FOUNDATION ENGINEERING
ENCE 4340 AND 4340G
FALL 2010

PREREQUISITE:

ENCE 3340 or consent of department.

CATALOG DATA:

Application of soil mechanics principles to the design of footings, foundations, and retaining walls. Subsurface investigations, dewatering, deep excavations, piles, caissons, and cofferdams. Case histories will be cited.

CLASS MEETING:

Wednesday, 6:00pm to 8:45pm - Room 321 Engineering Building

TEXTBOOK:

Principles of Foundation Engineering, B.M. Das, 6th Edition

INSTRUCTOR:

Chad Held Phone Number: 504-452-1107 Email cheld@eustiseng.com

PREREQUISITES BY TOPIC:

1. Physical and index properties of soils
2. Soil classification
3. Flow of water through soils
4. Stress distribution in soils
5. Consolidation and settlement of structures
6. Shear strength of soils

COURSE TOPICS:

1. Geotechnical properties of soil (self study).
2. Subsoil exploration.
3. Shallow foundations: Bearing capacity as per Terzaghi's theory, the general theory of bearing capacity (Meyerhof and Vesic), bearing capacity of foundations on slope, and effect of water table.
4. Shallow foundations (continued): Elastic and consolidation settlements of foundations on sand and clays.
5. Mat foundations: Common types of combined footings, bearing capacity, and settlement of mat foundations.
6. Lateral earth pressure or retaining walls: Earth pressure at rest, Rankine and Coulomb's theories of earth pressure, and their application to the analysis of gravity and cantilever retaining walls. Stability of retaining walls.
7. Flexible earth structures: Basic principles of sheetpile walls, design procedures for anchored sheetpile walls, and braced excavations.
8. Deep foundations: Basic principles of computation of static bearing capacity of piles; importance of pile load tests, pile-group, and their settlement; introduction to laterally loaded piles; design considerations for drilled shafts.
9. Ground modification and soil improvement: Overview of deep dynamic compaction, vibro-flotation, precompression methods; use of geosynthetics in soft soil.
10. Foundations on difficult soil: Overview of expansive and collapsible soils and design considerations for foundations in these types of soil.

READING/LECTURE	PRACTICE PROBLEMS
Chapter 1 - Self Study	1,5,6,8(A,C,E), 11,12, 17, 22, 26
Chapter 2 - 2.1 - 2.9 (Self Study) 2.10 - 2.20 2.21 - 2.22 (Self Study)	2, 3, 6, 11, 12, 15, 20
Chapter 3 - 3.1 - 3.6 3.11 - 3.12 3.14	1(A,C), 3, 5, 7, 10, 11, 14
Chapter 4 - 4.1 - 4.3 4.5 - 4.6	1, 3, 4, 10, 11
Chapter 5 - 5.1 - 5.5 (Self Study) 5.7 - 5.12 5.15 (Self Study) 5.16 - 5.20	2, 3, 5-7, 10, 13, 14, 18, 19
Chapter 6 - 6.1 - 6.8	1, 3, 6, 9, 13
Chapter 7 - 7.1 - 7.6 7.10 - 7.13	4, 7, 10, 15
Chapter 8 - 8.1 - 8.2 (Self Study) 8.3 - 8.9	2, 6, 7, 10, 12
Chapter 9 - 9.1 - 9.7 (Self Study) 9.8 - 9.16	12, 13, 16
Chapter 10 - 10.1 - 10.4 10.5 - 10.6	1, 2, 6-8, 10
Chapter 11 - 11.1 - 11.4 (Self Study) 11.5 - 11.8 11.11 - 11.15 11.17 - 11.19 11.22 - 11.26	1(A,B), 7, 8, 10, 22(A), 26, 27
Chapter 12 - 12.1 - 12.4 (Self Study) 12.5 - 12.8	To be assigned
Chapters 13 and 14 - Selected articles to be assigned	To be assigned

Some of the practice problems and/or other problem sheets will be assigned as homework problems after the completion of the teaching unit. It is your responsibility to diligently solve the practice problems as the class progresses and understand the concepts and the computational procedures involved. You can discuss homework assignments with other students (unless otherwise stipulated by the instructor), but copying is prohibited and will seriously affect your grade.

GENERAL COURSE POLICIES

ATTENDANCE:

Class attendance is in accordance with the published university policy. You must sign in on a sign-in sheet passed around during class. You are responsible for material identified in the Reading/Lecture schedule listed earlier and covered in class, even if absent from the class for authorized activities. Any absence should be coordinated with the instructor before the absence. Assignments or project work will not be accepted as late due to absence, unless that absence is coordinated with the instructor well in advance.

CLASSROOM PROCEDURE:

Come to class on time. The assigned homework will be due at the beginning of the class on the due date. If you are late, you stand the chance of missing a quiz and you will not be allowed to turn in the homework. Bring the course text, calculator, ruler, and other stationery material to each class period. **CELL PHONES SHOULD BE TURNED OFF BEFORE CLASS BEGINS.** Feel free to ask questions of the instructor during class, but **please do not ask other students, as talking disturbs my concentration and of other class members.** If you have difficulty understanding the material, make an appointment with me and I will help you to the best of my ability.

STUDY ASSIGNMENTS:

Study assignments for each class are listed in the Reading/Lecture schedule. Additionally, you will be informed about them at the end of the class period. You should study these articles at home and come to class with a general understanding of the concepts. This will make the classroom lecture more understandable.

ORGANIZATION OF SUBMISSIONS:

A significant part of engineering is written communication. Hence, it is important you turn in your written work neat and organized.

ACADEMIC INTEGRITY:

Academic integrity is fundamental to the process of learning and evaluating academic performance. As engineers, you will be responsible for upholding the canons of ethics of the profession. Academic dishonesty will not be tolerated and may lead to an F for that assignment/exam/project or for the whole course. **Academic dishonesty includes, but is not limited to, the following: cheating, plagiarism, and being an accessory to acts of academic dishonesty.** Refer to UNO Judicial Code for further information.

GRADING SYSTEM:

Class Exams (Two at 20%)	40%
Homework/Quizzes	20%
Homework Participation	5%
Project/Presentation	10%
Final Exam	25%

A = 90 and Above
B = 80 and < 90
C = 70 and < 80
D = 60 and < 70

GRADUATE STUDENTS NEED TO DO A PROJECT IN ADDITION TO THE REGULAR ASSIGNED PROJECT. THESE MUST BE DISCUSSED WITH THE INSTRUCTOR. IMPORTANT NOTE: NO MAKEUP EXAMINATIONS WILL BE GIVEN EXCEPT FOR MEDICAL REASONS WITH SUBSTANTIAL PROOF. NO LATE HOMEWORK WILL BE ACCEPTED. THE HOMEWORK ASSIGNMENT MUST BE TURNED IN BEFORE CLASS BEGINS. ASSIGNMENTS TURNED IN AT THE END OF CLASS WILL NOT BE ACCEPTED.

Chad Held

66
~~63~~/70

From: Donald Jerolleman [asce504@gmail.com]
Sent: Saturday, September 18, 2010 8:49 PM
To: Chad Held
Subject: Homework #3
Attachments: Foundations HW3 p001.jpg; Foundations HW3 p002.jpg; Foundations HW3 p003.jpg

Mr. Held

Thank you for allowing me to email this homework. Please let me know if the three attachments do not come out correctly.

Thank you,
Donald Jerolleman

--
Donald Jerolleman, Jr., S.M.ASCE, S.M.SAME
(504) 606-5599

ASCE - UNO
American Society of Civil Engineers
Chapter President

SAME - UNO
Society of American Military Engineers
Chapter Secretary

NHMA (www.nhma.info)
Natural Hazards Mitigation Association
Founding Member

1(A, C), 3, 5, 7, 10, 11, 14

3.1) (a) p. 138 equ. 3.3 continuous → $q_u = c'N_c + \overset{\nearrow \gamma D_f}{q} N_q + \frac{1}{2}\gamma B N_\gamma$

From table 3.1 ($\phi' = 28°$) → $N_c = 31.61$, $N_q = 17.81$, $N_\gamma = 13.70$

$q_u = 400\ \text{lb/ft}^2(31.61) + 3\text{ft}(110\ \text{lb/ft}^3)(17.81) + (0.5)(110\ \text{lb/ft}^3)(3\text{ft})(13.7) = \underline{20782\ \text{lb/ft}^2}$

$q_{all\,(gross)} = \frac{q_u}{FS} = \boxed{5195\ \text{lb/ft}^2}$ ✓

(c) Table 3.1 ($\phi = 30°$) → $N_c = 37.16$, $N_q = 22.46$, $N_\gamma = 19.13$

$q_u = (1.3)(\emptyset)(37.16) + (16.5\ \text{kN/m}^3)(2\text{m})(22.46) + (0.4)(16.5\ \text{kN/m}^3)(3\text{m})(19.13)$

$q_u = 1120\ \text{kN/m}^2$ $\quad \boxed{q_{all} = 280\ \text{kN/m}^2}$ ✓

Square Foundation → $q_u = 1.3(c')(N_c) + q(N_q) + 0.4(\gamma)(B)(N_\gamma)$

3.3) (a) (c)

(a) equ. 3.19 p. 143 ↓ Table 3.3 ($\phi = 28°$) $\underline{N_c = 25.80}$, $\underline{N_q = 14.72}$, $\underline{N_\gamma = 16.72}$

$q_u = c' N_c F_{cs} F_{cd} F_{ci} + \overset{\gamma(D_f)}{q} N_q F_{qs} F_{qd} F_{qi} + 0.5(\gamma) B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$

Table 3.4 p. 145 $\left(\frac{D_f}{B} = \frac{3}{3} = 1\right)$ → $\underline{F_{cs}, F_{qs}, F_{\gamma s} \sim 1}$ b/c continuous

→ ($\phi' > 0$) $F_{cd} = 1 + 0.4\left(\frac{D_f}{B}\right) = \underline{1.4}$ $\quad \underline{F_{\gamma d} = 1}$

$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right) = \underline{1.299}$

No part B

~~-3~~

OK

$q_u = (400\ \text{lb/ft}^2)(25.8)(1)(1.4)F_{ci} + (110\ \text{lb/ft}^3)(3\text{ft})(14.72)(1)(1.299)(F_{qi}) + 0.5(110\ \text{lb/ft}^3)(3\text{ft})(16.72)(1)(1)(F_{\gamma i})$

$= 14448(F_{ci}) + 6310(F_{qi}) + 2759(F_{\gamma i}) = 23517\ \text{lb/ft}^2$

Table 3.4
since $\beta = \emptyset$ $F_{ci}, F_{qi}, F_{\gamma i} = 1$ } ↗

$\boxed{q_{ALL} = 5879\ \text{lb/ft}^2}$ ✓

(c) Table 3.3 ($\phi = 30°$) → $N_c = 30.14$, $N_q = 18.40$, $N_\gamma = 22.4$

Table 3.4 $\left(\frac{B}{L} = 1\right)$ → $F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = \underline{1.61}$ $\quad F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi' = \underline{1.577}$ $\quad F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = \underline{0.6}$

$\left(\frac{D_f}{B} = \frac{2}{3} \leq 1\right)$ $F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right) = \underline{1.192}$ $\quad F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} = \underline{1.204}$

$F_{\gamma d} = 1$ $\quad$ since $\beta = \emptyset$ $\quad F_{ci} = F_{qi} = F_{\gamma i} = 1$

$q_u = \emptyset(30.14)(1.61)(1.204)(1) + 16.5\ \text{kN/m}^3(2\text{m})(18.4)(1.577)(1.192)(1) + 0.5(16.5\ \text{kN/m}^3)(3\text{m})(22.4)(0.6)(1)(1)$

$= 1474\ \text{kN/m}^2$ $\quad \boxed{q_{all} = 369\ \text{kN/m}^2}$ ✓

NOT clear which is B or L

3.5) $B = 2m$, $L = 3m$, $D_f = 2m$, $\phi' = 25^\circ$, $c' = 50\ KN/m^2$, $FS = 4$, $\frac{B}{L} = \frac{2}{3}$, $\frac{D_f}{B} = 1$

use. eqn 3.23 Find $\sigma_{All(net)}$ $\gamma'_{tab} = 16.8\ KN/m^2$ $\gamma_{sat} = 19.4\ KN/m^2$

Equ. 3.19

$$qu = c' N_c F_{cs} F_{cd} \overset{(1)}{\cancel{F_{ci}}} + \overset{\Sigma \gamma (D_f)}{q} N_q F_{qs} F_{qd} \overset{(1)}{\cancel{F_{qi}}} + 0.5 \gamma B N_\gamma F_{\gamma s} \overset{(1)}{\cancel{F_{\gamma d}}} \overset{(1)}{\cancel{F_{\gamma i}}}$$

Same as on Test!,

-1 * Did not cover 2 γs yet with these methods how do we address?, -1

Table 3.3 ($\phi' = 25^\circ$) $N_c = \underline{20.72}$ $N_q = \underline{10.66}$ $N_\gamma = \underline{10.88}$

$F_{cs} = 1 + \left(\frac{2}{3}\right)\left(\frac{10.66}{\cancel{10.88}\ 20.72}\right) = \underline{1.343}$ $F_{qs} = \underline{1.311}$ $F_{\gamma s} = 0.\underline{733}$

$F_{cd} = \underline{1.343}$ $F_{qd} = \underline{1.311}$

$$qu = 50\ ^{KN}/_{m^2}(20.72)(1.343)(1.343) + \left(\underbrace{\left[16.8\ ^{KN}/_{m^2}(1m) + 19.4\ ^{KN}/_{m^2}(1m)\right]}_{\text{guess}}(10.66)(1.311)(1.311) + \right.$$

$$(0.5)\underbrace{\left(\frac{16.8 \frac{KN}{m^2}(1m) + 19.4 \frac{KN}{m^2}(1m)}{2m}\right)}_{\text{guess}}(2m)(10.88)(0.733)(1)(1) = 2676\ ^{KN}/_{m^2}$$

$$q_{All(net)} = \frac{qu - q}{FS} = \frac{2676 - (1m(16.8) + 1m(19.4))}{4} = \boxed{660\ ^{KN}/_{m^2}}$$

3.7) $B = 8'$, $L = 8'$, grainular soil, $D_f = 5'$, $\gamma = 110\ lb/ft^3 = 0.06366\ lb/in^3$

z/N_{60} : 5'/11, 10'/14, 15'/16, 20'/21, 25'/24

(a) Find ϕ', $p_a = 14.7\ lb/in^2$ where is method/Formula in 7th ed?

→ eqn 2.27 (my book) $\phi' = \tan^{-1}\left(\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_o}{p_a}\right)}\right)^{0.34}$

$\frac{D_f}{B} = \frac{5}{8} < 1$

Z(in)	Z(ft)	N_{60}	$\sigma'_o = z(\gamma)$ [lb/in²]	p_a [lb/in²]	ϕ'
60	5'	11	3.82	14.7	40.5
120	10'	14	7.64	↓	40.3
180	15'	16	11.46		39.6
240	20'	21	15.28		40.5
300	25'	24	19.10		40.4

Avg. $\phi' = 40.3^\circ$ ✓

(b) equ 3.19 my book (assum $c' = \phi$)(FS not given)

$$q_u = \cancel{c'} N_c F_{cs} F_{cd} \cancel{F_{ci}} + \overset{[D_f(\gamma)]}{q} N_q F_{qs} F_{qd} \cancel{F_{qi}} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} \cancel{F_{\gamma d}} \cancel{F_{\gamma i}}$$

Table 3.3 ($\phi = 40^\circ$) $N_c = \underline{75.31}$, $N_q = \underline{64.2}$ ✓, $N_\gamma = \underline{109.41}$ ✓

$F_{cs} = 1 + 1\left(\frac{64.2}{75.31}\right) = \underline{1.852}$ $F_{qs} = 1 + 1(\tan(40)) = \underline{1.839}$ ✓ $F_{\gamma s} = \underline{0.6}$ ✓

$F_{cd} = 1.1338 - \frac{1 - 1.1338}{75.31 \tan 40} = \underline{1.136}$ $F_{qd} = 1 + 2(\tan 40)(1 - \sin 40)^2\left(\frac{5}{8}\right) = \underline{1.1338}$ ✓ $F_{\gamma d} = \underline{1}$ ✓

B/C $\beta = \phi$ $F_{ci} = F_{qi} = F_{\gamma i} = 1$

$$q_u = 5'(110\ ^{lb}/_{ft^3})(64.2)(1.839)(1.1338) + 0.5(110\ ^{lb}/_{ft^3})(8')(109.41)(0.6) = \boxed{102.5\ ^{K}/_{ft^2}}$$ ✓

-1 $Q_U = q_U(\text{Area}) = 102560(64\ ft^2) = 6563\ \text{Kips}$

3.10) p.159 $B' = B - 2e$, $L' = L$, $A' = B'L'$, $\phi = 26°$, $\gamma_{sat} = 122\ lb/ft^3$, $\gamma = 110\ lb/ft^3$

$c' = 500\ lb/ft^2$, $e = 0.65\,ft$ ∴ $B' = 6.7\,ft$, $L' = 8\,ft$

$D_f/B \le 1$

$$Q_{ult} = q_u' A' = \left[c' N_c F_{cs} F_{cd} F_{ci} + \{D_f \gamma\} N_q F_{qs} F_{qd} F_{qi} + 0.5(\gamma) B' N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}\right] B'L'$$

Table 3.3 ($\phi = 26°$) $N_c = 22.25$ ✓ $N_q = 11.85$ ✓ $N_\gamma = 12.54$ ✓

$F_{cs} = 1 + \frac{6.7\,ft}{B'}\left(\frac{11.85}{22.25}\right) = 1.446$ ✓ $F_{qs} = 1.408$ ✓ $F_{\gamma s} = 0.665$ ✓

-1 $F_{cd} = 1.273$ ✗ 1.325 $F_{qd} = (\text{use } B \text{ not } B') = 1.25$ ✓ (All other Factors = 1)

1,325

$$Q_{ult} = \left[500(22.25)(1.446) + \left[(3')(110\ lb/ft^3) + (3.5')(122\ lb/ft^3)\right](11.85)(1.408)(1.25) + (0.5)(122 - 62.4)(6.7)\right.$$
$$\left.(12.54)(0.665)\right](6.7)(8) = \boxed{1798^k}$$

1833 kips

3.11) $L' = ?$

can not be done L' is not given.

