**Solution 1.1**

Given: $S = 1.0$, $M_T = 500$g, $M_s = 450$ g

(a) Water Content (w)

$$w = \frac{M_w}{M_s} = \frac{M_T - M_s}{M_s} = \frac{500 - 450}{450} = 0.11 = 11\%$$

(b) Void Ratio (e)

$$e = \frac{G_s w}{S} = 2.7 \; 0.11 = 0.297$$

(c) Saturated Unit Weight ($\gamma_{sat}$)

$$\gamma_{sat} = \left(\frac{G_s + e}{1 + e}\right)\gamma_w = \left(\frac{2.7 + 0.297}{1 + 0.297}\right)9.8 = 22.6 \text{ kN/m}^3$$

(d) Effective Unit Weight ($\gamma'$)

$$\gamma' = \gamma_{sat} - \gamma_w = 22.6 - 9.8 = 12.8 \text{ kN/m}^3$$

**Solution 1.2**

Since the soil is saturated S = 1

Find void ratio

Se = $WG_s$ ; $e = 2.7 \times 0.38 = 1.03$

Find dry unit weight

$$\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.7 \times 9.8}{1+1.03} = 13.0 \text{ kN/m}^3$$

Find bulk unit weight

$\gamma = \gamma_d (1 + w) = 13(1 + 0.38) = 18 \text{ kN/m}^3$

Find porosity

$$n = \frac{e}{1+e} = \frac{1.03}{1+1.03} = 0.51 = 51\%$$

**Solution 1.3**

Given: $w = 0.08$, $\gamma = 16.5\ \text{kN/m}^3$, $G_s = 2.7$, $e_{max} = 0.87$ (loose state), $e_{min} = 0.51$ (dense state)

(a) Relative Density ($D_r$)

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \text{ x } 100$$

Determine e by first calculating $\gamma_d$

$$\gamma_d = \frac{\gamma}{1 + w} = \frac{16.5}{1.08} = 15.3\ \text{kN/m}^3$$

$$\gamma_d = \frac{G_s\ \gamma_w}{1 + e} \Rightarrow e = \frac{G_s\ \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.8}{15.3} - 1;\ e = 0.729$$

Calculate $D_r$:

$$D_r = \frac{0.87 - 0.729}{0.87 - 0.51} \times 100 = 39.2\%$$

(b) Degree of Saturation

$$S = \frac{wG_s}{e} = \frac{0.08 \times 2.7}{0.729} = 29.6\%$$

**Solution 1.4**

(a) CL-ML is mix of low plasticity (also low compressibility) clay and silts.

(b) You need information on how much coarse materials (% sand, % gravel), if any, are present.

(c) The rating for this soil (Table 1.1) is between 5 and 6 for seepage as important to 9 and 10 for seepage as unimportant. This material is not an excellent material for a tank foundation.

(d) Well graded gravel (GW). This material has a rating of 1 in Table 1.1.

**Solution 1.5**

(a) Montmorillonite
(b) Eq. (1.12): PI= 120 -32 = 88%; Eq. (1.13): LI = 0.55
(c) No. It will likely deform in a plastic manner (0 <LI <1).

**Solution 1.6**

(a) Clay

(b) No. Clays are practically impervious.

(c) Major concern: (a) high compressibility( time dependent settlement) (b) Poor workability (c) low shear strength and (d) frost susceptible.

(d) The ASTM standard falling head test measures the vertical permeability. The thin horizontal sandy silt layer will tend to increase horizontal flow, which may not be captured by the standard test. In practice, this soil layer could lead to instability.

**Solution 1.7**

(a) Minimum dry unit weight = 19 x 0.95 = 18 kN/m$^3$. Water content = 11%

(b) Field testing using either a sand cone, a balloon test or nuclear density meter.

