THIS EXAMINATION CONSISTS OF **13** PAGES (INCLUDING THIS PAGE).
PLEASE CHECK THAT IT IS COMPLETE.

THE UNIVERSITY OF BRITISH COLUMBIA

Department of Civil Engineering

FINAL EXAMINATION – DECEMBER 2012
SOIL MECHANICS II - CIVL 311
Instructors: Dr. D. Wijewickreme

Time: 3 hours

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1. **Closed Book** Examination; a calculator only is permitted.
2. Please ensure that you write your name and student number on the first page of all answer books.
3. Answer all 4 questions.
4. The formula sheet is attached.
5. Assume the unit weight of water to be 9.8 kN/m^3.
6. Make any reasonable assumptions (where appropriate and if required) to answer the questions.
7. Use sketches whenever possible.
8. Write clearly. Be neat and brief. Marks will be deducted for poor presentation.
9. Show all steps of your calculation to receive full marks.
10. Note the mark value distribution for each question.
11. Please return this exam paper at the end of the exam.
12. POSSIBLE MAXIMUM MARKS FOR THIS QUESTION PAPER = 168

Marks **Question 1**

The soil stratigraphy at a site for a proposed cylindrical fluid storage tank is shown in the schematic diagram below. As may be noted, the site is underlain by a uniform 24-m thick deposit of normally consolidated (NC) clayey silt extending from 12 m to 36 m depth below the ground surface. The results obtained from laboratory 1-D consolidation testing conducted on undisturbed samples of soil retrieved from the clayey silt deposit led to the following relationship between void ratio (e) and vertical effective stress (σ_v'): $e = 1.00 - 0.33 \log(\sigma_v'/100)$ where σ_v' = vertical effective stress in kPa.

It has been suggested that uniform bearing pressure of 120 kPa over a 12-m diameter circular foundation area as shown in the figure be considered for design purposes. It is also given that the vertical stress increase $\Delta\sigma_z$ at a depth z vertically beneath the centre of a circular area of diameter D carrying a uniform pressure q is given by $\Delta\sigma_z = q\, I_q$. The values of Stress Influence Factor (I_q) at several z/D ratios can be computed using elastic theory as per below:

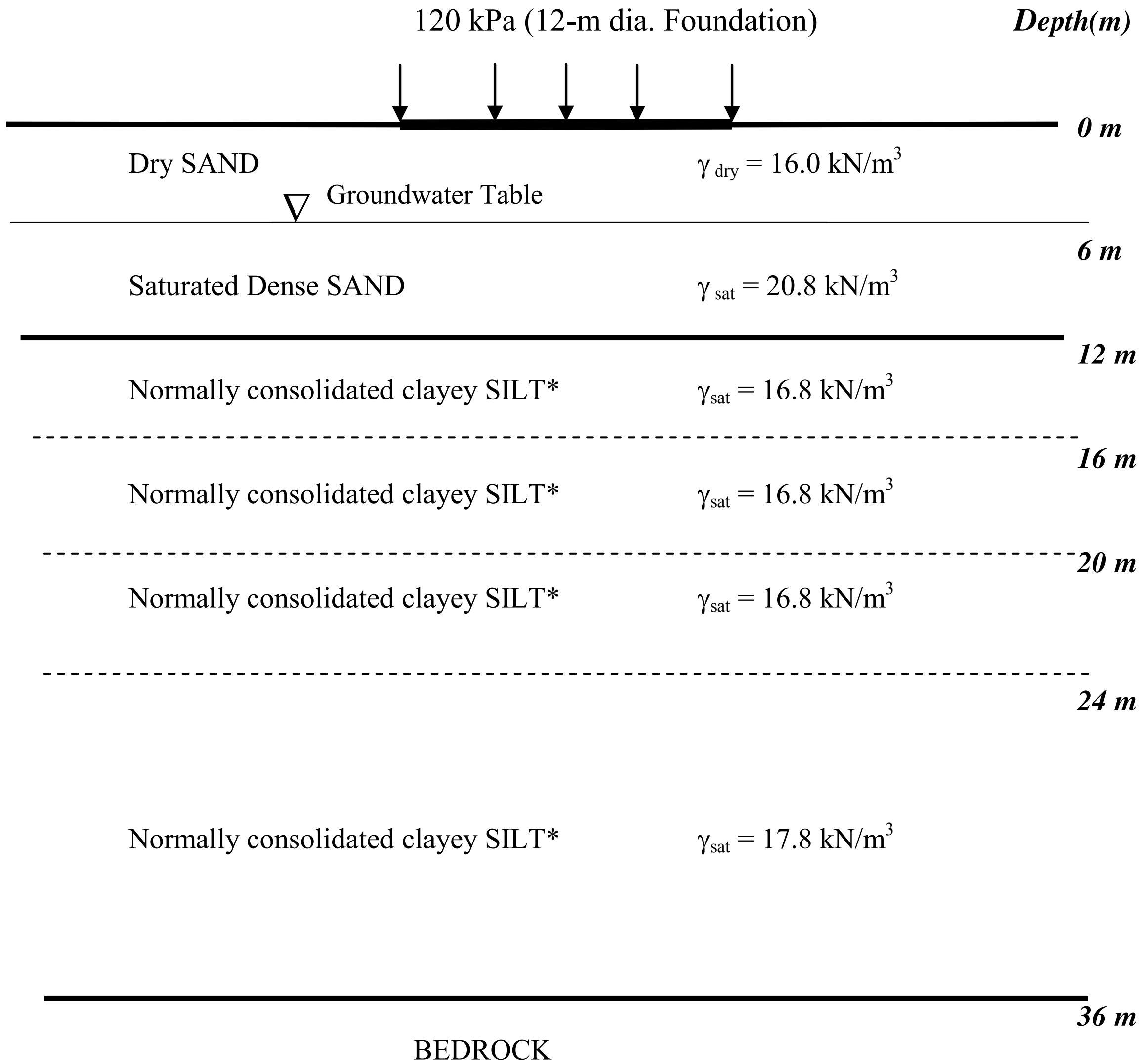
z/D Ratio	Stress Influence Factor (I_q)
1.17	0.24
1.50	0.15
1.83	0.10
2.0	0.00
> 2.0	0.00

(18) (a) Dividing the clayey silt deposit into four sub-layers as shown, and assuming that the stress conditions at the mid-depth of each sub-layer are representative of that sub-layer, estimate the expected ultimate consolidation settlement within the clayey silt deposit due to the applied load from the circular foundation [Note: (i) compute separate initial void ratios (e_o) for mid-depth of each sub-layer before the settlement calculations are undertaken; (ii) ignore settlements within the upper sand zones; (iii) no need to apply Skempton Bjerrum correction];

(10) (b) Estimate the time required for completion of 50% of the total consolidation settlement within the clayey silt deposit due to the placement of the foundation load. Assume that the load is placed is relatively quickly and that the average coefficient of consolidation (C_v) of the clayey silt deposit is 5.0×10^{-3} cm^2/sec;

(7) (c) In an alternate design concept (i.e., a new concept totally replacing the one considered above), the proposed foundation size is to have a diameter of 15 m with an average bearing pressure of 45 kPa. For this alternate design concept, estimate the time required for completion of 90% of the total consolidation settlement within the clayey silt deposit due to the placement of the load. Again assume that the load is placed is relatively quickly and that the C_v of the clayey silt deposit is 5.0×10^{-3} cm^2/sec. No need to compute the settlements for this case.