If L Assumed ∞ ; $c' = 400\ lb/ft^2$, $\phi' = 25°$, $\gamma = 105\ lb/ft^3$, $\gamma_{sat} = 118\ lb/ft^3$

-1 $Q_{ult} = B\left[c' N_{c(e)} + q N_{q(e)} + \frac{1}{2}\gamma B N_{\gamma(e)}\right]$ (equ. 3.42 p.159)

solve for e?

$\frac{e}{B} = \frac{2}{5} = 0.4$ ∴ $N_{q(e)} \cong 4$ (Figure 3.15 p.160) ; $N_{q(e)} \cong 4$ (Fig 3.16) ; $N_{\gamma(e)} \cong \varnothing$ (Fig. 3.17)

$$Q_{ult} = 5\,ft\left[400\ lb/ft^2(4) + \left[2'(105\ lb/ft^3) + 2'(118\ lb/ft^3)\right](4) + \varnothing\right] = 3384 = \boxed{3.4^K}$$

*Does not seem right

3.14) $e_L = 0.06\,ft$, $e_B = 1.5\,ft$, $B = 4\,ft$, $L = 6\,ft$, $D_f = 3\,ft$, $S.F. = 4$, $\gamma = 115\ lb/ft^3$, $\phi = 35°$, $c' = \varnothing$

$\frac{e_B}{B} = \frac{1.5}{4} > \frac{1}{6}$ $\frac{e_L}{L} = \frac{0.06}{6} < \frac{1}{6}$ ∴ CASE III p.166 $\frac{D_f}{B} < 1$

$= 0.375 < \frac{1}{2}$ $= 0.01$

$A' = \frac{1}{2}(B_1 + B_2)L$; p.168 Fig 3.22 $\left(\frac{e_B}{B}\right)0.375$, $\left(\frac{e_L}{L}\right)0.01 \rightarrow \frac{B_2}{B} = 0.225$, $\frac{B_1}{B} = 0.235$

$\rightarrow B_2 = 0.9\,ft$, $B_1 = 0.94\,ft$ $\rightarrow$ $A' = 5.52\,ft^2$, $B' = \frac{A'}{L} = 0.92\,ft$, $L' = L = 6\,ft$

$$Q_{ult} = A'\left[\varnothing + (3'(115\ lb/ft^3))(N_q F_{qs} F_{qd})(1) + 0.5(115\ lb/ft^3)(0.92)(F_{\gamma s})(1)(1)(N_\gamma)\right)$$

Table 3.3 ($\phi = 35°$) → $N_q = \underline{33.3}$, $N_\gamma = \underline{48.03}$ Table 3.4 $F_{qs} = 1 + \left(\frac{0.92}{6}\right)(\tan 35) = \underline{1.107}$

$F_{\gamma s} = 1 - 0.4\left(\frac{0.92}{6}\right) = \underline{0.939}$ $F_{qd} = 1 + (2\tan\phi(1 - \sin\phi)^2\left(\frac{D_f}{B}\right) = \underline{1.191}$ ✓

$Q_{ult} = 96.8^K$

$$Q_{All} = 24.2^K$$

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1.17) Refer to soil profile. (from 1.12) The clay is normally consolidated.

A laboratory consolidation test on the clay gave the following results:

Pressure $\left(\frac{lb}{in^2}\right)$	e
21	0.91
42	0.792

If avg. effective stress on clay increases by 1000 $\frac{lb}{ft^2}$

ⓐ what would be the total consolidation settlement?

ⓑ If $C_v = 1.45 \times 10^{-4} \frac{in^2}{s}$, how long will it take for ½ the consolidation settlement to take place?

equ. 1.49 p.35

$$C_c = \frac{e_1 - e_2}{\log_{10}\left(\frac{\sigma_2'}{\sigma_1'}\right)} = \frac{0.91 - 0.792}{\log_{10}\left(\frac{42}{21}\right)} = 0.392$$

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1.22 A direct shear test was conducted on dry sand. The results were as follows: Area of the specimen = 2" × 2"

Normal force (lb)	Shear force @ failure (lb)
50	43.5
110	95.5
150	132.0

Graph the shear stress at failure against normal stress and determine the soil friction angle, ϕ'.

1.26) 2- consolidated - drained triaxial tests on a clay:

<u>Test I</u>: $\sigma_3 = 140 \frac{kN}{m^2}$, $\sigma_{1(failure)} = 368 \frac{kN}{m^2}$

<u>Test II</u>: $\sigma_3 = 280 \frac{kN}{m^2}$, $\sigma_{1(failure)} = 701 \frac{kN}{m^2}$

* Find shear str. parameters; that is, c' & ϕ'.

2.2 A soil profile is shown in Figure P2.2 along with the standard penetration numbers in the clay layer. Use Eqs. (2.11) and (2.12) to determine and plot the variation of c_u and OCR with depth.

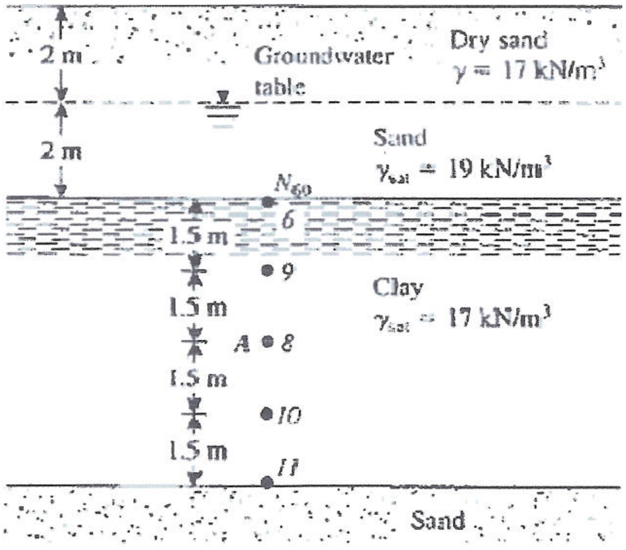


Figure P2.2

2.3 Following is the variation of the field standard penetration number (N_{60}) in a sand deposit:

Depth (m)	N_{60}
1.5	5
3	7
4.5	9
6	8
7.9	13
9	12

The groundwater table is located at a depth of 5.5 m. Given: the dry unit weight of sand from 0 to a depth of 5.5 m is 18.08 kN/m^3, and the saturated unit weight of sand for depth 5.5 to 10.5 m is 19.34 kN/m^3. Use the relationship of Liao and Whitman given in Eq. (2.14) to calculate the corrected penetration numbers.

2.6 Following are the standard penetration numbers determined from a sandy soil in the field:

Depth (ft)	Unit weight of soil (lb/ft^3)	N_{60}
10	106	7
15	106	9
20	106	11
25	118	16
30	118	18
35	118	20
40	118	22

Using Eq. (2.25), determine the variation of the peak soil friction angle, ϕ'. Estimate an average value of ϕ' for the design of a shallow foundation. (*Note:* For depth greater than 20 ft, the unit weight of soil is 118 lb/ft^3. Use p_a = 14.7 lb/in^2.

2.11 In a deposit of normally consolidated dry sand, a cone penetration test was conducted. Following are the results:

Depth (m)	Point resistance of cone, q_c (MN/m^2)
1.5	2.06
3.0	4.23
4.5	6.01
6.0	8.18
7.5	9.97
9.0	12.42

Assuming the dry unit weight of sand to be 15.5 kN/m^3, estimate the average peak friction angle, ϕ', of the sand. Use Eq. (2.47).

2.15 A dilatometer test was conducted in a clay deposit. The groundwater table was located at a depth of 3 m below the surface. At a depth of 8 m below the surface, the contact pressure (p_o) was 280 kN/m^2 and the expansion stress (p_1) was 350 kN/m^2. Determine the following:

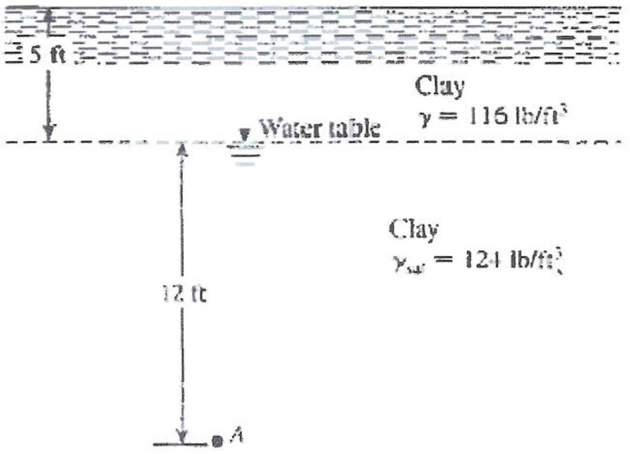


Figure P2.13

a. Coefficient of at-rest earth pressure, K_o
b. Overconsolidation ratio, OCR
c. Modulus of elasticity, E_s
Assume σ'_o at a depth of 8 m to be 95 kN/m^2 and $\mu_s = 0.35$.

2.20 Repeat Problem 2.19 for the following data.

Distance from the source of disturbance (ft)	Time of first arrival of *P*-waves (sec × 10^3)
25	49.06
50	81.96
75	122.8
100	148.2
150	174.2
200	202.8
250	228.6
300	256.7

1.21 Eq. (1.66): $T_c = \frac{C_v t_c}{H^2}$. T_c = 60 days = 60 × 24 × 60 × 60 s; $H = \frac{2}{2}$ m = 1000 mm

$$T_c = \frac{(8 \times 10^{-3})(60 \times 24 \times 60 \times 60)}{(1000)^2} = 0.0415$$

After 30 days: $T_v = \frac{C_v t}{H^2} = \frac{(8 \times 10^{-3})(30 \times 24 \times 60 \times 60)}{(1000)^2} = 0.0207$

From Figure 1.28, for $T_v = 0.0207$ and $T_c = 0.0415$, $U = 5\%$. So

$S_c = (0.05)(150) = 7.5$ mm

After 120 days: $T_v = \frac{(8 \times 10^{-3})(120 \times 24 \times 60 \times 60)}{(1000)^2} = 0.083$

From Figure 1.28, for $T_v = 0.083$ and $T_c = 0.0415$, $U = 27\%$. So

$S_c = (0.27)(150) = 40.5$ mm

1.22

Area (in.^{2})	Normal force, N (lb)	$\sigma' = N/A$ (lb / in.^{2})	Shear force, S (lb)	$\tau = S/A$ (lb / in.^{2})	ϕ'= tan-1(τ/σ')
4	50	12.5	43.5	10.88	41.06
4	110	27.5	95.5	23.88	40.97
4	150	37.5	132	33	41.35

When plotted, $\phi' \approx 41°$

1.23 $c' = 0$. Eq. (1.69):

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right)$$

$$30 + 96 = 30 \tan^2\left(45 + \frac{\phi'}{2}\right); \quad \phi' = 2\left[\tan^{-1}\left(\frac{126}{30}\right)^{1/2} - 45\right] \approx 38°$$

1.24 $$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right)$$

$$20 + 40 = 20 \tan^2\left(45 + \frac{\phi'}{2}\right); \quad \phi' = 30°$$

1.25 $c' = 0$. Eq. (1.69): $\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) = 140\tan^2\left(45 + \frac{28}{2}\right) = \mathbf{387.8\ kN/m^2}$

1.26 Eq. (1.69): $\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\phi'}{2}\right) + 2c' \tan^2\left(45 + \frac{\phi'}{2}\right)$

$368 = 140\tan^2(45 + \phi'/2) + 2c'\tan(45 + \phi'/2)$ (a)

$701 = 280\tan^2(45 + \phi'/2) + 2c'\tan(45 + \phi'/2)$ (b)

Solving Eqs. (a) and (b), $\phi' = \mathbf{24°}$ and $c' = \mathbf{12\ kN/m^2}$

1.27 $\phi = \sin^{-1}\left(\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3}\right) = \sin^{-1}\left(\frac{32 - 13}{32 + 13}\right) = \mathbf{25°}$

$\phi = \sin^{-1}\left(\frac{\sigma_1' - \sigma_3'}{\sigma_1' + \sigma_3'}\right)$

$\sigma_1' = 32 - 5.5 = 26.5\ \text{lb/in.}^2$; $\sigma_3' = 13 - 5.5 = 7.5\ \text{lb/in.}^2$

$\phi' = \sin^{-1}\left(\frac{26.5 - 7.5}{26.5 + 7.5}\right) = \mathbf{34°}$

Normally consolidated clay; $c_{cu} = \mathbf{0}$; $c' = \mathbf{0}$

1.28 $\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right)$. $\sigma_1 = 150\tan^2\left(45 + \frac{20}{2}\right) = 305.9\ \text{kN/m}^2$

$\frac{\sigma_1'}{\sigma_3'} = \tan^2\left(45 + \frac{\phi'}{2}\right)$; $\frac{305.9 - u}{150 - u} = \tan^2\left(45 + \frac{28}{2}\right)$; $u = \mathbf{61.9\ kN/m^2}$

1.29 $\frac{c_u}{\sigma_o'} = 0.11 + 0.0037\text{PI}$; $\frac{c_u}{(8)(19.6 - 9.81)} = 0.11 + (0.0037)(21) = \mathbf{14.7\ kN/m^2}$

CHAPTER 2

2.1 a. Eq. (2.5): $A_R(\%) = \dfrac{D_o^2 - D_i^2}{D_i^2} \times 100 = \left[\dfrac{(2)^2 - (1.875)^2}{(1.875)^2}\right](100) = \mathbf{13.78\%}$

b. $A_R = \dfrac{10}{100} = \dfrac{(2)^2 - D_i^2}{D_i^2} = \mathbf{1.907\ in.}$

2.2

Depth from ground surface (m)	N_{60}	c_u [a] (kN / m²)	σ'_o (MN / m²)	OCR [b]
4.0	6	**105.4**	$\frac{1}{1000}[2(17)+2(19-9.81)] = 0.0524$	**5.06**
5.5	9	**141.1**	$0.0524 + \frac{1}{1000}(17-9.81)(1.5) = 0.0632$	**5.88**
7.0	8	**129.6**	$0.0632 + \frac{1}{1000}(17-9.81)(1.5) = 0.074$	**4.86**
8.5	10	**152.2**	$0.074 + \frac{1}{1000}(17-9.81)(1.5) = 0.0848$	**5.16**
10.0	11	**163.0**	$0.0848 + \frac{1}{1000}(17-9.81)(1.5) = 0.0956$	**5.08**

[a] $c_u\ (\mathrm{kN/m^2}) = 29N_{60}^{\ 0.72}$; [b] $\mathrm{OCR} = 0.193(N_{60}/\sigma'_o)^{0.689}$

2.3

Depth (m)	σ'_o (kN / m²)
1.5	$18.08 \times 1.5 = 27.12$
3	$18.08 \times 3.0 = 54.24$
4.5	$18.08 \times 4.5 = 81.36$
6	$18.08 \times 5.5 + (19.34 - 9.81)(0.5) = 104.2$
7.5	$18.08 \times 5.5 + (19.34 - 9.81)(2) = 118.5$
9	$18.08 \times 5.5 + (19.34 - 9.81)(3.5) = 132.8$

Eq. (2.14): $C_N = \left[\frac{1}{(\sigma'_o/p_a)}\right]^{0.5}$. Now the following table can be prepared.