(c) The soil should be compacted wet of optimum because it is expansive.
(d)
Calculate the dry unit weight of borrow pit soil

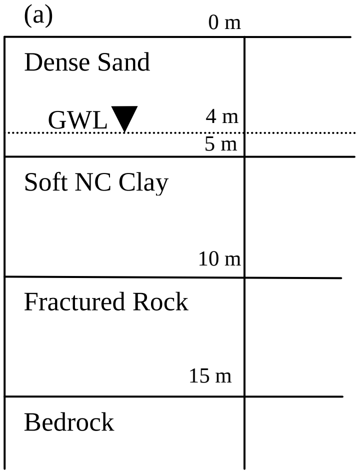
$$\gamma_d = \frac{17}{1+0.082} = 15.7\ kN/m^3$$

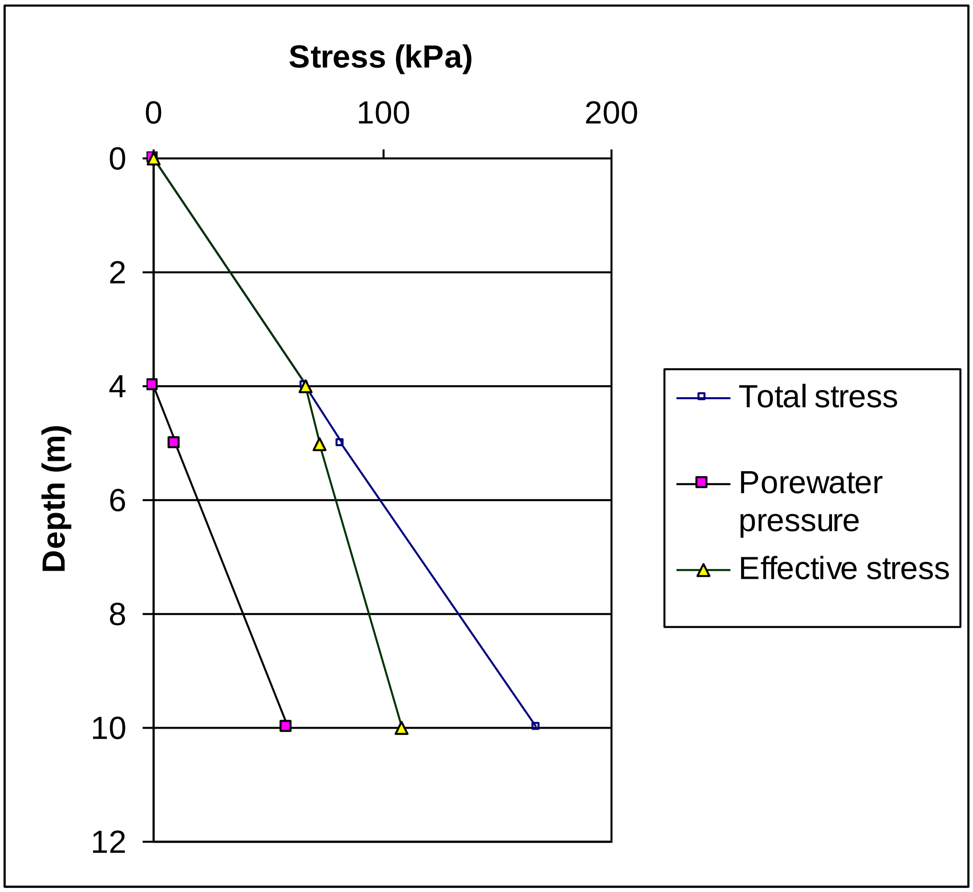
Volume of borrow pit soil required is $\dfrac{\gamma_{d\ \ emb}}{\gamma_{d\ \ pit}} V_{emb} = \dfrac{18}{15.7} \times 100,000 = 114650 m^3$

Number of truck loads = $\dfrac{114,650\ m^3}{10\ m^3} = 11,465$

**Solution 1.8**

(a)




(b)

(c) 90.7 kPa

**Solution 1.9**

i = 0.2

$\gamma_{sat} = \left(\frac{2.7+0.6}{1.6}\right)\gamma_w = 20.2\ kN/m^3$ $\gamma' = 20.2 - 9.8 = 10.4\ kN/m^3$