(Please Note: Question 1(d) is given next page, after the figure)



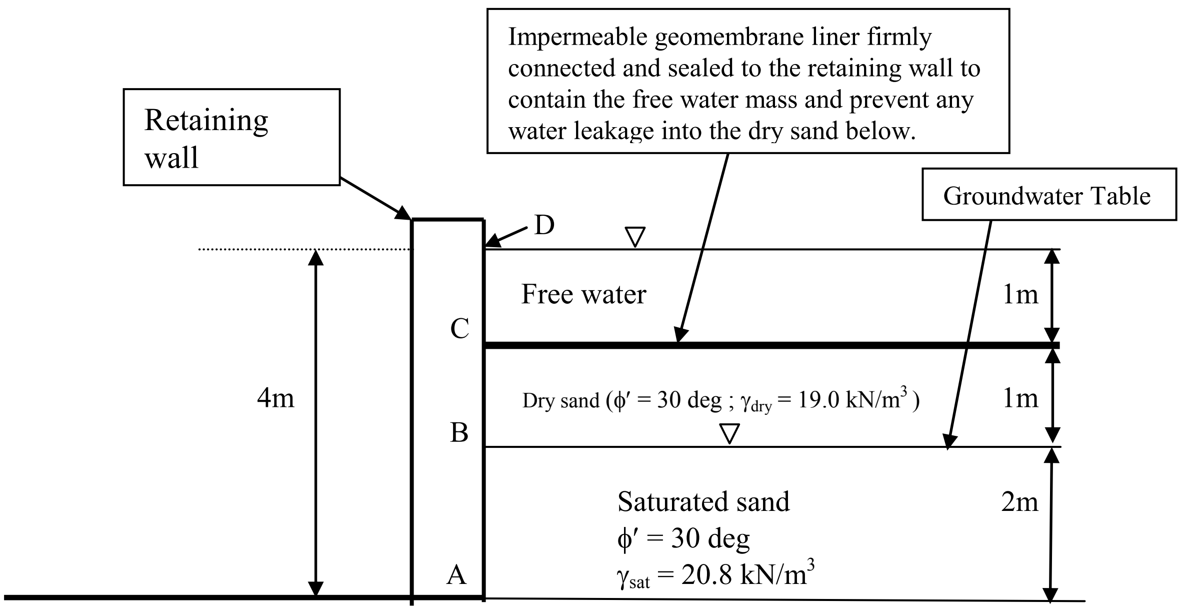
Schematic diagram only. Not to scale.

* For your computation purposes, assume that properties γ_{sat}, C_c, C_v are constant for the whole deposit of normally consolidated clayey silt.

(10) (d) In soils, both compaction and consolidation signify a distinct process of compression. State three main differences between the two processes.

Question 2

(15) (a) The design concept to provide lateral support to a soil mass is presented in the figure below. As shown, the design should account for the presence of a free water mass (1 m depth of water) stored above the sand mass; this water storage is accomplished using a light-weight, impermeable geomembrane liner as shown below. Using the strength and density parameters given, estimate the lateral pressure distribution on the wall along line DCBA (i.e., pressures arising from both the soil and water masses) considering active backfill soil conditions. Assume a smooth wall and static groundwater conditions, and use Rankine theory for the computations.



NOTE: Not to scale; schematic diagram only.

(12) (b) Considering the lateral wall movements in a rigid retaining wall (assuming a sand backfill and smooth wall) and using a graph, qualitatively illustrate how the coefficient of lateral effective earth pressure would vary with wall movement:

(i) when changing from an "at rest" condition to active failure condition; and
(ii) when changing from an "at rest" condition to passive failure condition.

(12) (c) Using Mohr circle diagrams, illustrate the stress state that would prevail in an element of soil experiencing: (i) active failure; and (ii) passive failure conditions. Assume a sand backfill and rigid retaining wall with a smooth wall face.

(6) (d) State three modes of failure that need to be considered in the design of a gravity retaining wall.

Question 3

(18) (a) Given the following information, compute the ultimate net bearing capacity of a 1.5-m wide strip footing founded on a sand deposit.

(i) Peak friction angle: $\phi'_p = 32°$;
(ii) Depth of footing base below ground surface = 0.75 m;
(iii) Groundwater table is located 2.0 m below ground surface;
(iv) Total unit weight of sand above groundwater level = 18.8 kN/m^3
(v) Total unit weight of sand below groundwater level γ_{sat}= 19.8 kN/m^3
(vi) Assume centric loading.