Depth (m)	N_{60}	σ'_o (kN/m²)	C_N	$(N_1)_{60}$ [a]
1.5	5	27.12	1.92	**10**
3	7	54.24	1.36	**10**
4.5	9	81.36	1.11	**10**
6	8	104.2	0.98	**8**
7.5	12	118.5	0.92	**11**
9	11	132.8	0.87	**10**

[a] rounded to nearest whole number

2.4

Depth (m)	N_{60}	σ'_o (kN / m²)	ϕ' (deg) [Eq. (2.24)	ϕ' (deg) [Eq. (2.25)]
1.5	5	27.12	28.59	33.04
3.0	7	54.24	29.17	33.63
4.5	9	81.36	29.76	33.98
6.0	8	104.2	29.47	31.61
7.5	12	118.5	30.62	34.48
9.0	11	132.8	30.33	33.0
			Av. 29.66° ≈ **30°**	Av. 33.29° ≈ **33°**

2.5

Depth (m)	σ'_o (kN / m²)	N_{60}	D_{50} (mm)	p_a (kN / m²)	D_R (%) [Eq. (2.19)]
1.5	$1.5 \times 18 = 27$	6	0.6	100	**61.2**
3.0	$3 \times 18 = 54$	8	0.6	100	**50**
4.5	$4.5 \times 18 = 81$	9	0.6	100	**43.3**
6.0	$6 \times 18 = 108$	8	0.6	100	**35.4**
7.5	$108 + 1.5(20.2 - 9.81) = 123.6$	13	0.6	100	**42.1**
9.0	$123.6 + 1.5(20.2 - 9.81) = 139.2$	14	0.6	100	**41.2**

2.6

Depth (ft)	γ (lb / ft^3)	σ'_o (lb / in.^{2})	p_a (lb / in.^{2})	N_{60}	ϕ' (deg)^a [Eq. (2.25)]
10	106	7.36	14.7	7	34
15	106	11.04	14.7	9	34
20	106	14.72	14.7	11	35
25	118	18.82	14.7	16	37
30	118	22.92	14.7	18	36
35	118	27.02	14.7	20	36
40	118	31.12	14.7	22	36

^{a}rounded to nearest whole number

$$\text{Average } \phi' = \frac{1}{7}(34 + 34 + 35 + 37 + 36 + 36 + 36) = 35.4° \approx 35°$$

2.7 Eq. (2.18b): $D_r\,(\%) = 12.2 + 0.75\left[222N_{60} + 2311 - 771\text{OCR} - 779\left(\frac{\sigma'_o}{p_a}\right) - 50C_u^2\right]^{0.5}$

At 10 ft: $$D_r = 12.2 + 0.75\left[\begin{matrix}(222)(9) + 2311 - 711(3.0) \\ -779\left(\frac{1150}{2000}\right) - (50)(3.2)^2\end{matrix}\right]^{0.5} = 38.35\%$$

At 15 ft: $$D_r = 12.2 + 0.75\left[\begin{matrix}(222)(11) + 2311 - 711(3.0) \\ -779\left(\frac{1725}{2000}\right) - (50)(3.2)^2\end{matrix}\right]^{0.5} = 40.6\%$$

At 20 ft: $$D_r = 12.2 + 0.75\left[\begin{matrix}(222)(12) + 2311 - 711(3.0) \\ -779\left(\frac{2030}{2000}\right) - (50)(3.2)^2\end{matrix}\right]^{0.5} = 41.63\%$$

Average $D_r = (⅓)(38.35 + 40.6 + 41.63) \approx$ **40%**

2.8 Eq. (2.29): $c_u = \frac{T}{K}$

Eq. (2.31): $K = 366 \times 10^{-8} D^3 = 366 \times 10^{-8}(6.35)^3 = 93.7 \times 10^{-5}$

$$c_{u(\text{VST})} = \frac{0.072}{93.7 \times 10^{-5}} = 76.84 \text{ kN / m}^2$$

$$c_{u\,(\text{corrected})} = \lambda c_{u\,(\text{VST})} = [1.7 - 0.54\log(\text{PI})](76.84)$$

$$= [1.7 - 0.54\log(51 - 18)](76.84) = \mathbf{67.62\ kN / m^2}$$

2.9 a. From Eq. (2.33), $K = 0.0021D^3 = 0.0021(2)^3 = 0.0168$

$$c_{u(\text{VST})} = \frac{T}{K} = \frac{12.4}{0.0168} = \mathbf{738.1\ lb / ft^2}$$

b. $c_{u\,(\text{corrected})} = [1.7 - 0.54\log(\text{PI})](738.1)$

$$= [1.7 - 0.54\log(64 - 29)](738.1) = \mathbf{639.3\ lb / ft^2}$$

2.10 $\text{OCR} = \beta\dfrac{c_{u(\text{field})}}{\sigma'_o}$; $\beta = 22(\text{PI})^{-0.48} = 22(51 - 18)^{-0.48} = 4.11$

$\sigma'_o = (2)(17) + (2)(19 - 9.81) + (3)(17 - 9.81) = 73.95\ \text{kN / m}^2$

$c_{u\,(\text{VST})} = 76.84\ \text{kN/m}^2$

$$\text{OCR} = (4.11)\left(\frac{76.84}{73.95}\right) = \mathbf{4.27}$$

2.11 $\gamma = 15.5\ \text{kN / m}^3$; $\underbrace{\sigma'_o}_{(\text{MN/m}^2)} = \left(\underbrace{\gamma}_{(\text{kN/m}^3)} \times \underbrace{\text{depth}}_{\text{m}}\right)\dfrac{1}{1000}$

Depth	σ'_o (MN / m^2)	q_c (MN / m^2)	ϕ'[a] (deg)
1.5	0.0233	2.06	40
3.0	0.0465	4.23	40.2
4.5	0.0698	6.01	39.9
6.0	0.093	8.18	40
7.5	0.1163	9.97	39.9
9.0	0.1395	12.42	40.1
[a]Eq. (2.47)		Av. $\phi' = 40°$	

2.12

Depth (m)	σ'_o (kN / m²)	p_a (kN / m²)	q_c (kN / m²)	D_R (%) [Eq. (2.45)]
1.5	23.3	100	2060	**42.9**
3.0	46.5	100	4230	**53.9**
4.5	69.8	100	6010	**58.3**
6.0	93	100	8180	**63.1**
7.5	116.3	100	9970	**65.7**
9.0	139.5	100	12420	**69.5**

2.13 a. $\sigma_o = (5)(116) + (124)(12) = 2068 \text{ lb/ft}^2$. Eq. (2.51):

$$c_u = \frac{q_c - \sigma_o}{N_k} = \frac{(90 \times 144) - 2068}{15} \approx \mathbf{726 \text{ lb/ft}^2}$$

b. $\sigma'_o = (5)(116) + (124 - 62.4)(12) = 1319.2 \text{ lb/ft}^2$. Eq. (2.55):

$$\text{OCR} = 0.37\left(\frac{q_c - \sigma_o}{\sigma'_o}\right)^{1.01} = (0.37)\left[\frac{(90 \times 144) - 2068}{1319.2}\right]^{1.01} = \mathbf{3.12}$$

2.14 Eq. (2.56):

$$E_p = 2(1 + \mu_s)(V_o + v_m)\left(\frac{\Delta p}{\Delta v}\right) = (2)(1 + 0.5)\left(535 + \frac{45 + 180}{2}\right)\left(\frac{326.5 - 42.4}{180 - 46}\right)$$

$$= \mathbf{4121.6 \text{ kN/m}^2}$$

2.15 a. $K_D = \dfrac{p_o - u_o}{\sigma'_o} = \dfrac{280 - (9.81)(8 - 3)}{95} = 2.43$. Eq. (2.63):

$$K_o = (K_D/1.5)^{0.47} - 0.6 = (2.43/1.5)^{0.47} - 0.6 = \mathbf{0.65}$$

b. Eq. (2.64): $\text{OCR} = (0.5K_D)^{1.56} = (0.5 \times 2.43)^{1.56} = \mathbf{1.36}$

c. $E = (1 - \mu^2)E_D = (1 - 0.35^2)(34.7)(350 - 280) = \mathbf{2131 \text{ kN/m}^2}$

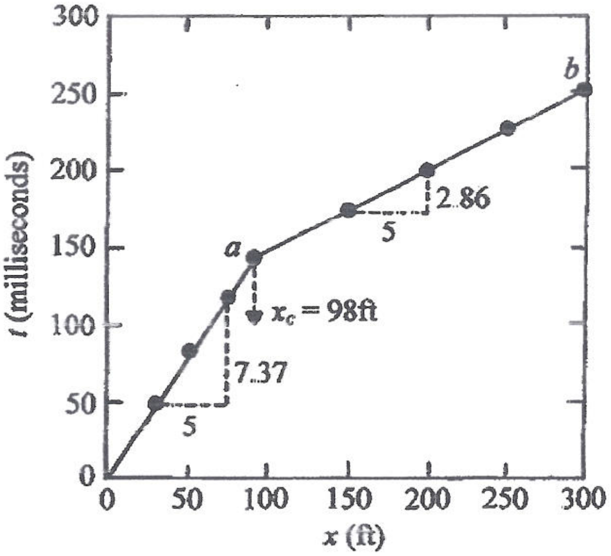
Eq. (2.74): $Z_2 = \frac{1}{2}\left[T_{i2} - \frac{2Z_1\sqrt{v_3^2 - v_1^2}}{(v_3)(v_1)}\right]\left[\frac{v_3 v_2}{\sqrt{v_3^2 - v_2^2}}\right]$; $T_{i2} \approx 20 \times 10^{-3}$ sec

$$\frac{2Z_1\sqrt{v_3^2 - v_1^2}}{(v_3)(v_1)} = \frac{(2)(2.6)\sqrt{(3390)^2 - (492)^2}}{(3390)(492)} = 0.0105$$

$$\frac{(v_3)(v_2)}{\sqrt{v_3^2 - v_2^2}} = \frac{(3390)(1390)}{\sqrt{(3390)^2 - (1390)^2}} = 1524$$

So, $Z_2 = (½)(0.02 - 0.0105)(1524) = \mathbf{7.24\ m}$

2.20 A time-distance plot is shown.



Slope of $Oa = \frac{1}{v_1} = \frac{7.37 \times 10^{-3}}{5}$

$\mathbf{v_1 = 678.4\ ft / sec}$

Slope of $ab = \frac{1}{v_2} = \frac{2.86 \times 10^{-3}}{5}$

$\mathbf{v_2 = 1748.25\ ft / sec}$

$$Z_1 = \frac{1}{2}\sqrt{\frac{v_2 - v_1}{v_2 + v_1}}x_c$$

$$= \frac{1}{2}\sqrt{\frac{1748.25 - 678.4}{1748.25 + 678.4}} \times 98 = \mathbf{32.87\ ft}$$

CHAPTER 1

1.1 d. $\gamma = \frac{(87.5)(9.81)}{(1000)(0.05)} = 17.17\ \mathrm{kN/m^3}$

c. $\gamma = \frac{\gamma}{1+w} = \frac{17.17}{1+0.15} = 14.93\ \mathrm{kN/m^3}$

a. Eq. (1.12): $\gamma_d = \frac{G_s\gamma_w}{1+e}$

$$14.93 = \frac{(2.68)(9.81)}{1+e}; \quad e = 0.76$$

b. Eq.(1.6): $n = \frac{e}{1+e} = \frac{0.76}{1+0.76} = 0.43$

e. From Figure 1.3b: $S = \frac{V_w}{V_v} = \frac{wG_s}{e} = \left[\frac{(0.15)(2.68)}{0.76}\right](100) = 53\%$

1.2 a. From Eqs. (1.11) and (1.12) it can be seen that

$$\gamma_d = \frac{\gamma}{1+w} = \frac{20.1}{1+0.22} = 16.48\ \mathrm{kN/m^3}$$

b. $\gamma_d = 16.48\ \mathrm{kN/m^3} = \frac{G_s\gamma_w}{1+e} = \frac{G_s(9.81)}{1+e}$

Eq. (1.14): $e = wG_s = (0.22)(G_s)$. So

$$16.48 = \frac{9.81G_s}{1+0.22}; \quad G_s = 2.67$$

1.3 a. Eq. (1.6): $n = \frac{e}{1+e} = \frac{0.81}{1+0.81} = 0.45$

b. Eqs. (1.7) and (1.14): $S = \frac{wG_s}{e} = \left[\frac{(0.21)(2.68)}{0.81}\right](100) = 69.5\%$

c. Eq. (1.11): $\gamma = \frac{G_s\gamma_w(1+w)}{1+e} = \frac{(2.68)(9.81)(1+0.21)}{1+0.81} = \mathbf{17.58\ kN/m^3}$

d. Eq. (1.12): $\gamma = \frac{G_s\gamma_w}{1+e} = \frac{(2.68)(9.81)}{1+0.81} = \mathbf{14.53\ kN/m^3}$

1.4 a. Eq. (1.11): $\gamma = \frac{G_s\gamma_w(1+w)}{1+e}$

$$122 = \frac{(2.68)(62.4)(1+0.147)}{1+e};\quad e = \mathbf{0.57}$$

b. Eq. (1.6): $n = \frac{e}{1+e} = \frac{0.57}{1+0.57} = \mathbf{0.36}$

c. $S = \frac{wG_s}{e} = \left[\frac{(0.147)(2.68)}{0.57}\right](100) = \mathbf{69.1\%}$

d. From Eqs. (1.11) and (1.12): $\gamma_d = \frac{\gamma}{1+w} = \frac{122}{1+0.147} = \mathbf{106.4\ lb/ft^3}$

1.5 a. Eq. (1.15): $\gamma_{sat} = \frac{G_s\gamma_w + e\gamma_w}{1+e} = \frac{(62.4)(2.68+0.57)}{1+0.57} = \mathbf{129.2\ lb/ft^3}$

b. Water to be added = $\gamma_{sat} - \gamma = 129.2 - 122 = \mathbf{7.2\ lb/ft^3}$

c. $S = \frac{wG_s}{e};\ w = \frac{Se}{G_s} = \frac{(0.8)(0.57)}{2.68} = 0.17$

$$\gamma = \frac{G_s\gamma_w(1+w)}{1+e} = \frac{(2.68)(62.4)(1+0.17)}{1+0.57} = \mathbf{124.6\ lb/ft^3}$$

1.6 From Eqs. (1.11) and (1.12): $\gamma_d = \frac{116.64}{1+0.08} = 108\ \text{lb/ft}^3$

Eq. (1.12): $\gamma_d = \frac{G_s\gamma_w}{1+e};\ 108 = \frac{(2.65)(62.4)}{1+e};\quad e = 0.53$

Eq. (1.19): $D_r = 0.82 = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{e_{max} - 0.53}{e_{max} - 0.44};\quad e_{max} = \mathbf{0.94}$

$$\gamma_{d(\min)} = \frac{G_s \gamma_w}{1+e_{\max}} = \frac{(2.65)(62.4)}{1+0.94} = \mathbf{85.2\ lb/ft^3}$$

1.7 Refer to Table 1.5 for classification.

SOIL A: **A-7-6(9)** (*Note*: PI is greater than LL – 30.)

$$\text{GI} = (F_{200} - 35)[0.2 + 0.005(\text{LL} - 40)] + 0.01(F_{200} - 15)(\text{PI} - 10)$$
$$= (65 - 35)[0.2 + 0.005(42 - 40)] + 0.01(65 - 15)(16 - 10) = 9.3 = \mathbf{9}$$

SOIL B: **A-6(5)**

$$\text{GI} = (55 - 35)[0.2 + 0.005(38 - 40)] + 0.01(55 - 15)(13 - 10) = 5.4 \approx \mathbf{5}$$

