kN/m}^2$

- 2.11 The soil below the water table can be assumed to be fully saturated (i.e., $S = 1$).

$$e = wG_s = 0.25 \times 2.70 = \mathbf{0.675}$$

The saturated unit weight can be computed as

$$\gamma_{\text{sat}} = \frac{(G_s + e)\gamma_w}{1 + e} = \frac{(2.70 + 0.675) \times 9.81}{1 + 0.675} = \mathbf{19.8 \text{ kN/m}^3}$$

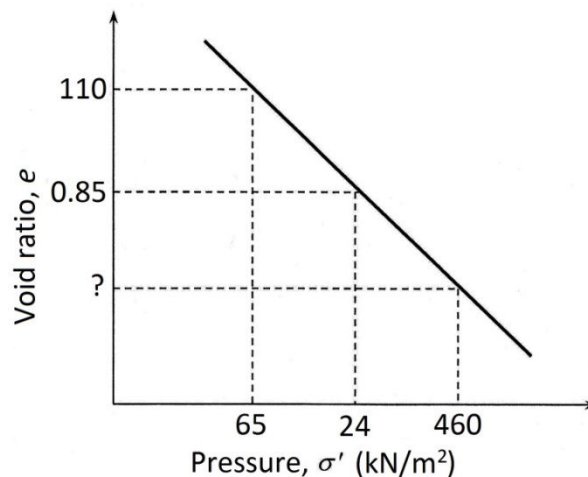
At a depth of 5 m into the sandy clay,

$$\sigma = 4 \times 9.81 + 5 \times 19.8 = \mathbf{138.3 \text{ kN/m}^2}$$

$$u = 9 \times 9.81 = \mathbf{88.3 \text{ kN/m}^2}$$

$$\sigma' = \sigma - u = 138.3 - 88.3 = \mathbf{50 \text{ kN/m}^2}$$

- 2.12 Refer to the figure.



- a. The compression index C_c is given by [Eq. (2.53)],

$$C_c = \frac{e_1 - e_2}{\log \sigma'_2 - \log \sigma'_1} = \frac{1.10 - 0.85}{\log 240 - \log 65} = 0.441$$

- b. Let the void ratio at 460 kN/m² pressure be e_3 .

$$e_1 - e_3 = C_c(\log 460 - \log 65) = 0.441 \times \log\left(\frac{460}{65}\right) = 0.375$$

$$e_3 = 1.10 - 0.375 = \mathbf{0.725}$$

- 2.13 a. The clay is below the water table and, hence, is saturated. The initial void ratio e_o can be determined as

$$e_o = wG_s = 0.225 \times 2.72 = 0.612$$

The saturated unit weight is determined as

$$\gamma_{\text{sat}} = \frac{(G_s + e)\gamma_w}{1 + e} = \frac{(2.72 + 0.612)(9.81)}{1 + 0.612} = 20.3 \text{ kN/m}^3$$

The effective overburden stress at the middle of the clay is

$$\sigma'_o = 2 \times 17.0 + 3(20.2 - 9.81) + 1.5(20.3 - 9.81) = 80.3 \text{ kN/m}^2 < 110.0 \text{ kN/m}^2$$

Since the preconsolidation pressure is greater than the current overburden pressure, the **clay is overconsolidated**. The overconsolidation ratio

$$\text{OCR} = 110.0/80.3 = \mathbf{1.37}$$

- b. The 2-m-high compacted fill imposes a surcharge of $2 \times 20 = 40 \text{ kN/m}^2$ (i.e., $\Delta\sigma' = 40.0 \text{ kN/m}^2$, $\sigma'_o = 80.3 \text{ kN/m}^2$, and $\sigma'_c = 110.0 \text{ kN/m}^2$)

Since $\sigma'_o + \Delta\sigma' > \sigma'_c$, the consolidation settlement can be computed from Eq. (2.69) as

$$\begin{aligned} S_p &= \frac{C_s H}{1 + e_o} \log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right) \\ &= \frac{0.06 \times 3000}{1 + 0.612} \log\left(\frac{110.0}{80.3}\right) + \frac{0.52 \times 3000}{1 + 0.612} \log\left(\frac{80.3 + 40.0}{110}\right) \\ &= \mathbf{52.9 \text{ mm}} \end{aligned}$$

- 2.14 For $U = 75\%$, $T_v = 0.477$ (Table 2.12)

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

From two-way (doubly drained) to one-way (singly drained), H_{dr} is doubled. For the same U and, hence, the same T_v , this would increase the time fourfold. Therefore, it will take **4t years**.