(18) (b) A rectangular footing, 1 m x 2 m in plan dimensions, is to be founded on a clay deposit. The depth of footing embedment is 0.50 m, and the soil has a total (saturated) unit weight of 17 kN/m^3. Based on data from a geotechnical laboratory testing program, it has been determined that the soil has an average unconfined compressive strength of 80 kPa. Determine the ultimate net bearing capacity of the foundation. Assume centric loading.

Question 4

(30) (a) A closed-ended steel pipe pile (i.e., closed-ended cylindrical steel pile) having a diameter of 0.3 m is driven to a depth of 12 m into a normally consolidated clay deposit. Given the following information, estimate the allowable short-term pile load capacity assuming a factor of safety of 3.0. Ignore weight of pile.

(i) Average soil parameters for the clay deposit for different depth zones below the ground surface should be assumed to vary as follows:

a. 0 m ≤ D ≤ 4 m; S_u (remoulded) = 20 kPa; γ_{sat}= 17.8 kN/m^3;
b. 4 m ≤ D ≤ 8 m; S_u (remoulded) = 30 kPa; γ_{sat}= 18.8 kN/m^3;
c. 8 m ≤ D ≤ 12 m; S_u (remoulded) = 40 kPa; γ_{sat}= 19.8 kN/m^3;
d. 12 m ≤ D; S_u (remoulded) = 40 kPa; γ_{sat}= 19.8 kN/m^3
(Note: D = Depth below ground surface).

(ii) Groundwater table is located at the ground surface; and

(iii) Assume that shaft frictional stress (f_s) computed considering vertical effective stress (σ'_{z0}) at the mid-point of a given clay depth zone can be considered applicable for the length of pile segment located within that depth zone.

(3) (b) What is the typical pile displacement required to mobilize full skin friction in a pile?
(3) (c) What is the typical pile displacement required to mobilize full end bearing in a pile?
(3) (d) What density changes would typically occur (around the pile) in a loose sand deposit due to pile driving?
(3) (e) What density changes would typically occur (around the pile) in a dense sand deposit due to pile driving?

EQUATION SHEET

Computation of static effective stresses in a soil mass

$\sigma'_z = \Sigma \gamma_i h_i$; $\sigma' = \sigma - u$;
γ_i = Bulk unit weight of i th layer of soil;
h_i = Thickness of i th layer of soil;
σ'_z = Vertical effective stress;
$\sigma' = \sigma - u$; u = Pore water pressure

Under static groundwater (no flow) conditions: $u = \gamma_w z$

Under seepage flow conditions:
Total Head = Pore water pressure Head + Elevation Head (neglecting velocity head)
$h_{total} = h_{pwp} + h_{elev}$
$h_{total} = (u/\gamma_w) + z_{above\text{-}datum}$
Hydraulic gradient between A and B = $i_{AB} = [(h_{total})_{at\ point\ A} - (h_{total})_{at\ point\ B}]/ L_{AB}$
Where L_{AB} = Distance between points A and B

Consolidation Settlements

For a given layer,
Consolidation settlement = $\rho_{pc} = \varepsilon_z \cdot H_0$ where
$\varepsilon_z = [\Delta e / (1+e_o)]$;
H_0 = Initial thickness of the layer considered for calculating consolidation settlements;
e_o = initial void ratio;
Δe = change in void ratio;

Based on this, for normally consolidated soil,

$$\rho_{pc} = H_o \frac{\Delta e}{1+e_o} = \frac{H_o}{1+e_o} C_c \log \frac{\sigma'_{\text{fin}}}{\sigma'_{\text{zo}}}; \quad \text{OCR} = 1 \qquad (6.14)$$

$$\sigma'_{\text{fin}} = \sigma'_{zo} + \Delta\sigma_z$$

σ'_{zo} = initial vertical effective stress; $\Delta\sigma_z$ = vertical stress increase
σ'_{fin} = final vertical effective stress
C_c = Compression index = $(e_2-e_1)/[\log(\sigma'_{z2} / \sigma'_{z1})]$ along normal consolidation line;