SOIL C: **A-3(0)**

SOIL D: **A-4(5)**

$$\text{GI} = (64 - 35)[0.2 + 0.005(35 - 40)] + 0.01(64 - 15)(9 - 10) = 4.585 \approx \mathbf{5}$$

SOIL E: **A-2-6(1)**

$$\text{GI} = (F_{200} - 15)(\text{PI} - 10) = 0.01(33 - 15)(13 - 10) = 0.54 \approx \mathbf{1}$$

SOIL F: **A-7-6(19)** (PI is greater that LL – 30.)

$$\text{GI} = (76 - 35)[0.2 + 0.005(52 - 40)] + 0.01(76 - 15)(24 - 10) = 19.2 \approx \mathbf{19}$$

1.8 *SOIL A*: Table 1.6: 65% passing No. 200 sieve. Fine grained soi.; LL = 42; PI = 16
Figure 1.5: ML
Figure 1.7: Plus No. 200 > 30%; Plus No. 4 = 0
% sand > % gravel – **sandy silt**

SOIL B: Table 1.6: 55% passing No. 200 sieve. Fine grained soil. LL = 38, PI = 13
Figure 1.5: Plots below A-line – ML
Figure 1.7: Plus No. 200 > 30%
% sand > % gravel – **sandy silt**

SOIL C: Table 1.6: 8% passing No. 200 sieve.
% sand > % gravel – sandy soil – SP
Figure 1.6: % gravel = 100 – 95 = 5% < 15% – **poorly graded sand**

SOIL D: Table 1.6: 65% passing No. 200 sieve. Fine grained soil. LL = 35, PI = 9
Figure 1.5 – ML
Plus No. 200 = 36% > 30%
% sand (31%) > % gravel (5%) – **sandy silt**

SOIL E: Table 1.6: 33% passing No. 200 sieve. 100% passing No. 4 sieve. Sandy soil.
LL = 38, PI = 13
Figure 1.5: Plots below A-line – **SM**
Figure 1.6: % gravel (0%) < 15% – **silty sand**

SOIL F: Table 1.6: 76% passing No. 200 sieve. LL = 52, PI = 24
Figure 1.5: **CH**
Figure 1.7: Plus No. 200 is 100 – 76 = 24%
% sand > % gravel – **fat clay with sand**

1.9 $$\gamma_d = \frac{G_s\gamma_w}{1+e}; \quad e = \frac{G_s\gamma_w}{\gamma_d} - 1 = \frac{(2.68)(62.4)}{117} - 1 = 0.43$$

Eq. (1.26):

$$\frac{k_1}{k_2} = \frac{\dfrac{e_1^3}{1+e_1}}{\dfrac{e_2^3}{1+e_2}}; \quad \frac{0.22}{k_2} = \frac{\dfrac{0.63^3}{1+0.63}}{\dfrac{0.43^3}{1+0.43}}$$

$$\boldsymbol{k_2 = 0.08 \text{ cm/sec}}$$

1.10 From Eq. (1.26)

$$k_2 = \left(\frac{e_2^3}{1+e_2}\right)\left(\frac{1+e_1}{e_1^3}\right)k_1$$

$$0.115 = \left(\frac{e_2^3}{1+e_2}\right)\left(\frac{1+0.7}{0.7^3}\right)(0.25) = 1.239\left(\frac{e_2^3}{1+e_2}\right)$$

By trial and error, $e_2 = 0.52$

1.11 $$\frac{k_1}{k_2} = \frac{\dfrac{e_1^{51}}{1+e_1}}{\dfrac{e_2^{51}}{1+e_2}}; \quad \frac{5.4\times10^{-6}}{k_2} = \frac{\dfrac{(0.92)^{51}}{1+0.92}}{\dfrac{(0.72)^{51}}{1+0.72}} = 3.126$$

$$\boldsymbol{k_2 = 1.73 \times 10^{-6} \text{ cm / sec}}$$

1.12 $\gamma_{\text{dry(sand)}} = \dfrac{G_s\gamma_w}{1+e} = \dfrac{(2.66)(62.4)}{1+0.55} = 107.09 \text{ lb / ft}^3$

$\gamma_{\text{sat(sand)}} = \dfrac{G_s\gamma_w + e\gamma_w}{1+e} = \dfrac{(62.4)(2.66+0.48}{1+0.48} = 132.39 \text{ lb / ft}^3$

$\gamma_{\text{sat(clay)}} = \dfrac{G_s\gamma_w(1+w)}{1+wG_s} = \dfrac{(2.74)(62.4)(1+0.3478)}{1+(0.3478)(2.74)} = 117.99 \text{ lb / ft}^3$

At A: $\sigma = \mathbf{0};\ u = \mathbf{0};\ \sigma' = \mathbf{0}$

At B: $\sigma = (107.09)(8) = \mathbf{856.72\ lb / ft^2};\ u = \mathbf{0};\ \sigma' = \mathbf{856.72\ lb / ft^2}$

At C: $\sigma = \sigma_B + (132.39)(4) = 856.72 + 529.56 = \mathbf{1386.28\ lb / ft^2}$
$u = (62.4)(4) = \mathbf{249.6\ lb / ft^2}$
$\sigma' = 1386.28 - 249.6 = \mathbf{1136.68\ lb / ft^2}$

At D: $\sigma = \sigma_C + (117.99)(15) = 1386.28 + 1769.85 = \mathbf{3156.13\ lb / ft^2}$
$u = (62.4)(19) = \mathbf{1185.6\ lb / ft^2}$
$\sigma' = \mathbf{1970.53\ lb / ft^2}$

1.13 In the top sand layer: $y_{\text{sat(sand)}} = \dfrac{\gamma_w(G_s+e)}{1+e} = \dfrac{(62.4)(2.66+0.55)}{1.+0.55} = 129.2 \text{ lb / ft}^3$

When the ground water table is 4 ft below the ground surface,

$\sigma = (107.09)(4) + (129.2)(4) + (132.39)(4) + (117.99)(15) = 3244.57 \text{ lb / ft}^2$
$u = (62.4)(23) = 1435.2 \text{ lb / ft}^2$
$\sigma' = 3244.57 - 1435.2 = 1809.37 \text{ lb / ft}^2$

Change in effective stress $= 1809.37 - 1970.53 = \mathbf{-161.2\ lb / ft^2}$

1.14 Equation (1.37): $i_{cr} = \dfrac{G_s - 1}{1+e}$

$$\left.\begin{aligned} i_{cr} &= \frac{2.65-1}{1+0.42} = \mathbf{1.16} \\ i_{cr} &= \frac{2.65-1}{1+0.85} = \mathbf{0.89} \end{aligned}\right\} \text{Range}$$

1.15 Eq. (1.47): $S_c = \frac{C_c H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}$

$$= \frac{[0.009(42-10)](3.7)}{1+0.82} \log\left(\frac{155}{110}\right) = 0.087 \text{ m} = \mathbf{87 \text{ mm}}$$

1.16 Eq. (1.51): $S = \frac{H_c}{1+e_o}\left[C_s \log\left(\frac{p_c}{p_o}\right) + C_c \log\left(\frac{p_c + \Delta p}{p_o}\right)\right]$

$C_c = 0.009(42 - 10) = 0.288$; $C_s = C_c/5 = 0.0576$

$$S_c = \frac{3.7}{1+0.82}\left[0.0576 \log\left(\frac{128}{110}\right) + 0.288 \log\left(\frac{155}{128}\right)\right] = 0.056 \text{ m} = \mathbf{56 \text{ mm}}$$

1.17 a. Eq. (1.38): $C_c = \frac{e_1 - e_2}{\log_{10}\left(\frac{\sigma'_2}{\sigma'_1}\right)} = \frac{0.91 - 0.792}{\log_{10}\left(\frac{42}{21}\right)} = 0.392$

From Eq.(1.47): $S_c = \frac{C_c H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'}{\sigma'_o}$

Using the results of Problem 1.12

$$\sigma'_o = \overbrace{\overset{Z_{B-A}}{(8)}\overset{\gamma_B}{(107.09)}}^{\sigma'_B} + \overset{\sigma'_C - \sigma_B}{(4)(132.39 - 62.4)} + \overset{(\sigma'_D - \sigma_C) \div 2}{(15/2)(117.99 - 62.4)} = 1553.6 \text{ lb/ft}^2$$

$e_o = wG_s = (0.3478)(2.74) = 0.953$

$$S_c = \frac{(0.392)(15 \times 12)}{1+0.953} \log\left(\frac{1553.6 + 1000}{1553.6}\right) = \mathbf{7.8 \text{ in.}}$$

b. Eq. (1.55): $T_v = \frac{C_v t}{H^2}$. For $U = 50\%$, $T_v = 0.197$ (Figure 1.21)

$0.197 = \frac{1.45 \times 10^{-4} t}{(15 \times 12)^2}$; $t = 4402 \times 10^4$ sec = **509.5 days**

1.18 a. Eq. (1.38): $C_c = \frac{e_2 - e_1}{\log\left(\frac{\sigma'_2}{\sigma'_1}\right)} = \frac{0.82 - 0.64}{\log\left(\frac{360}{120}\right)} = 0.377$

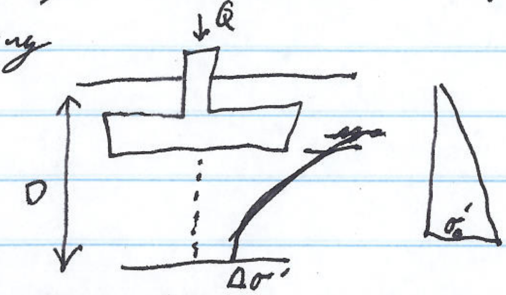
Foundations Aug25 ①

Chap2 2.10 sub soil exploration

- 7 reasons why (read)

Need to know b/f

- Types of structure
- Column loads or slab loads
- Bridges : span , piers , Abutment Loads
- (canals, rivers, etc) Slopes : existing bank slope, proposed cuts, Bank loading
- General idea of Topograph , types of soils
- Reconocense (see 1-6 in textbook, important)
- Site investigation
  - Types of Borings Undisterbed disterbed ⟵ split spoon (SPT) Auger
  - Determining



$q = \frac{Q}{A}$

\# of Borings: 1 Borehole / (Foundation area) 250-650 m^2 or / 2700-7000 ft^2

if less than 1.15 round down ≥1.15 round up

so, $\frac{10,000 \ ft^2 \text{ building foundation}}{7000 \ ft^2 \ \cancel{\text{foundation}}} = 1.4 \therefore 2$ boaring

↳ large b/c have rule of thumb experience in area or know some other borings

Depth of Borings: $D_1 = \frac{\Delta\sigma}{q} = 0.01$

shallow foundations ⇅

$D_2 = \frac{\Delta\sigma}{\sigma_o'} = 0.05$

} ~~max~~ min of two = min D for Boring (can go more)

$D = 10 \cdot S^{0.7}$ (lightly loaded / Narrow structure) $D = 20 \cdot S^{0.7}$ (heavy loaded / wide building) { S = # of stories

≤ 250 psf lightly loaded in N.O. >= heavy

≤ 1000 psf lightly loaded rest of country > = heavy

Foundations Aug 25 ②

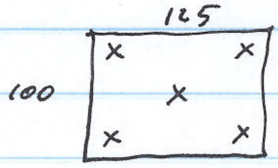
Ex. 5 story Building, concrete, 100' × 125', No exp. at site

a) How many borings b) depth of borings

Soln: a) $125 \times 100 = 12500 \text{ ft}^2$ (Foundation area)

$\# \text{ borings} = \frac{12500}{2700} = 4.6 = \boxed{5 \text{ borings}}$

b) $D = 20(\text{wide building})\, S^{0.7} = 20(5^{0.7}) = 61.7' \approx \boxed{65'}$



New Orleans Mod.

- will likely be on deep piles so should be deeper boring

1 story ~ 60' (lightly loaded)

2-3 " ~ 100'

3-4 " ~ 150'

5+ " ~ 200'

changes may be made for: - Experience - Structure loading

Exploration should cost 0.1 - 0.5% of Structure Cost

Types of Borings

- Auger: - Hand held equip. < Post hole Auger, helical Auger
  - Power Auger < continuous flight, Hollow stem

  } limitation ~15' due to hole calapse
- Wash Borings (not used much in US)
- Rotary Drilling (most common other than Auger)
  - Various Drilling bits, Water or Drilling Mud, cuttings come back to surface

Foundations Aug 25 ③

Disterbed Samples

- SPT - Standard Penetration Test (split spoon)
- Auger
- Uses: Grain size, LL & PL, S.G., organic content, moisture %, classification

SPT - 18" long, Driven by 140# hammer w/ 30" Fall height

- 1st 6" called 'seat', next 12" is ~~N field value~~ $N_{\text{field value}}$ (~~in field value~~ 2. ~~en site~~ ?)

Formula to calc, if Sample is Disterbed

outside Dia. inside Dia.

- $\text{Area \%} = \frac{D_o^2 - D_i^2}{D_i^2}(100)$ when Area % is 10% or less = Undisterbe

Factors the effect N value

- hammer efficiency, borehole Dia., Sampling rod length

$$E_r(\%) = \frac{\text{actual hammer energy to sampler}}{\text{input energy}}(100) = wh$$

w = hammer weight, h = drop height

- E_r can vary from 30% to 90%, usually corrolate to $N_{60} = 60\%$

$$N_{60} = N(n_h)(n_B)(n_s)(n_R)\left(\frac{1}{60}\right)$$

N = Field Vaue, n_h = hammer eff., n_B = correction for bore hole Dia,

n_s = correction for Sample, n_R = Correction for Rod length

Foundations Aug 25 ④

Cohesive Soils - correlate N value to consistency, unconfined compressive strength, consistancy index ($CI = \frac{LL - w}{LL - PI}$

*** Cohesion is $\frac{1}{2}$ of unconfined Compresion strength

For Cohesionless Soils - Nvalue can be corrolated to relative density (D_r) ; $(N_1)_{60} = C_n N_{60}$ ← From Field

↖ correction for overburden

$$D_r \% = 11.7 + 0.76(222 N_{60} + 1600 - 53 \sigma_0' - 50 C_u^2)^{0.5}$$

← uniformity Coefficient

N_{value} can be corrolated to ϕ (most common)

$$\phi = \tan^{-1}\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma_0'}{P_a}\right)}\right]^{0.34}$$

← usually can assume standard atm. press.

Thin Wall tube Sampling (Shelby Tube) or (Piston sampler) — 2 types

- Undisterbed

usually better but takes longer

N.O. - 3" & 5" dia. (5" less disterbed)

Water Table Observation

- take after 24 hrs after boring
- If varying water tables or don't want to wait use piezometer

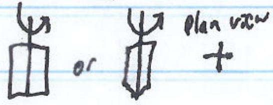
all en situ tests

all not used often except CPT, it is used alot

Vane Shear Test

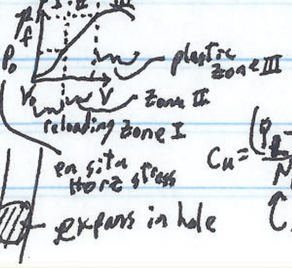
- soft clays
- τ applied @ 0.1%/sec

$C_u = \frac{\tau}{k}$



Pressuremeter

- take inital Vol. (V_0)



$C_u = \frac{(P_f - P_0)}{N_p}$

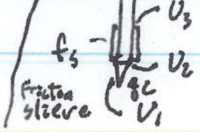
N_p ~ 5 to 12

Dilatometer

1. Press. A - lift or expand membrane
2. Press B - press req. for membrane to expand 1.1mm or 0.4"

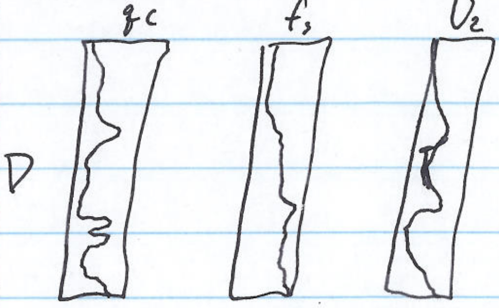
CPT (Cone Penetroniter Test)

Types of Cones:	- 2.5 TON	0-300
	- 5 TON	300-1000
	- 10 TON	>1000



Found. Aug 25 ⑤

CPT con't



from this can get Friction ratio = $\frac{f_s}{q_c}$

$$\phi = \tan^{-1}\left[0.1 + 0.35 \log_e\left(\frac{q_c}{\sigma_o'}\right)\right]$$

$C_u = \frac{q_c - \sigma_o}{N_k}$, N_k = bearing Capacity Factor ~ 15

σ_o' = effective stress , σ_o = total stress

Rock Coring

- Single tube & Double barrel
- Recovery Ratio = $\frac{\text{Length Recovery}}{\text{theor. Length}}$
- Rock Quality Designation = $\frac{\Sigma \text{ Recovery Lenght}}{\text{theor. Length}}$

Read Boring Logs ~ p. 105

Foundation 9-1-10 ③

chap 3 **Footings**
- Square
- Strip or Continuous – Length >> width
- Circular
- Rectangular

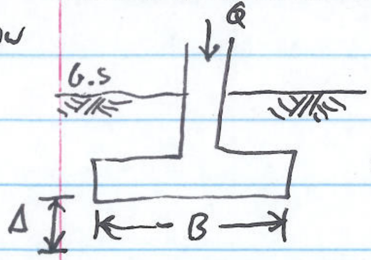
Shallow Foundation $D_f \le B$

Deep $D_f > B$

D_f = Depth of Foundation
B = footing size

---

shallow



γ, c, ϕ

---

ch3 p.134 see diagrams

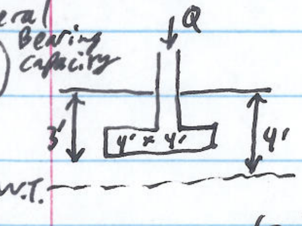
General Shear   Local Shear   Punching Shear

3.3 Terzaghi's Bearing Capacity Theory p. 137 → 145

* see notes in Book

General Bearing capacity

Ex.