- 2.15 a. The clay layer with one-way drainage has H_c of 6.0 m. After one year,

$$T_v = \frac{c_v t}{H^2} = \frac{0.0014 \times 365 \times 24 \times 3600}{600^2} = 0.123; \text{ settlement, } S_{c(t)} = 160 \text{ mm}$$

From Table 2.12, $U = 39.6\%$

$$U = \frac{S_{c(t)}}{S_{c(\max)}}; S_{c(\max)} = 160/0.396 = 404 \text{ mm}$$

When $t = 2$ years, $T_v = 0.246$.

From Table 2.12, $U = 55.8\%$.

Consolidation settlement during the first two years is $0.558 \times 404 = \mathbf{225.4 \text{ mm}}$

- b. The initial effective overburden stress at the middle of the clay is

$$\sigma'_o = 1.5 \times 17.0 + 0.5(18.5 - 9.81) + 3.0(19.0 - 9.81) = 57.4 \text{ kN/m}^2$$

$$\Delta\sigma' = 3 \times 19 = 57 \text{ kN/m}^2$$

$$S_c = \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}\right)$$

$$404 = \frac{C_c \times 6000}{1 + 0.810} \log\left(\frac{57.4 + 57.0}{57.4}\right); C_c = \mathbf{0.41}$$

- 2.16 a. For the clay layer, assuming $S = 100\%$ below the water table,

$$e_o = 0.45 \times 2.70 = 1.215$$

$$\gamma_{\text{sat}} = \frac{(G_s + e)\gamma_w}{1 + e} = \frac{(2.70 + 1.215) \times 9.81}{1 + 1.215} = 17.3 \text{ kN/m}^3$$

The initial effective overburden stress at the middle of the clay layer is

$$\sigma'_o = 1 \times 16 + 1(19.0 - 9.81) + 1.5(17.3 - 9.81) = 36.4 \text{ kN/m}^2$$

With $\text{OCR} = 1.5$, the preconsolidation pressure $\sigma'_c = 1.5 \times 36.4 = 54.6 \text{ kN/m}^2$

When the fill is placed, it imposes a surcharge of $\Delta\sigma' = 20 \times 1.5 = 30.0 \text{ kN/m}^2$

From Eq. (2.69),

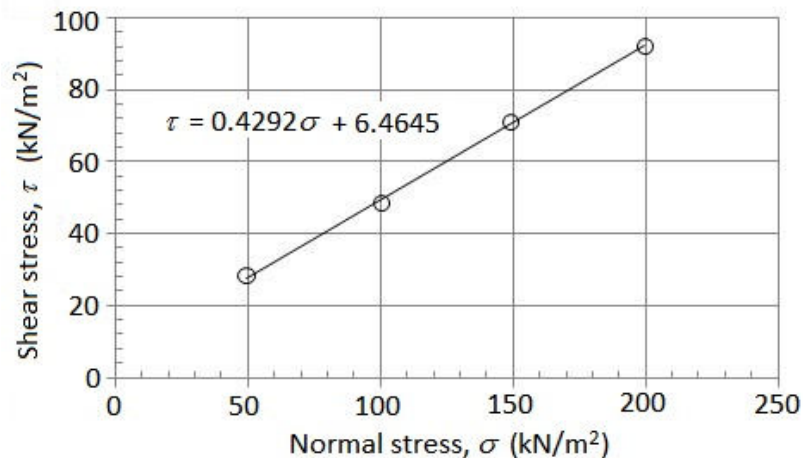
$$\begin{aligned} S_c &= \frac{C_s H}{1 + e_o} \log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right) \\ &= \frac{0.08 \times 3000}{1 + 1.215} \log\left(\frac{54.6}{36.4}\right) + \frac{0.65 \times 3000}{1 + 1.215} \log\left(\frac{66.4}{54.6}\right) = \mathbf{93.9 \text{ mm}} \end{aligned}$$