Based on this, for overconsolidated soil,

$$\rho_{pc} = \frac{H_o}{1+e_o} C_r \log \frac{\sigma'_{\text{fin}}}{\sigma'_{\text{zo}}}; \quad \sigma'_{\text{fin}} < \sigma'_{\text{zc}} \qquad (6.15)$$

$$\rho_{pc} = \frac{H_o}{1+e_o} \left(C_r \log \frac{\sigma'_{\text{zc}}}{\sigma'_{\text{zo}}} + C_c \log \frac{\sigma'_{\text{fin}}}{\sigma'_{\text{zc}}} \right); \quad \sigma'_{\text{fin}} > \sigma'_{\text{zc}} \qquad (6.16)$$

C_r = Recompression index = $(e_2-e_1)/[\log(\sigma'_{z2} / \sigma'_{z1})]$ along recompression line;

Rate of Consolidation

T_v = Time Factor
H_{dr} = Drainage Path Length
t_{90}= Time for 90% degree of consolidation
C_v = Coefficient of Consolidation

$$T_\upsilon = \frac{C_\upsilon t_{90}}{H_{dr}^{\ 2}} = 0.848$$

$$T_\upsilon = \frac{\pi U^2}{4} \ for U < 0.6$$

Note: Degree of consolidation (U) is expressed as a decimal value in this equation.

Lateral Earth Pressures

(i) Effective stress analysis

Coefficient of active earth pressure = K_a
Effective lateral earth pressure under active conditions = $(\sigma'_x)_a$
Effective vertical stress at a given depth = σ'_z

$(\sigma_x')_a = K_a \sigma_z'$

$$\frac{(\sigma'_x)_a}{(\sigma'_z)} = \frac{1-\sin\phi'}{1+\sin\phi'} = K_a$$

Coefficient of passive earth pressure = K_p
Effective lateral earth pressure under passive conditions = $(\sigma'_x)_p$
Effective vertical stress at a given depth = σ'_z

$(\sigma_x')_p = K_p \sigma_z'$

$$\frac{(\sigma'_x)_p}{(\sigma'_z)} = \frac{1+\sin\phi'}{1-\sin\phi'} = K_p$$

(ii) Total stress analysis:

Active earth pressure = $(\sigma_x)_a$
Total vertical stress at a given depth = σ_z

$$(\sigma_x)_a = (\sigma_z) - 2S_u$$

Note: At tension crack depth level, Z_{crack} below the ground surface, $(\sigma_x)_a = 0$.

Passive earth pressure = $(\sigma_x)_p$
Total vertical stress at a given depth = σ_z

$$(\sigma_x)_p = (\sigma_z) + 2S_u$$

Bearing capacity of Shallow Foundations

Effective Stress Analysis (ESA)

$$q_u = \gamma D_f (N_q - 1)(s_q d_q w_q) + 0.5\gamma B' N_\gamma (s_\gamma d_\gamma w_\gamma)$$

Total Stress Analysis (TSA)

$$q_u = 5.14 s_u (s_c d_c)$$

Definition of the factors to account for the different variations:

s_q, s_γ, s_c - Shape of Footing

d_q, d_γ, d_c - Depth of Footing Embedment

w_q, w_γ - Groundwater level

q_u - Net bearing capacity; S_u - Undrained shear strength

$$N_q = e^{\pi \tan\phi'_p} \tan^2\left(45^\circ + \frac{\phi'_p}{2}\right); \ \phi'_p \text{ in degrees}$$

Caquot and Kerisel (1953)[2]: $N_\gamma = 2(N_q + 1)\tan\phi'_p$; ϕ'_p in degrees

Meyerhof (1976): $N_\gamma = (N_q - 1)\tan(1.4\phi'_p)$; ϕ'_p in degrees

Davis and Booker (1971)[3] :

smooth foundation, $N_\gamma = 0.0663\exp(9.3\phi'_p)$; ϕ'_p in radians

rough foundation, $N_\gamma = 0.1054\exp(9.6\phi'_p)$; ϕ'_p in radians

Ueno et al. (1998)[4] :

rough foundation, $N_\gamma = 0.477\exp(6.52\phi'_p)$; ϕ'_p in radians

Note: Unless specifically stated otherwise, use Davis and Booker (1971) for rough foundations given above to determine N_γ.