$\gamma_t = 115psf$

$\gamma_{sat} = 122.4psf$

$c = 100 \text{ psf}$

$\phi = 30°$

* Find gross allow. soil Bearing Value

$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma') = 60 + \frac{1}{4}(115-60)$
$= 73.75psf$

$\gamma' = \gamma_{sat} - \gamma_w = 122.4 - 62.4$
$= 60psf$

Sol'n: From Table (3.3): $N_c = 30.14$, $N_q = 18.40$, $N_\gamma = 22.40$

$\frac{N_q}{N_c} = 0.61$, $\tan\phi = 0.58$

plug into eq. 3.19

$q_u = 20463.7$

$q_{all} = \frac{q_u}{3}$
$= 6821psf$

p.145

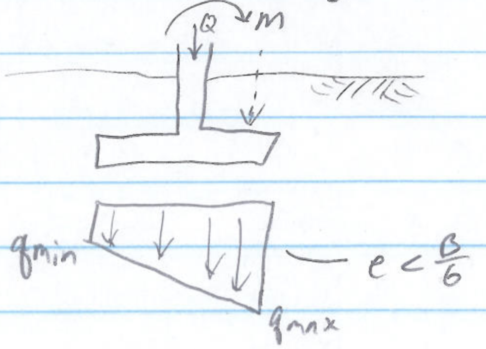
SHAPE FACTORS	Depth Factors	Inclination Factor
$F_{cs} = 1 + 1(0.61) = 1.61$	$F_{cd} = 1 + (0.4)\frac{D_f}{B} = 1.3$	$F_{ci} = F_{qi} = 1$
$F_{qs} = 1 + 1(0.58) = 1.58$	$F_{qd} = 1.27$	$F_{\gamma i} = 1$
$F_{\gamma s} = 1 - (0.4)(1) = 0.60$	$F_{\gamma d} = 1$	$q = \gamma D_f = 115(3) = 345psf$

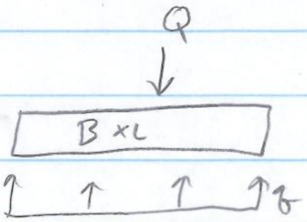
$q_{all\ net}$
$= 6206ps$

https://sites.google.com/site/
geomechanique/

Foundations Sep 8 ①

Excentric Loading



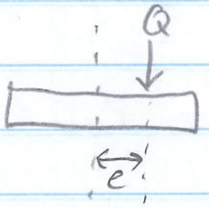


$q = \frac{Q}{B \times L}$

p. 157

$q\,max = \frac{Q}{B \times L} + \frac{6M}{B^2 L}$

$q\,min = \frac{Q}{BL} - \frac{6M}{B^2 L}$



$M = Q \cdot e$

$y = \frac{B}{2}$

$I = \frac{1}{12} L B^3$

$$\sigma = \frac{Q}{A} \pm \frac{My}{I} = \frac{Q}{BL} \pm \frac{QeB12}{2LB^3}$$

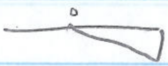
$$= \frac{Q}{BL} \pm \frac{6Qe}{B^2 L}$$

$$= \frac{Q}{BL}\left(1 \pm \frac{6e}{B}\right)$$

$q\,max = \frac{Q}{BL}\left(1 + \frac{6e}{B}\right) \quad \left\{e < \frac{B}{6}\right\}$

$q\,min = \frac{Q}{BL}\left(1 - \frac{6e}{B}\right. \quad \left\{e = \frac{B}{6}\right\}$

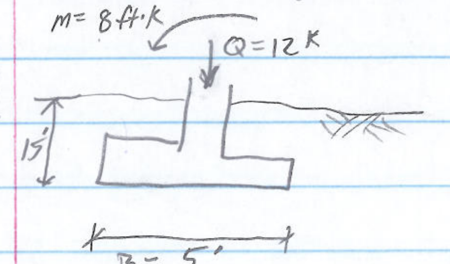
$e = \frac{B}{6}$



$e > \frac{B}{6}$

Foundations Sep 8 ②

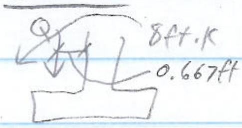
Ex:



Find q_{max}, q_{min}, strip footing

Assume L=1

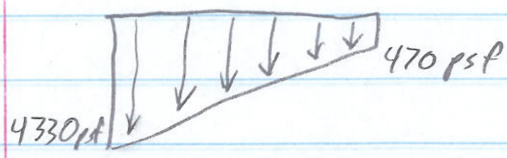
Soln: $e = \frac{m}{Q} = \frac{8ft\text{-}k}{12k} = 0.667\ ft$



$\frac{B}{6} = \frac{5}{6} = 0.833ft > e$ okay

$$q_{max} = \frac{12^k}{5'(1')}\left(1 + \frac{6(0.67)}{5}\right) = 4330\ psf$$

$$q_{min} = \frac{12k}{5'(1')}\left(1 - \frac{6(0.67)}{5}\right) = 470\ psf$$

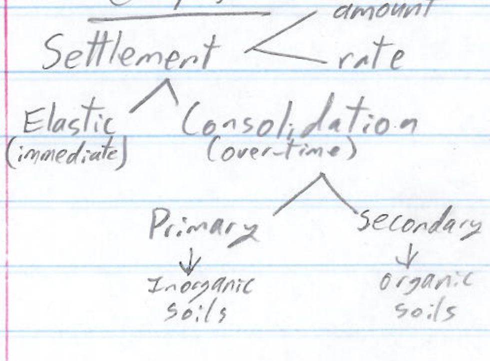


See notes p.159, 165

Foundations Sep 15 ①

Chap 5

P.223



1) Calc Vertical stress
2) " Elastic Settlement
3) " Consolidation settlement

p.224 – P.230

P.245 – 252 Theory of Elasticity

P. 243 – 245 Elastic Settlement for Sat. Clay

Ex. $D_f = 5'$, $H = 35'$, $B = 10\ ft^2$, $Q = 500k$

material type = medium clay

-get table for soil type → recommended modulus of Elas..

use $E_s = 4500\ lb/in^2$ $\quad$ $S_e = A_1 A_2 \frac{q_0 B}{E_s}$

soln

$$q_0 = \frac{Q}{A} = \frac{500k}{10'(10')} \left(\frac{1000 lb}{k}\right)\left(\frac{1 ft^2}{144 in^2}\right) = 34.72\ lb/in^2$$

see Fig 5.14 $\quad \frac{H}{B} = \frac{35}{10} = 3.5$, $\frac{L}{B} = \frac{10}{10} = 1$, $\frac{D_f}{B} = \frac{5}{10} = 0.5$

↳ $A_1 = 0.62$, $A_2 = 0.95$

$$\therefore S_e = 0.62(0.95)\frac{34.72(10')\left(\frac{12''}{ft}\right)}{4500} = 0.545\ in$$

p. 258-263 Sandy Soil, Strain Influence Factor metod

P. 260 ☆ Fig 5.22 & steps

p. 263-64 5.13 settlement of sand based on Standard Penetration ~~Test~~ Resistance

Ex $\quad 1.75m \times 1.75m$, $D_f = 1m$, $N_{60} = 10$, $q_{net} = 120\ \frac{kN}{m^2}$

$F_d = 1 + 0.33\left(\frac{1}{1.75}\right) = 1.189$

b/c 1.75 "B" > 1.22 use equ 5.66

$$S_e = \frac{2(120)}{10(1.189)}\left(\frac{1.75}{1.75+0.3}\right)^2 = 14.7mm$$

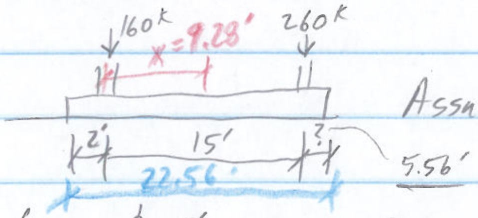
Found. Sep.15 ②

P. 273-74 Primary Consolidation Settlement

P. 279^{8}5.17 Secondary Consolidation

Found. Sep. 29 ①

Ex Matt



Assume $\frac{L}{B} \sim 5$

$\gamma = 120$ psf $\phi = 0$ $c = 1700$ psf ($\therefore$ clay soil)

Use general bearing capacity equ. to find dimensione of Footing

Sol'n:

(1) $A = \frac{Q_1 + Q_2}{q_{net}}$ $\overset{1700\text{psf}}{q_{\underset{clay}{ult\,net}} = C N_c F_{ci} F_{cd} F_{cs}}$ $\therefore$ simplified $N_c = 5.14$, $F_{cs} = 1 + \frac{B}{L}(.2) = 1.04$ $F_{cd} = 1$ $F_{ci} = 1$

$q_{\underset{net}{ult}} = 9087$ psf $= 9.087$ ksf $q_{allow} = \frac{9.08\text{ ksf}}{3} = 3$ ksf

$A = \frac{160^k + 260^k}{3\text{ ksf}} = 140\text{ ft}^2 = B(L) \rightarrow B = \frac{140\text{ft}^2}{L}$

(2) Moment Arm $X = \frac{Q_2 L_3}{Q_1 + Q_2} = \frac{260(15')}{160+260} = 9.28'$

(3) $L = 2(L_2 + X) = 2(2' + 9.28') = 22.56'$

(4) $L_1 = L - L_2 - L_3 = 22.56 - 2 - 15 = 5.56'$

(5) determine width $B = \frac{140}{L} = \frac{140}{22.56} = 6.2'$

USE $B = 6.5'$, $L = 23'$

Exp. 6.4 (old book) (similar to new book)

Stopped @ p. 304 not covering rest of ch. 6

①

Chapter 5

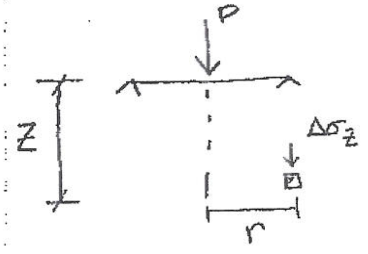
9/14/06

Settlement
- amount
- rate

Settlement
- Elastic or immediate
- consolidation
  - Primary → Inorganic soils
  - Secondary → organic soils

1. Calculate Vertical Stress increase
2. Calculate Elastic Settlement
3. Calculate Consolidation settlement.

Concentrated or Point load ⇒ Boussinesq's eqn.

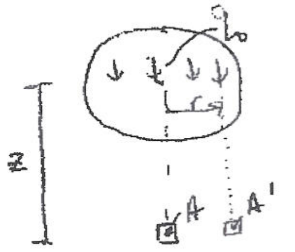


$$\Delta\sigma_z = \frac{3P}{2\pi z^2}\left\{\frac{1}{[1+(\frac{r}{z})]^{5/2}}\right\}$$

$$r = \sqrt{x^2+y^2}$$

Load on a circular radius R

q_0 = uniform pressure
Radius = $B/2$



$$\Delta\sigma_{z_A} = q_0\left\{1 - \frac{1}{[1+(\frac{B}{2z})^2]^{3/2}}\right\}$$ – at center of Circular Area

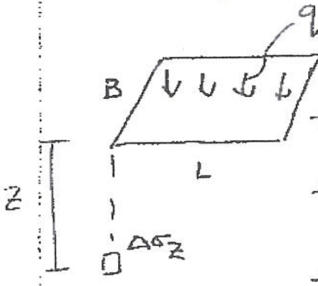
$$\Delta\sigma_{z_{A'}} = \Delta\sigma_{z_A}(\Delta\sigma/q_0 \text{ Factor})$$

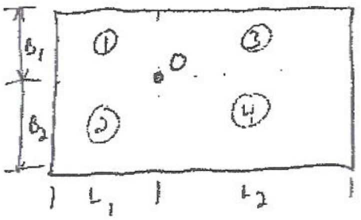
Rectangular Area

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

Use Table 5.2

$$\Delta\sigma = q_0(I_1+I_2+I_3+I_4)$$





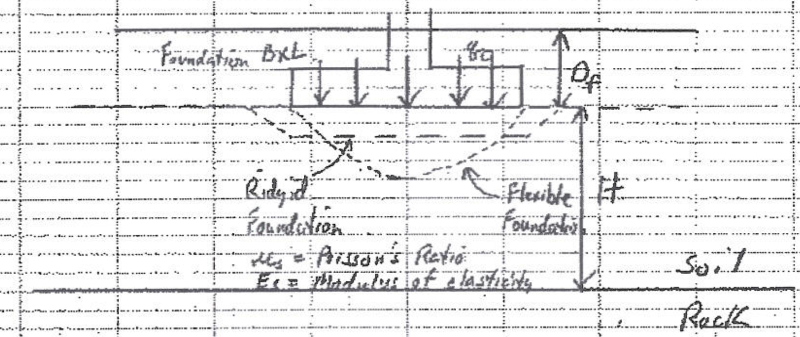
Center Stress

$\Delta\sigma_z = q_0 I_c$

$m_1 = \frac{L}{B}$ $n_1 = \frac{z}{(\frac{B}{2})}$

Use table 5.3

Elastic Settlement based on Theory of Elasticity



For Flexible Foundation

$$S_e = q_o(\alpha B')\frac{1-\mu_s^2}{E_s} I_s I_f$$

$$I_s = F_1 + \frac{1-2\mu_s}{1-\mu_s} F_2$$

$F_1 = \frac{1}{\pi}(A_0 + A_1)$ Table 5.4

$F_2 = \frac{n'}{2\pi}\tan^{-1} A_2$ Table 5.5

$$A_0 = m' \ln \frac{(1+\sqrt{m'^2+1})\sqrt{m'^2+n'^2}}{m'(1+\sqrt{m'^2+n'^2+1})}$$

$$A_1 = \ln \frac{(m'+\sqrt{m'^2+1})(\sqrt{1+n'^2})}{m'+\sqrt{m'^2+n'^2+1}}$$

$$A_2 = \frac{m'}{n'\sqrt{m'^2+n'^2+1}}$$

S_e = elastic settlement

E_s = modulus of elasticity $z = 0$ to $z = 4B$

μ_s = Poisson's ratio

$B' = B/2$ for center of foundation

$= B$ for center of foundation

I_s = shape factor

I_f = depth factor - Fig 5.15

α = factor depending on location of the foundation where settlement is calculated.