- b. Including the fill load and the warehouse load, $\Delta\sigma' = 30 + 40 = 70 \text{ kN/m}^2$

$$\begin{aligned}
 S_c &= \frac{C_s H}{1 + e_o} \log\left(\frac{\sigma'_c}{\sigma'_o}\right) + \frac{C_c H}{1 + e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'}{\sigma'_c}\right) \\
 &= \frac{0.08 \times 3000}{1 + 1.215} \log\left(\frac{54.6}{36.4}\right) + \frac{0.65 \times 3000}{1 + 1.215} \log\left(\frac{106.4}{54.6}\right) \\
 &= 274.2 \text{ mm}
 \end{aligned}$$

Consolidation settlement due to the warehouse alone is $274.2 - 93.9 = \mathbf{180.3 \text{ mm}}$

- 2.17 The direct shear test data are plotted in the figure. From the failure envelope, $c' = \mathbf{6.5 \text{ kN/m}^2}$ and $\phi' = \tan^{-1}(0.4292) = \mathbf{23.2^\circ}$



- 2.18 a. $\sigma'_3 = 100 \text{ kN/m}^2$ and $\Delta\sigma_f = 260 \text{ kN/m}^2$

Therefore, $\sigma'_1 = \sigma'_3 + \Delta\sigma_f = 360 \text{ kN/m}^2$

$$\text{From Eq. (2.91), } \sigma'_1 = \sigma'_3 \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan\left(45 + \frac{\phi'}{2}\right)$$

For normally consolidated clays, $c' = 0$; hence,

$$360 = 100 \tan^2\left(45 + \frac{\phi'}{2}\right); \phi' = \mathbf{34.4^\circ}$$

- b. For the second specimen, $\sigma'_3 = 200 \text{ kN/m}^2$

$$\sigma'_1 = 200 \tan^2 \left(45 + \frac{34.4}{2} \right) = 719.5 \text{ kN/m}^2$$

$$\Delta\sigma_f = 719.5 - 200 = \mathbf{519.5 \text{ kN/m}^2}$$

- 2.19 a. In normally consolidated clay, $c' = 0$

For the first specimen (consolidated drained test), using Eq. (2.91),

$$260 + 150 = 410 = 150 \tan^2 \left(45 + \frac{\phi'}{2} \right); \phi' = 27.7^\circ$$

In the second specimen (consolidated undrained test), applying the same value of ϕ' in Eq. (2.91),

$$\sigma_3 = 150 \text{ kN/m}^2 \text{ and } \Delta\sigma_f = 115 \text{ kN/m}^2$$

$$\sigma_1 = \sigma_3 + \Delta\sigma_f = 265 \text{ kN/m}^2$$

$$\sigma'_3 = 150 - u_f \text{ and } \sigma'_1 = 265 - u_f$$

$$\sigma'_1 = \sigma'_3 \tan^2 \left(45 + \frac{\phi'}{2} \right) = \sigma'_3 \tan^2 \left(45 + \frac{27.7}{2} \right) = 2.737 \sigma'_3$$

$$265 - u_f = (150 - u_f) \times 2.737; u_f = \mathbf{83.8 \text{ kN/m}^2}$$

- b. From Eq. (2.96), $A_f = \frac{u_f}{\Delta\sigma_f} = \frac{83.8}{115} = \mathbf{0.73}$

- 2.20 At failure the pore water pressure is u_f , $\sigma_3 = 100 \text{ kN/m}^2$ and $\sigma_1 = 207 \text{ kN/m}^2$.