No Seepage

$\sigma_z' = \gamma' z$ = 10.4 x 3 = 31.2 kPa

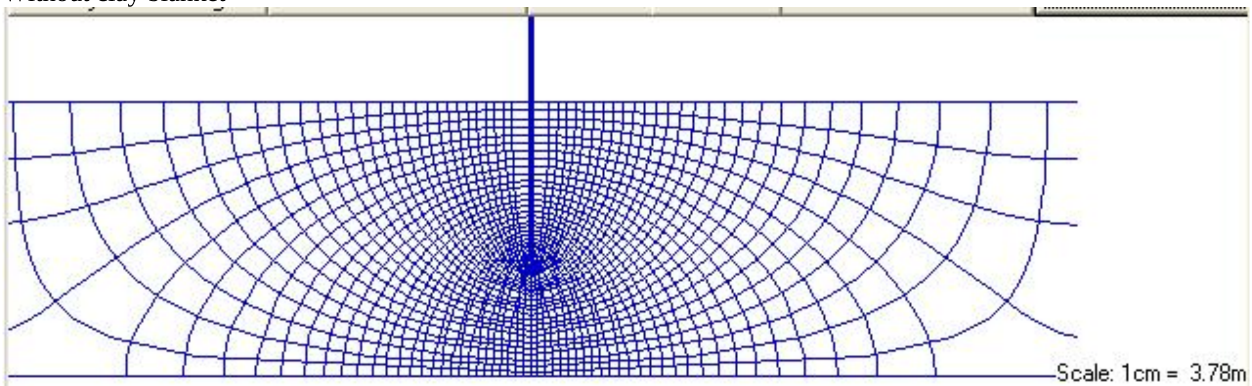
Seepage Upwards

$\sigma_z' = \gamma' z - iz\gamma_w$ = 31.2- 0.2 x 3 x 9.8 = 25.3 kPa

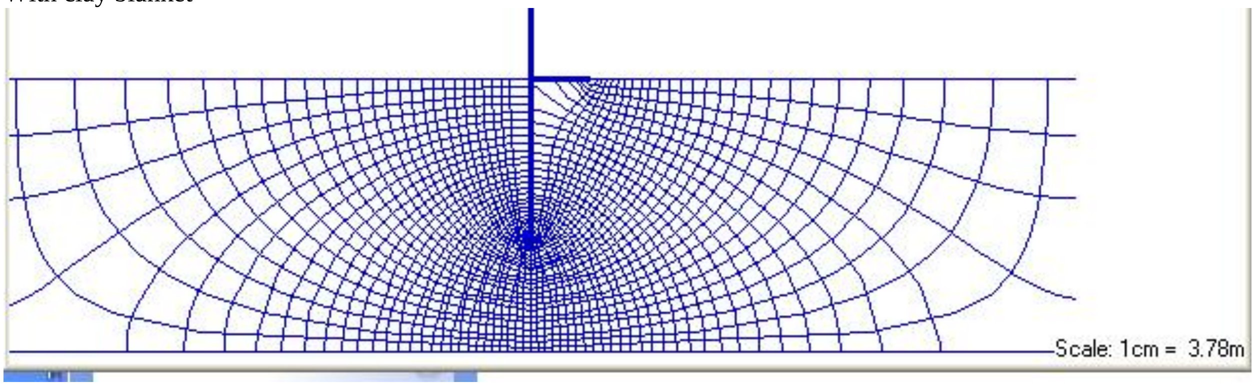
**Solution 1.10**

(a)

Without clay blanket




With clay blanket




(b) Without drainage blanket: $N_f = 28$, $N_d = 60$, $q = k\Delta H \dfrac{N_f}{N_d} = (0.0004/100)\times 6\times\dfrac{28}{60} = 1.12\times10^{-5} m^3/s$

With drainage blanket: $N_f = 27$, $N_d = 60$, $q = k\Delta H \dfrac{N_f}{N_d} = (0.0004/100)\times 6\times\dfrac{27}{60} = 1.08\times10^{-5} m^3/s$

**Solution 1.11**

(a) $e = \frac{wG_s}{S} = 0.6(2.65) = 1.59$

$\gamma_{sat} = \left(\frac{2.65 + 1.59}{2.59}\right)\gamma_w = 16.1 kN/m^3$ $\quad\quad \gamma' = 16.1 - 9.8 = 6.3 kN/m^3$

$\sigma_z = \gamma_{sat} z = 16.1 \times 10 = 161 kPa$ $\quad\quad \sigma_z' = \gamma' z = 6.3 \times 10 = 63 kPa$

$u = \gamma_w z = 9.8 \times 10 = 98 kPa$

(b) $\sigma_z = \gamma_{sat} z = 0 kN/m^2$ $\quad\quad \sigma_z' = \sigma_z - u_{table}$

$63 = 0 - u_{table}; u_{table} = -63 kPa$ $\quad\quad \sigma_z' = 63 kPa$

(c) No. The negative porewater pressure acts like a suction pressure, holding the soil particles together.

(d) The sample would collapse as the effective stress becomes zero. However, air entrainment will cause a negative pressure that may hold some of the particles temporary. If the soil is cemented homogeneously, the cementation will hold the particles together.