To calculate settlement at the center

$\alpha = 4$
$m' = L/B$
$n' = H/(B/2)$

To calculate settlement at the corner

$\alpha = 1$
$m' = L/B$
$n' = H/B$

For Rigid Foundation

$$S_{e(rigid)} = 0.93\, S_{e(flexible)}$$

2/5/98

Elastic Settlement
Christian + Carrier Method – Saturated Clay ☆
(Janbu's Method)

Example. $D_f = 5$ ft — Medium Clay

$H = 35$ ft

$B = 10$ ft (square)

$Q = 500^K$

$$q = \frac{Q}{A} = \frac{500^K}{10 \times 10}\left(\frac{1000\,lb}{1k}\right)\left(\frac{1\,ft^2}{144\,in^2}\right) =$$

$$q = 34.72\,lb/in^2$$

Find the immediate settlement

$$S_e = A_1 A_2 \frac{q_0 B}{E_s}$$

$L/B = 1 \quad H/B = \frac{35}{10} = 3.5$

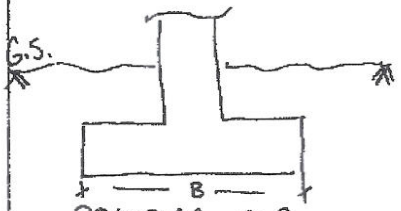
$D_f/B = 5/10 = .5$

From Figure 3.28		From Table 3.9
$A_2 = .95$	$A_1 = .62$	$E_s = 4500\ lb/in^2$

$$S_e = (.62)(.95)\frac{34.72\,(120\,in)}{4500\,lb/in^2} = .545\,in$$

⑤

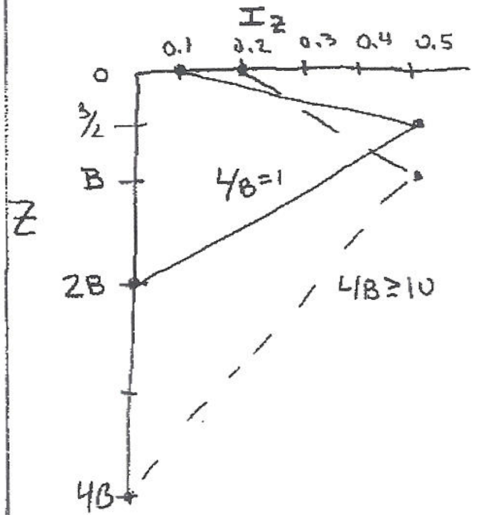
Schmertmann's strain influence factor method for sands ✱



Square or circular foundation
↓
(Axisymmetric case)

Rectangular or Long foundations
↓
(Plane-strain)

p.236 (Fig5.22)



L/B = 1 → Square or circular

L/B ≥ 10 → Rect. or long

$$S_e = C_1 C_2 (\bar{q} - q) \sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z$$

I_z = strain influence factor

C_1 = correction factor for depth foundation embedment

C_2 = correction factor for creep in soil

$\bar{q}$ = stress at the level of the foundation

$q = \gamma D_f$

$C_1 = 1 - 0.5 [q/(\bar{q} - q)]$

$C_2 = 1 + 0.2 \log\left(\frac{\text{time in years}}{0.1}\right)$

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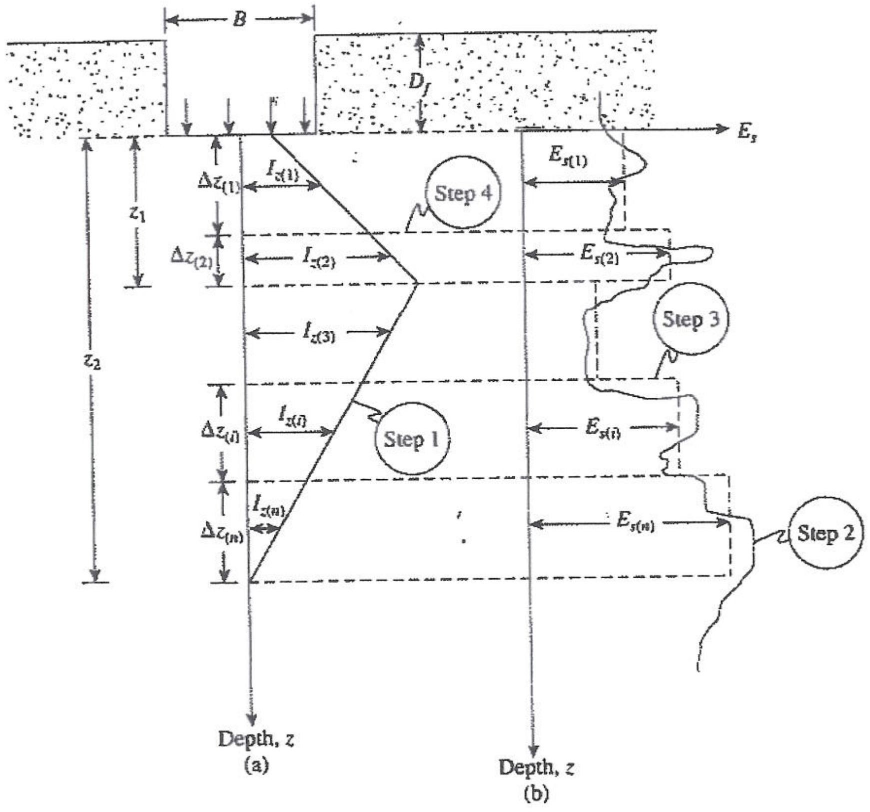


Figure 5.23 Procedure for calculation of S_e using the strain influence factor

Step 1. Plot the foundation and the variation of I_z with depth to scale (Figure 5.23a).

Step 2. Using the correlation from standard penetration resistance (N_{60}) or cone penetration resistance (q_c), plot the actual variation of E_s with depth (Figure 5.23b). Schmertmann et al. (1978) suggested $E_s \approx 3.5q_c$.

Step 3. Approximate the actual variation of E_s into a number of layers of soil having a constant E_s, such as $E_{s(1)}, E_{s(2)}, \ldots, E_{s(i)} \ldots E_{s(n)}$ (Figure 5.23b).

Step 4. Divide the soil layer from $z = 0$ to $z = z_2$ into a number of layers by drawing horizontal lines. The number of layers will depend on the break in continuity in the I_z and E_s diagrams.

Step 5. Prepare a table (such as Table 5.6) to obtain $\Sigma \frac{I_z}{E_s} \Delta z$.

Step 6. Calculate C_1 and C_2.

Step 7. Calculate S_e from Eq. (5.42).

Settlement of Foundation on Sand Based on Standard Penetration Resistance – Meyerhof's Method

Correlation of settlement to q_{net}

$$S_{e\,(in)} = \frac{2.5\, q_{net}\,(K/ft^2)}{N_{60}\, F_d} \quad \text{For } B \leq 4'$$

English

$$S_{e\,(in)} = \frac{4\, q_{net}\,(K/ft^2)}{N_{60}\, F_d}\left(\frac{B}{B+1}\right)^2 \quad \text{for } B > 4'$$

$$S_{e\,(in)} = \frac{1.25\, q_{net}\,(kN/m^2)}{N_{60}\, F_d} \quad \text{for } B \leq 1.22m$$

Metric

$$S_{e\,(in)} = \frac{2\, q_{net}\,(kN/m^2)}{N_{60}\, F_d}\left(\frac{B}{B+0.3}\right)^2 \quad \text{for } B > 1.22m$$

B in meters
S_e in mm

F_d = depth Factor = $1 + 0.33\,(D_f/B)$ Figure 5.25

B = foundation width in feet
S_e = settlement in inches

Example: 1.75m x 1.75m
D_f = 1m
N_{60} = 10
q_{net} = 120 kN/m2
S_e = ?

$$F_d = 1 + 0.33\left(1/1.75\right) = 1.189$$

$$S_e = \frac{2\,(120\ kN/m^2)}{10\,(1.189)}\left(\frac{1.75m}{1.75m + 0.3}\right)^2$$

$$S_e = 14.7\ mm$$

Primary Consolidation Settlement

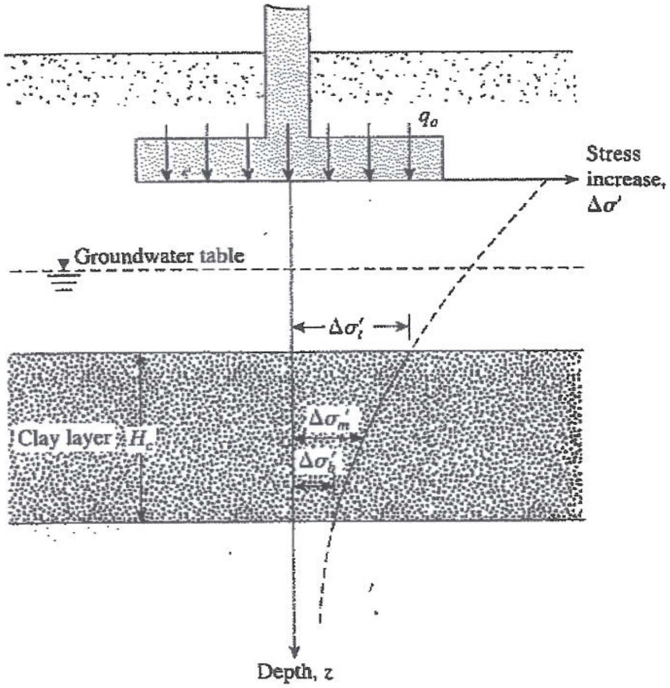


Figure 5.32 Consolidation settlement calculation

So,

$$S_{c(p)} = \frac{C_c H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{\text{av}}}{\sigma'_o} \quad \text{(for normally consolidated clays)} \quad \text{[Eq. (1.47)]}$$

$$S_{c(p)} = \frac{C_s H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{\text{av}}}{\sigma'_o} \quad \text{(for overconsolidated clays with } \sigma'_o + \Delta\sigma'_{\text{av}} < \sigma'_c\text{)} \quad \text{[Eq. (1.49)]}$$

$$S_{c(p)} = \frac{C_s H_c}{1+e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{\text{av}}}{\sigma'_c} \quad \text{(for overconsolidated clays with } \sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'_{\text{av}}\text{)} \quad \text{[Eq. (1.51)]}$$

where

σ'_o = average effective pressure on the clay layer before the construction of the foundation
$\Delta\sigma'_{\text{av}}$ = average increase in effective pressure on the clay layer caused by the construction of the foundation
σ'_c = preconsolidation pressure
e_o = initial void ratio of the clay layer
C_c = compression index
C_s = swelling index
H_c = thickness of the clay layer

The procedures for determining the compression and swelling indexes were discussed in Chapter 1.

Note that the increase in effective pressure, $\Delta\sigma'$, on the clay layer is not constant with depth: The magnitude of $\Delta\sigma'$ will decrease with the increase in depth measured from the bottom of the foundation. However, the average increase in pressure may be approximated by

$$\Delta\sigma'_{av} = \tfrac{1}{6}(\Delta\sigma'_t + 4\Delta\sigma'_m + \Delta\sigma'_b) \tag{5.70}$$

where $\Delta\sigma'_t$, $\Delta\sigma'_m$, and $\Delta\sigma'_b$ are, respectively, the effective pressure increases at the *top, middle,* and *bottom* of the clay layer that are caused by the construction of the foundation.

The method of determining the pressure increase caused by various types of foundation load is discussed in Sections 5.2 through 5.6. $\Delta\sigma'_{av}$ can also be directly obtained from the method presented in Section 5.5.

5.16 Three-Dimensional Effect on Primary Consolidation Settlement

The consolidation settlement calculation presented in the preceding section is based on Eqs. (1.47), (1.49), and (1.51). These equations, as shown in Chapter 1, are in turn based on one-dimensional laboratory consolidation tests. The underlying assumption is that the increase in pore water pressure, Δu, immediately after application of the load equals the increase in stress, $\Delta\sigma$, at any depth. In this case,

$$S_{c(p)-\text{oed}} = \int \frac{\Delta e}{1 + e_o} dz = \int m_v \Delta\sigma'_{(1)} dz$$

where

$S_{c(p)-\text{oed}}$ = consolidated settlement calculated by using Eqs. (1.47), (1.49), and (1.51)

$\Delta\sigma'_{(1)}$ = effective vertical stress increase

m_v = volume coefficient of compressibility (see Chapter 1)

In the field, however, when a load is applied over a limited area on the ground surface, such an assumption will not be correct. Consider the case of a circular foundation on a clay layer, as shown in Figure 5.33. The vertical and the horizontal stress increases at a point in the layer immediately below the center of the foundation are $\Delta\sigma_{(1)}$ and $\Delta\sigma_{(3)}$, respectively. For a saturated clay, the pore water pressure increase at that depth (see Chapter 1) is

$$\Delta u = \Delta\sigma_{(3)} + A[\Delta\sigma_{(1)} - \Delta\sigma_{(3)}] \tag{5.71}$$

where A = pore water pressure parameter. For this case,

$$S_{c(p)} = \int m_v \Delta u \, dz = \int (m_v)\{\Delta\sigma_{(3)} + A[\Delta\sigma_{(1)} - \Delta\sigma_{(3)}]\} \, dz \tag{5.72}$$

Now assuming that the 2:1 method of stress increase (see Figure 5.5) holds good, the area of distribution of stress at the top of the clay layer will have dimensions

$$B' = \text{width} = B + z = 1 + (1.5 + 0.5) = 3 \text{ m}$$

and

$$L' = \text{width} = L + z = 2 + (1.5 + 0.5) = 4 \text{ m}$$

The diameter of an equivalent circular area, B_{eq}, can be given as

$$\frac{\pi}{4} B_{eq}^2 = B'L'$$

so that

$$B_{eq} = \sqrt{\frac{4B'L'}{\pi}} = \sqrt{\frac{(4)(3)(4)}{\pi}} = 3.91$$

Also,

$$\frac{H_c}{B_{eq}} = \frac{2.5}{3.91} = 0.64$$

From Figure 5.34, for $A = 0.6$ and $H_c/B_{eq} = 0.64$, the magnitude of $K_\alpha \approx 0.78$. Hence,

$$S_{e(p)} = K_\alpha S_{e(p)-\text{oed}} = (0.78)(46.5) \approx 36.3 \text{ mm}$$ ■

5.17 Settlement Due to Secondary Consolidation

At the end of primary consolidation (i.e., after the complete dissipation of excess pore water pressure) some settlement is observed that is due to the plastic adjustment of soil fabrics. This stage of consolidation is called *secondary consolidation*. A plot of deformation against the logarithm of time during secondary consolidation is practically linear as shown in Figure 5.36. From the figure, the secondary compression index can be defined as

$$C_\alpha = \frac{\Delta e}{\log t_2 - \log t_1} = \frac{\Delta e}{\log (t_2/t_1)} \tag{5.77}$$

where

C_α = secondary compression index
Δe = change of void ratio
t_1, t_2 = time

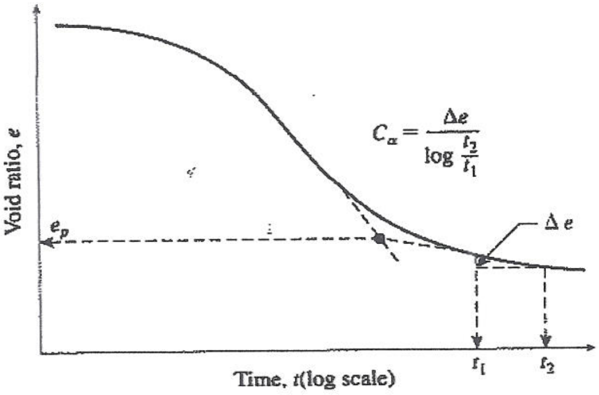


Figure 5.36 Variation of e with log t under a given load increment, and definition of secondary compression index

The magnitude of the secondary consolidation can be calculated as

$$S_{c(s)} = C'_\alpha H_c \log(t_2/t_1) \tag{5.78}$$

where

$$C'_\alpha = C_\alpha/(1 + e_p) \tag{5.79}$$

e_p = void ratio at the end of primary consolidation
H_c = thickness of clay layer

Mesri (1973) correlated C'_α with the natural moisture content (w) of several soils, from which it appears that

$$C'_\alpha \approx 0.0001w \tag{5.80}$$

where w = natural moisture content, in percent. For most overconsolidated soils, C'_α varies between 0.0005 to 0.001.

Mesri and Godlewski (1977) compiled the magnitude of C_α/C_c (C_c = compression index) for a number of soils. Based on their compilation, it can be summarized that

- For inorganic clays and silts:

$$C_\alpha/C_c \approx 0.04 \pm 0.01$$

- For organic clays and silts:

$$C_\alpha/C_c \approx 0.05 \pm 0.01$$

- For peats:

$$C_\alpha/C_c \approx 0.075 \pm 0.01$$

Secondary consolidation settlement is more important in the case of all organic and highly compressible inorganic soils. In overconsolidated inorganic clays, the secondary compression index is very small and of less practical significance.