$$\sigma'_3 = 100 - u_f \text{ and } \sigma'_1 = 207 - u_f$$

Substituting for σ'_3 and σ'_1 in Eq. (2.91),

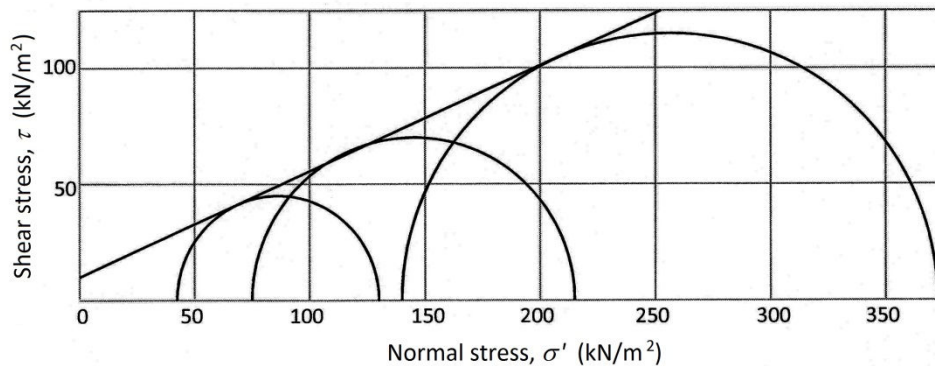
$$\sigma'_1 = \sigma'_3 \tan^2 \left(45 + \frac{\phi'}{2} \right) + 2c' \tan \left(45 + \frac{\phi'}{2} \right)$$

$$(207 - u_f) = (100 - u_f) \tan^2 \left(45 + \frac{26}{2} \right) + 2 \times 10 \tan \left(45 + \frac{26}{2} \right)$$

$$u_f = \mathbf{51.9 \text{ kN/m}^2}$$

- 2.21 The following table can be prepared from the given data, and the Mohr circles are plotted as shown in the figure.

Sample No.	σ_3 (kN/m ²)	$(\Delta\sigma_d)_f$ (kN/m ²)	$(\Delta u_d)_f$ (kN/m ²)	σ_1 (kN/m ²)	σ'_3 (kN/m ²)	σ'_1 (kN/m ²)
1	100	88.2	57.4	188.2	42.6	130.8
2	200	138.5	123.7	338.5	76.3	214.8
3	350	232.1	208.8	582.1	141.2	373.3



The failure envelope is drawn tangent to the Mohr circles in the figure and, from measurements, $c' = 10.0 \text{ kN/m}^2$ and $\phi' = 24.7^\circ$

- 2.22 The unconfined compressive strength of the clay specimen $q_u = 2c_u = 90 \text{ kN/m}^2$

$$\text{Cross sectional area of the specimen} = \frac{\pi}{4} \times 75^2 = 4417.9 \text{ mm}^2$$

$$\text{Maximum load the specimen can carry} = 90 \times 4417.9 \times 10^{-6} = 0.398 \text{ kN} = 398 \text{ N}$$

$$\text{Weight of one steel plate} = 1.5 \times 9.81 \text{ N} = 7.358 \text{ N}$$

$$\text{Therefore, number of plates that can be stacked on the specimen} = 398/7.358 = 54$$

With $q_u = 90 \text{ kN/m}^2$ (see Table 2.14), it is a **medium clay** (consistency).

- 2.23 a. From Figure P2.7, $D_{10} = 0.7 \text{ mm}$, $D_{15} = 0.9 \text{ mm}$, $D_{30} = 1.2 \text{ mm}$,

$$D_{50} = 1.4 \text{ mm}, D_{60} = 1.6 \text{ mm}, \text{ and } D_{85} = 2.05 \text{ mm}$$

$$C_u = \frac{D_{60}}{D_{10}} = \frac{1.6}{0.7} = 2.3; \quad C_c = \frac{D_{30}^2}{D_{10} \times D_{60}} = \frac{1.2^2}{0.7 \times 1.6} = 1.3$$

$$\text{From Eq. (2.87), } \phi' = 26 + (10 \times 0.8) + (0.4 \times 2.3) + 1.6 \log(1.4) = 35.2^\circ$$

b. From Eq. (2.89), $a = 2.101 + 0.097 \left(\frac{D_{85}}{D_{15}} \right) = 2.101 + 0.097 \left(\frac{2.05}{0.9} \right) = 2.322$

From Eq. (2.90), $b = 0.845 - 0.398a = 0.845 - 0.398 \times 2.322 = -0.0792$

From Eq. (2.88),

$$\phi' = \tan^{-1} \left(\frac{1}{ae + b} \right) = \tan^{-1} \left(\frac{1}{2.322 \times 0.61 - 0.0792} \right) = \mathbf{36.8^\circ}$$