Effective Footing Dimensions

$$B' = B - 2e_B$$

$$L' = L - 2e_L$$

Definition of the factors to account for the different variations:

s_q, s_γ, s_c - Shape of Footing

d_q, d_γ, d_c - Depth of Footing Embedment

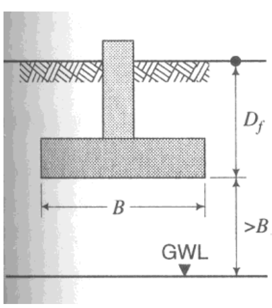
w_q, w_γ - Groundwater level

$s_q = 1 + [(B'/L')*\tan \phi'_p]$; $s_\gamma = 1 - 0.4*(B'/L')$; $s_c = 1 + 0.2*(B'/L')$

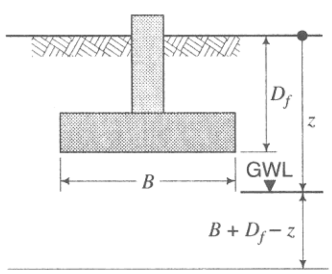
$d_q = 1 + [2*\tan \phi'_p]*(1-\sin \phi'_p)^2 * \tan^{-1}(D_f/B')$ for $(D_f/B') > 1$
$d_q = 1 + [2*\tan \phi'_p]*(1-\sin \phi'_p)^2 * (D_f/B')$ for $(D_f/B') \le 1$

$d_\gamma = 1$

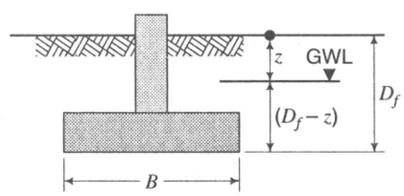
$d_c = 1 + [0.33* \tan^{-1}(D_f/B')]$ for $(D_f/B') > 1$
$d_c = 1 + [0.33* (D_f/B')]$ for $(D_f/B') \le 1$



Case 1: $Z > (B+D_f)$



Case 2: $(B+D_f) > Z > D_f$



Case 3: $Z < D_f$

$$Case\_1: w_q = 1; w_\gamma = 1$$

$$Case\_2: w_q = 1; w_\gamma = \frac{(z - D_f)}{B} + \frac{\gamma'}{\gamma_{sat}}(1 + \frac{D_f}{B} - \frac{z}{B})$$

$$Case\_3: w_q = \frac{z}{D_f} + \frac{\gamma'}{\gamma_{sat}}(1 - \frac{z}{D_f}); w_\gamma = \frac{\gamma'}{\gamma_{sat}}$$