Combined Footings

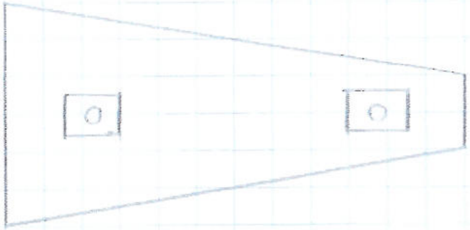
In certain cases, a footing may be more desirable (economical) to support two or more columns under one footing. These footings are called combined footings.

Three general classifications of combined footings.

1) Rectangular combined footing

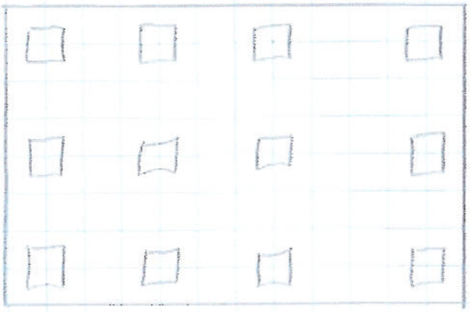
– Rectangular Footing

2) Trapezoidal Combined footing



– Trapezoidal Footing

3) Mat or Raft Footing

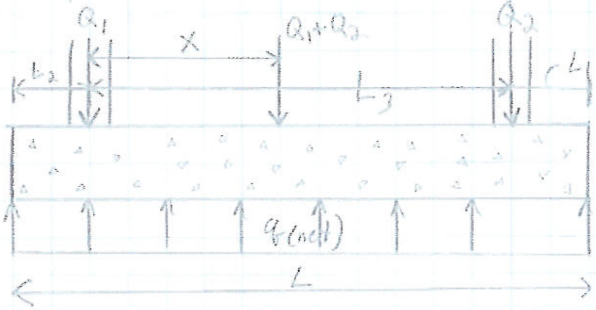


– Mat Foundation

Rectangular Combined Footing

Calculate the net allowable soil bearing value for a Rectangular Footing

- Determine the size of the footing



1) Area of Foundation

$$A = \frac{Q_1 + Q_2}{q_{(net)}}$$

A = area of Foundation
$Q_1 + Q_2$ = Column Loads
$q_{(net)}$ = net allowable soil bearing capacity

2) Determine the location of the resultant column load,

$$X = \frac{Q_2 L_3}{Q_1 + Q_2}$$

3) For Uniform pressure distribution under Foundation the Resultant load passes through centroid

$$L = 2(L_2 + X)$$ L_2 is known distance based on structure design

4) Once Length is determined, L_1 can be obtained

$$L_1 = L - L_2 - L_3$$

5) The width of the foundation is then

$$B = \frac{A}{L}$$

Bearing Capacity of Mat Foundation

From General Bearing Capacity equation

$$q_u = c'N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

Use Chapter 3 for proper shape, depth, + inclination factors

where B is smallest dimension of the mat.

$$q_{net} = q_u - q$$

For Clays w/ $\phi = 0$ $\quad q_u = c_u N_c F_{cs} F_{cd} + q$

- $N_c = 5.14, N_q = 1, N_\gamma = 0$

$$\therefore \quad q_u = 5.14 c_u \left(1 + \frac{0.195B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right) + q$$

Net Bearing Capacity

$$q_{net} = q_u - q = 5.14c\left(1 + \frac{0.195B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right)$$

Trapezoidal Combined

- Trapezoidal Footings are sometimes used where columns are carrying large loads where space is tight.

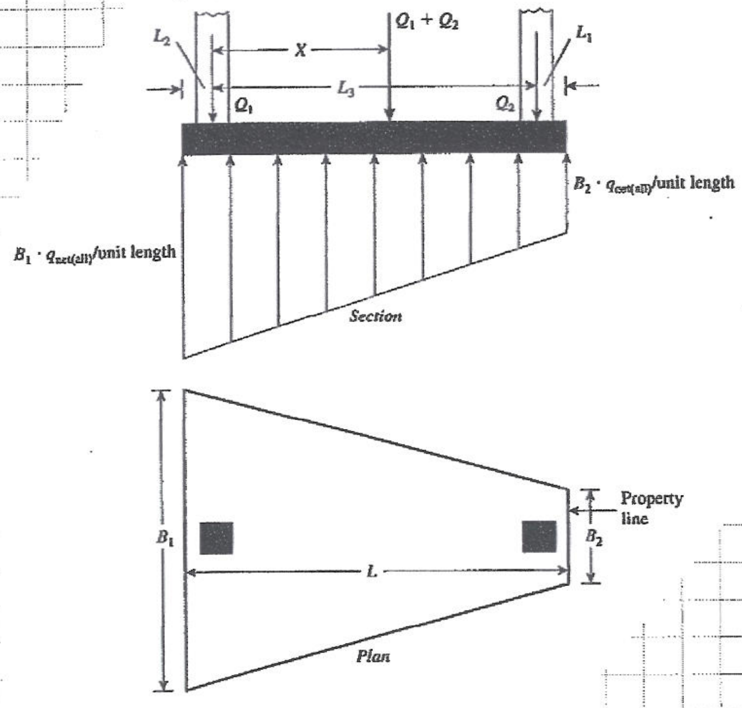


Figure 6.2 Trapezoidal combined footing

1) Determine net allowable soil bearing value

- Then determine the Area of foundation

$$A = \frac{Q_1 + Q_2}{q_{net}} \quad \text{where } A = \frac{B_1 + B_2}{2} L$$

2) Determine the location of the resultant.

$$X = \frac{Q_2 L_3}{Q_1 + Q_2}$$

3) From Property of trapezoid

$$X + L_2 = \left(\frac{B_1 + 2B_2}{B_1 + B_2}\right)\frac{L}{3}$$

With known values of A, L, X and L_2 solve for B_1 + B_2

- For trapezoid $\frac{L}{3} < X + L_2 < \frac{L}{2}$

Mat Foundations

Mat Foundations are sometimes preferred for soils with low bearing capacity

Various types of Mat Foundations - Can be used with or without piles,

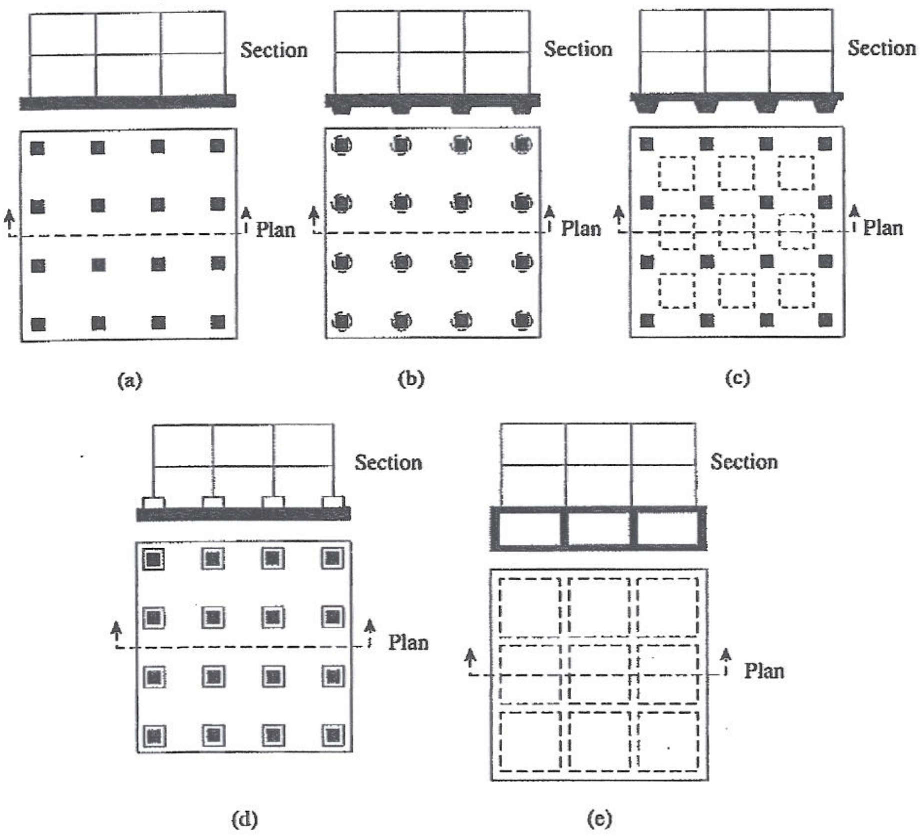
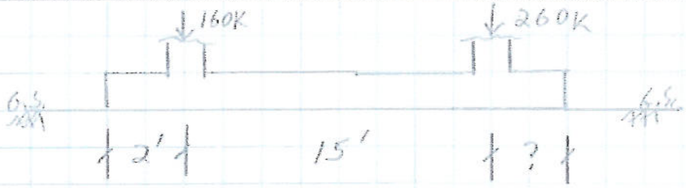


Figure 6.4 Common types of mat foundation

Ex)



Assum $\frac{L}{B} \approx 5$

Soil Properties below mat: $\gamma = 120pcf$; $c = 1700psf$
$\phi = 0$; $F.S. = 3$

- Use General bearing capacity equation to find footing dimensions of the combined rectangular foundation

1) Area of Foundation

$$A = \frac{Q_1 + Q_2}{q_{net}} \qquad Q_1 = 160 Kip \quad Q_2 = 260 Kip$$

$q_{ult} = c N_c F_{cs} F_{cd} F_{ci}$ $\qquad N_c = 5.14$

$= 1700psf(5.14)(1.04)(1)(1)$ $\qquad F_{cs} = 1 + \frac{B}{L} .2$

$\qquad F_{cs} = 1 + .2(.2) = 1.04$

$= 9,087psf = 9.08Ksf$ $\qquad F_{cd} = 1, F_{ci} = 1$

$$q_{all} = \frac{9087psf}{F.S. = 3} = 3,029psf = 3Ksf \quad \therefore A = \frac{160 + 260}{3ksf} = 140ft^2$$

$$\therefore \quad B \times L = \frac{160Kip + 260 kip}{3 Ksf} = B \times L = 140 ft^2 \text{ or } B = \frac{140ft^2}{L}$$

2) Determine Location of the resultant column load.

$$X = \frac{Q_2 L_3}{Q_1 + Q_2} = \frac{260K(15')}{160K + 260K} = 9.28'$$

3) For Uniform Pressure - Determine L

$$L = 2(L_2 + X) = 2(2' + 9.28') = 22.56'$$

4) Determine L_1

$$L_1 = L - L_2 - L_3 = 22.56' - 2' - 15' = 5.56'$$

5) Determine width of Foundation

$$B = \frac{A}{L} \text{ or } B = \frac{140ft}{L} = \frac{140ft}{22.56'} = 6.2'$$

$\therefore$ Use $L = 23'$
+ $B = 6.5'$

5.19

Depth (ft)	N_{60}
5	10
10	12
15	9
20	14
25	16

Average $N_{60} \approx 12$

From Eq. (5.46b):

$$\text{Allowable } q_{\text{net}} = \frac{N_{60}}{4}\left(\frac{B+1}{B}\right)^2 F_d S_e$$

$B = 5$ ft; $S_e = 1$ in.

$F_d = 1 + 0.33(D_f/B) = 1 + (0.33)(3/5) = 1.198$

$$q_{\text{net}} = \frac{12}{4}\left(\frac{5+1}{5}\right)^2 (1.198)(1) \approx \mathbf{5.18\ kip / ft^2}$$

5.20 Eq. (5.56):

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

$$z' = (1.4)(0.3)\left(\frac{1}{0.3}\right)^{0.75} = 1.04$$

Eq. (5.57): $$\frac{S_e}{B_R} = \alpha_1\alpha_2\alpha_3\left[\frac{1.25\left(\frac{L}{B}\right)}{0.25+\frac{L}{B}}\right]^2\left(\frac{B}{B_R}\right)^{0.7}\left(\frac{q'}{p_a}\right)$$

Normally consolidated sand:

$\alpha_1 = 0.14$

$$\alpha_2 = \frac{1.71}{(\overline{N}_{60})^{1.4}} = \frac{1.71}{(8)^{1.4}} = 0.093$$

5.19 Following are the results of standard penetration tests in a granular soil deposit.

Depth (ft)	Standard penetration number, N_{60}
5	10
10	12
15	9
20	14
25	16

What will be the net allowable bearing capacity of a foundation planned to be 5 ft × 5 ft? Let $D_f = 3$ ft and the allowable settlement = 1 in., and use the relationships presented in Section 5.12.

5.20 A shallow foundation measuring 1 m × 2 m in plan is to be constructed over a normally consolidated sand layer. Given: $D_f = 1$ m, N_{60} increases with depth, $\overline{N}_{60}$ (in the depth of stress influence) = 8, and $q_{net} = 153$ kN/m^2. Estimate the elastic settlement using Burland and Burbidge's method.

5.21 Following are the average values of cone penetration resistance in a granular soil deposit:

Depth (m)	Cone penetration resistance, q_c (MN/m^2)
2	2.1
4	4.2
6	5.2
8	7.3
10	8.7
15	14

Assume that $\gamma = 16.5$ kN/m^3 and estimate the seismic ultimate bearing capacity (q_{uE}) for a continuous foundation with $B = 1.5$ m, $D_f = 1.0$ m, $k_h = 0.2$, and $k_v = 0$. Use Eqs. (2.47), (5.67), and (5.68).

5.22 In problem 5.21, if the design earthquake parameters are $V = 0.35$ m/sec and $A = 0.3$, determine the seismic settlement of the foundation. Assume that FS = 4 for use in obtaining the static allowable bearing capacity.

5.23 Estimate the consolidation settlement of the clay layer shown in Figure P5.6 using the results of Problem 5.6.

5.24 Estimate the consolidation settlement of the clay layer shown in Figure P5.6 using the results of Problem 5.7.

References

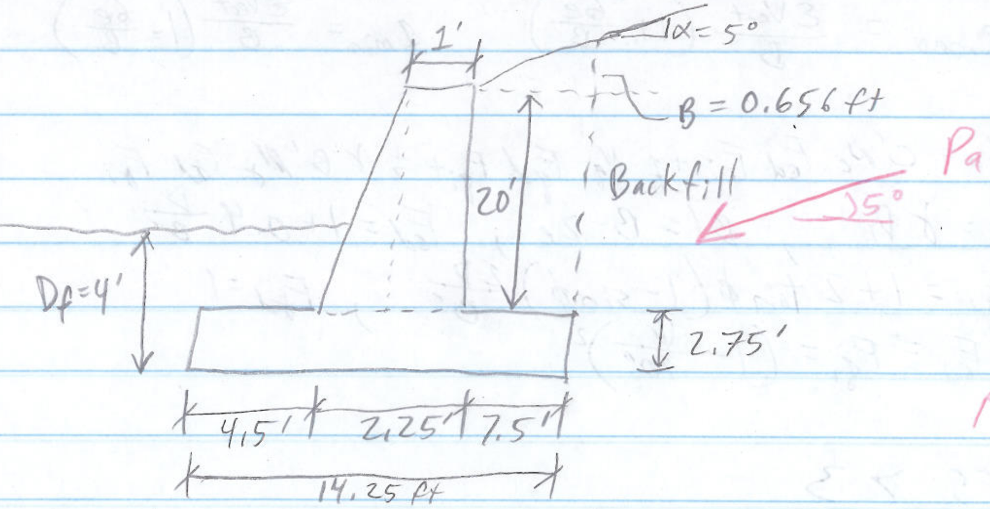
Ahlvin, R. G., and Ulery, H. H. (1962). *Tabulated Values of Determining the Composite Pattern of Stresses, Strains, and Deflections beneath a Uniform Load on a Homogeneous Half Space.* Highway Research Board Bulletin 342, pp. 1–13.

Do chap 6 H.W., chap 7. H.W., chap. 9 H.W.