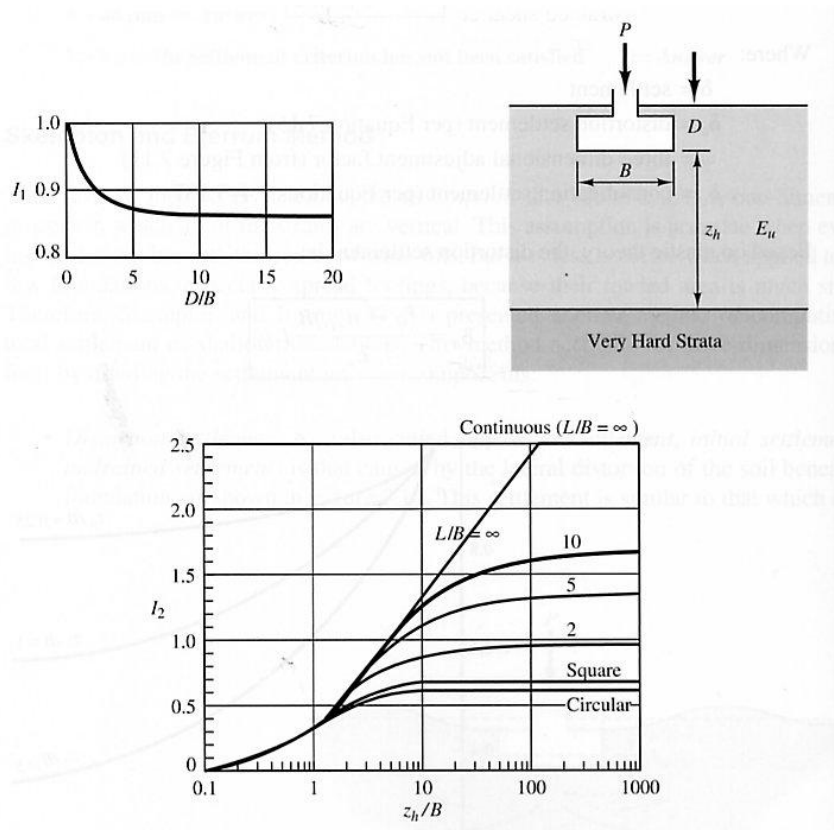
Estimation of settlements of foundations on clay

(a) Distortion Settlements in clay

$$\rho_e = \frac{(q - \sigma'_{zD})B}{E_u} I_1 I_2$$

Where

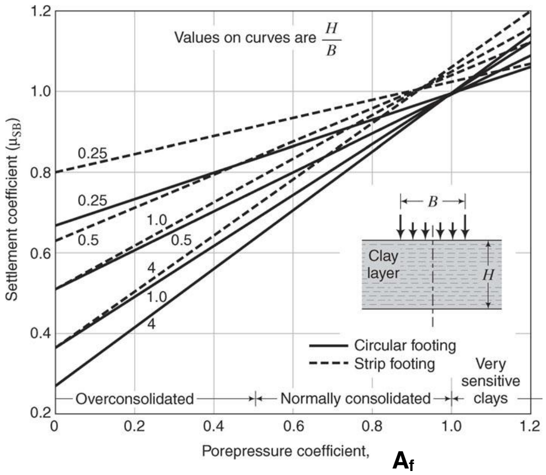
ρ_e = distortion settlement
q = bearing pressure
σ_{zD}' = vertical effective stress at a depth D below the ground surface
B = foundation width
I_1, I_2 = influence factors (per Figure 7.12)
E_u = undrained modulus of elasticity of soil



(b) Consolidation Settlements in clay

Use formulae given in Page 1 to compute 1-D consolidation settlements.

Skempton Bjerrum correction factor.



$$\rho_{pc,3D} = \mu_{SB}\rho_{pc,1D}$$

(c) Secondary compression settlements

$$C_a = -\frac{(e_t - e_p)}{\log(t/t_p)}; t > t_p \qquad \rho_{sc} = \frac{H_0}{(1+e_p)} C_\alpha \log(t/t_p)$$

<u>Estimation of settlements of foundations on sand</u>

$$\boxed{C_1 = 1 - 0.5\left(\frac{\sigma'_{zD}}{q - \sigma'_{zD}}\right)}$$

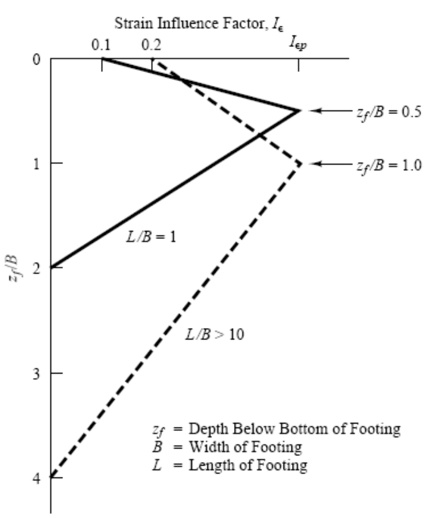
$$\boxed{C_2 = 1 + 0.2\log\left(\frac{t}{0.1}\right)}$$

$$\boxed{C_3 = 1.03 - 0.03\,L/B \geq 0.73}$$

$$\rho = C_1 C_2 C_3 (q - \sigma'_{zD}) \Sigma \frac{I_\varepsilon H}{E}$$

Where:

ρ = settlement of footing
C_1 = depth factor
C_2 = secondary creep factor (see discussion in Section 7.8)
C_3 = shape factor = 1 for square and circular foundations
q = bearing pressure
σ_{zD}' = effective vertical stress at a depth D below the ground surface
I_ϵ = influence factor at midpoint of soil layer
H = thickness of soil layer
E_s = equivalent modulus of elasticity in soil layer
t = time since application of load (yr) ($t \geq 0.1$ yr)
B = foundation width
L = foundation length



Peak value of $I_\varepsilon = I_{\varepsilon p} = 0.5 + 0.1\sqrt{[(q-\sigma'_{zd})/\sigma'_{zp}]}$

σ'_{zp} = value of vertical effective stress at $I_{\varepsilon p}$
σ'_{zd} = vertical effective stress at foundation level; q = bearing pressure

Load Capacity of Pile foundations

(a) Axial Pile Capacity - Static Pile Analysis

$$Q_{ult} = Q_f + Q_b - W_p \qquad Q_a = \frac{Q_{ult}}{FS}$$

$Q_b >> Q_f$ End Bearing Pile; $Q_f >> Q_b$ Friction Pile

Q_b = Ultimate End Bearing Resistance; Q_f = Ultimate Shaft Frictional Resistance

W_p = Weight of pile; FS = safety factor

(b) Pile Capacity in fine-grained saturated soils

(b.1) Shaft Friction

$Q_f = \Sigma\, (f_s)_i.\ (\text{perimeter})_i\ .\ (\text{length})_i$; Where: f_s= skin friction stress along pile

Use lower of the following: $f_s = 0.5\sqrt{(s_u\, \sigma'_{zo})}$ or $f_s = 0.5\, (s_u^{0.75})(\sigma'_{zo})^{0.25}$

S_u = Undrained shear strength (typically use remolded S_u for driven piles).

(b.2) End bearing

$f_b = N_c(S_u)_b$

$Q_b = N_c(S_u)_b\, A_b$

Where:
f_b = End bearing capacity (stress)

$(S_u)_b$ = Undrained shear strength at pile tip level

N_c = Bearing capacity coefficient; $N_c = 9$ for $(S_u)_b > 25$ kPa; $N_c = 6$ for $(S_u)_b < 25$ kPa.

A_b = Pile tip (base) cross sectional area

(c) Pile Capacity in coarse-grained soils

(c.1) Shaft Friction

$Q_f = \Sigma\ \beta_i.\ (\sigma'_z)_i.\ (\text{perimeter})_i\ .\ (\text{length})_i$

Where:

i given above refers to the i^{th} layer of soil; δ = pile-soil interface friction angle;

β = empirical factor → use $\beta = (1\text{-sin } \phi'_{cs})\ (\tan \delta)$ – Burland (1973)

ϕ'_{cs} = critical state friction angle of soil

σ'_z = vertical effective stress; σ'_x = lateral effective stress

(c.2) End bearing

$Q_b = f_b.\ A_b$

$Q_b = N_q.\ (\sigma'_z)_b\ .\ A_b$

Where

N_q = Bearing capacity coefficient → use $N_q = 0.6^* \exp[0.126\ (\phi'_{cs})_b]$

$(\sigma'_z)_b$ = vertical effective stress at end bearing level

A_b = cross sectional area of pile at pile tip level

$(\phi'_{cs})_b$ = critical state friction angle of soil at pile tip level in degrees