Foundations Oct. 27 ①

chap. 13.1 - 13.7 My Project

Ex



Backfill = γ = 117 psf $\phi = 34°$

Foundation Soil = γ = 107 $\phi = 18°$ $C = 1.05$ ksf

Solve Using Rankine's Theory

Table (7.13) = 0.286

earth pressure $P_a = \frac{1}{2}\gamma H^2 K_a = \frac{1}{2}(117)(2.75+20+0.656)^2(0.286)$

P_a = 9.16 k/ft acts @ $\frac{H}{3}$ from bottom of base

① Stability Against Overturning $F.S. = \frac{\Sigma M_{righting}}{\Sigma M_{o/t}} > 1.5$

② F.S. against sliding $= \frac{\text{Resisting Force}}{\text{Driving Force}} > 1.5$

vertical Forces

$\tau = c + \sigma \tan\phi = B c_a + B \sigma \tan\phi = B_{ca} + \Sigma V \tan$

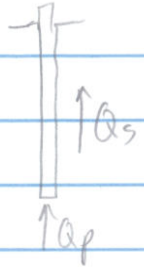
τ = 29.32 k/ft tan(12°) + B(0.7 ksf) = 16.2 k/ft

F.S. sliding $= \frac{16.2}{9.14} = 1.8$ > 1.5 okay

Foundations NOV 10 ①

Piles

Air Hammer	Diesel Hammer
0.67%	0.70%
Hydraulic Hammer	Vibratory Hammer
0.80% more efficient, quieter	Ξ



Forms of capacity

- static testing
- Dynamic testing

$Q_{ult} = Q_s + Q_p$

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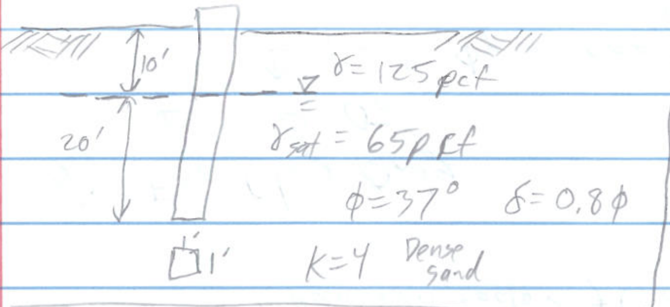
Example



Tip Capacity:

$*q_p = q' N_q$; $q' = \sigma_o'$ @ pile tip

$= 10'(125\text{pcf}) + 20(65$

$= 2550$ psf

$Q_p = q_p(A_p)$

$= 510$ K

N_q (table 11.5 p. 558) ~ 200

check $Q_p \leq A_p q_1$ $\{ q_1 = 0.5(2000)(200)(\tan 37°) ; q_1 A_p = 150\text{K} \}$

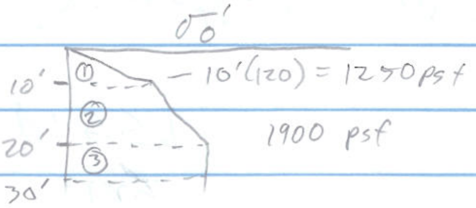
$\therefore$ USE $150\text{K} = Q_p$

Friction Capacity

$L' = \text{Dense} = 20D = 20'$

perimeter K tan δ

$Q_s = \frac{1}{2}(1250(10)) + \frac{1}{2}((1250+1900)(10)) + 10(1900)(4\times1)(4)($



$Q_s = 267{,}700$ lb

$Q_{ult} = 267.7 + 150 = 417.7$ K

$Q_{all} = \frac{417.7}{3} = 139$K

F.S. w/ no load testing

Foundations Nov 10 ②

$Q_{All} = \frac{164.5}{3} = 55^k$

- Using λ method

$$Cu_{Avg} = \left(\frac{600(25) + 2000(15)}{40\ ft}\right) = 1125\ lb/ft^2$$

$\sigma_o' = 2200$ psf

avg over depths

use chart! $\frac{0.2 - 0.245}{15-10} = \frac{0.2 - x}{15 - 12.5} \rightarrow x = 12.12$

$\hookrightarrow \lambda = 0.228$

$f_{avg} = 0.225(2200 + (2(1125))) = 1001$

$Q_s = \underbrace{(4(\tfrac{14}{12}))}_{p}\underbrace{(40)}_{L}\underbrace{(1001)}_{f_{avg}} = 187^k$

$Q_{ult} = Q_s + Q_p = 187 + 24.5 = 211.5$

$Q_{all} = \frac{211.5}{\underset{F.S.}{3}} = 70.5$

If can't choose b/t methods then:

$Q_{S\ Avg} = \frac{\overset{\alpha}{187} + \overset{\lambda}{140}}{2} = 163.5^k \quad Q_u = 163.5 + 24.5 = 188^k$

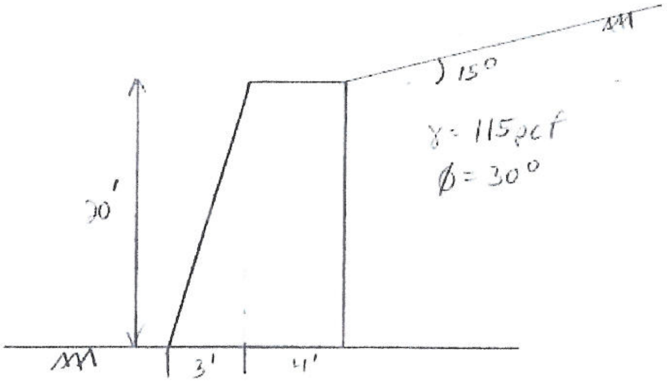
$Q_{All} = \frac{188^k}{3} = 63^k$

89/100

NAME: Donald Jerolleman

SHOW ALL COMPUTATIONS. LIST ANY ASSUMPTIONS YOU MAKE. BE NEAT. SOLVE THE FOLLOWING PROBLEMS BY HAND COMPUTATIONS.

Problem 1: Consider the retaining wall shown. Using Rankine's Theory, compute the active earth pressure on the wall and show the magnitude, direction, and location of the force on the sketch. (15 Points)



Problem 2: Check the stability of the wall shown. Use Rankine's Theory. Neglect passive pressure. The ultimate bearing capacity is known to be 9 ksf for the foundation soil. The foundation soil has an angle of shearing resistance of φ = 25°

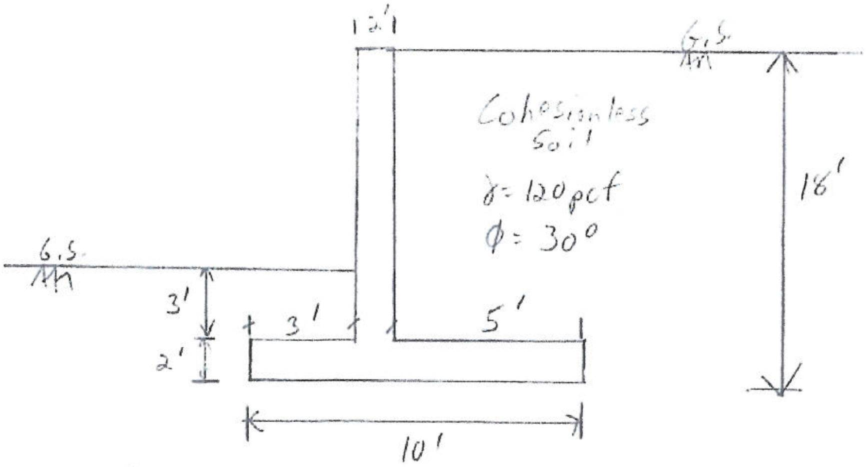
qu

c=0

Υ concrete = 150 pcf

Assume δ = φ for foundation soil

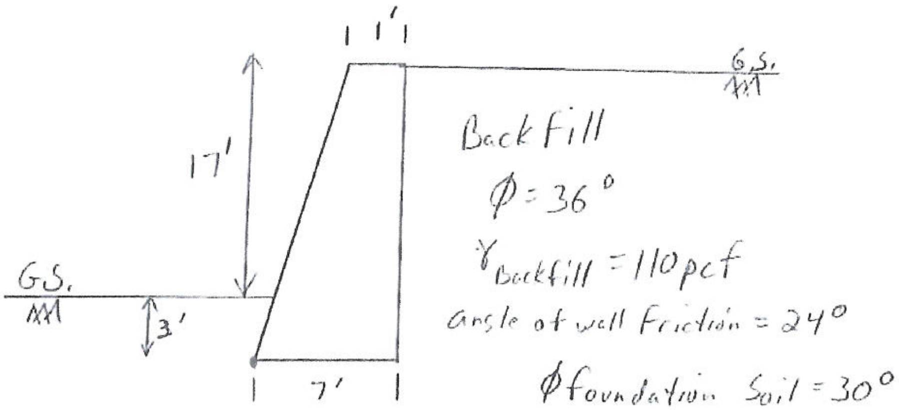
(15 Points)



Problem 3: For the retaining wall shown below:

a) Check the factor of safety against overturning
b) Check the factor of safety against sliding
c) If the allowable bearing pressure for foundation soil is 3 ksf, is the wall safe against failure of the foundation soil?

Use Coulomb's Theory for computations. Comment on the factor of safety determined. Neglect passive resistance. (20 Points)

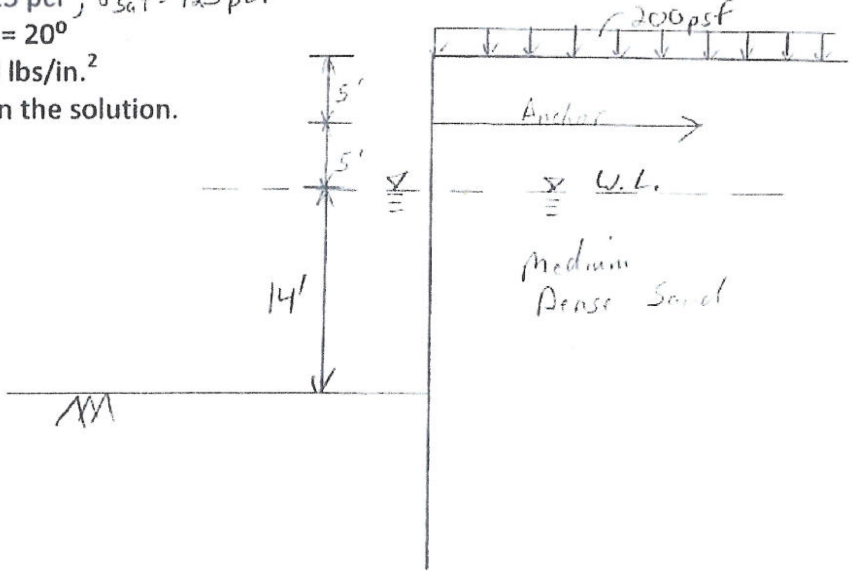


Problem 4: Design an anchored bulkhead for the conditions shown by:

a) Free earth support method and
b) CPD method
c) In Part A, select a sheetpile section by applying Rowe's Moment Reduction Method.

Given:

Υ moist = 113 pcf , γ_{sat} = 125 pcf
ϕ = 30°; δ = 20°
σ_{all} = 25,000 lbs/in.^{2}
Comment on the solution.
(25 Points)

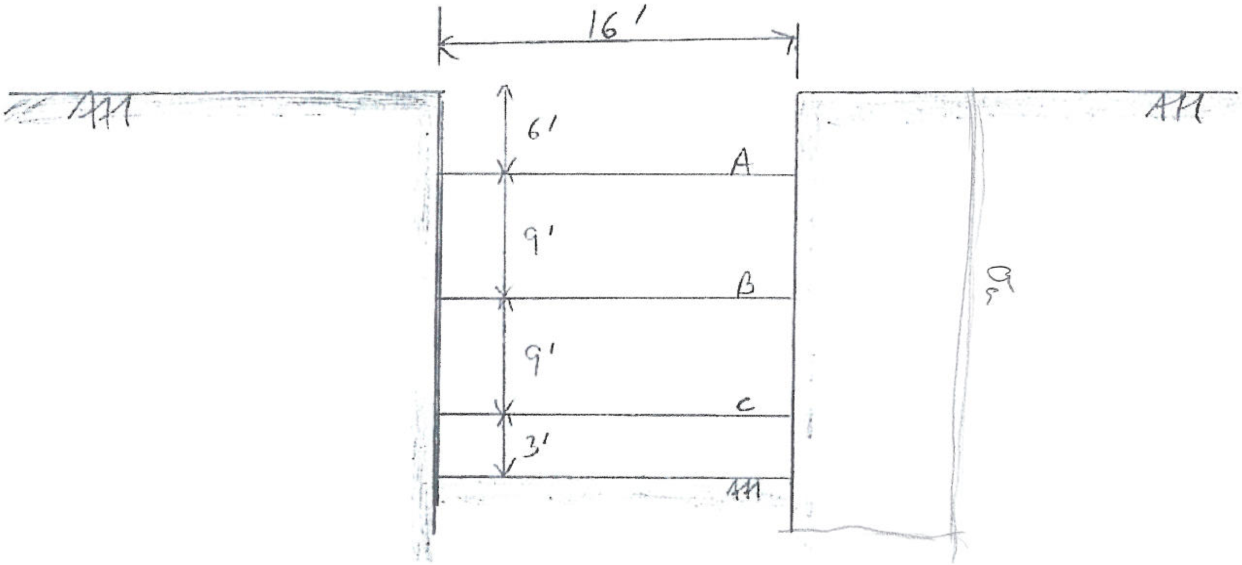


Problem 5: According to the braced cut shown below, for which $\gamma = 112$ lb/ft^3, $\phi = 32^0$, and c = 0. The struts are located at 12-ft centers in the plan.

a) Draw the earth pressure envelope and determine the strut loads at Levels A, B, and C.

b) For the braced cut, assume $\sigma_{all} = 25{,}000$ lbs/in.^{2} and determine the sheetpile section.

(25 Points)



YOUR SIGNATURE: ______________________________ **I PLEDGE ON MY HONOR THAT I HAVE NOT TAKEN HELP OR GIVEN HELP ON THIS EXAM.**

Donald Jerolleman

1) $K_a = \cos\alpha \dfrac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi'}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi'}}$

$\alpha = 15^\circ$
$\cos\alpha = 0.966$
$\cos\alpha^2 = 0.933$
$\cos^2\phi = 0.75$

+15

$K_a = 0.373$

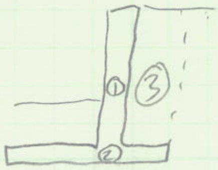
$P_a = \frac{1}{2}\gamma H^2 K_a = 0.5(115)(20)^2(0.373) = 8579$ lb/ft

$\frac{20'}{3} = 6.67$ ft

---

2) $K_a = \tan^2(45 - \frac{\phi'}{2}) = 0.333$ $\phi' \leftarrow 30$ $P_a = \frac{1}{2}\gamma H^2 K_a = 6480$ ✓

+15



sec	wt	M_{arm}	M
1	4800	4'	19200
2	3000	5'	15000
3	9600	7.5'	72000
	$\Sigma = 17{,}400 = \Sigma V$ ✓		$\Sigma = 106{,}200$ ✓

$M_{OT} = P_a\left(\frac{H'}{3}\right) = 38\,880$ ft-lb/ft ✓ K·ft/ft wall

$F.S._{OT} = \frac{106200}{38\,880} = 2.73$ ✓ > 2 ∴ OKAY

$F.S._{SL} = \dfrac{(\Sigma V)\tan\delta' + B c_a' + P_p}{P_a \cos\alpha}$ $c_a' = P_p = \alpha = \emptyset$; $\tan\delta = \tan 25 = 0.466$ $B = 10$

$= 1.3 < 1.5$ ∴ NOT OKAY

$e = \frac{B}{2} - \dfrac{\Sigma M_R - \Sigma M_{OT}}{\Sigma V} = 1.13$ ft

$q_{toe} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) = 2920.8$

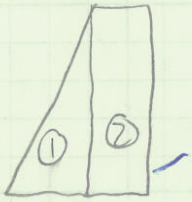
$FS_{BC} = \frac{q_u}{q_{toe}} = \frac{9\text{ksf}}{2920.8\text{ lbsf}} = 3.08$ ✓ > 3 OKAY

Be neat!!

Show all calculations!

3) $K_a \sim 0.24$ Table 7.3

$P_a = 1/2\ K_a \gamma H^2 = 0.5(0.24)(110)(20)^2 = 5280$



+14

-4

Sect	Wt	Marm	M
1	9000	4	36000
2	3000	6.5	19500

$\Sigma V = 12000$ 21,103 k/ft 7' $\Sigma M_R = \underline{55500}$

70,225 k-ft/ft

$\underline{3}$ · Live Load $\rightarrow$

$M_{OT} = 5280(6.67)$
$= 35200$

$F.S_{OT} = \frac{M_R}{M_{OT}} \overset{70.22}{} = 1.58 <$ $\boxed{\text{FAIL}}$

2.2

$\{ P_p = c_2' = \emptyset \}$

$$FS_{SL} = \frac{\Sigma V \tan\left(\frac{2}{3}\phi_2'\right) + \frac{2}{3}c_2' B + P_p}{P_a}$$

No calcs

-1

$= \overset{1.08}{0.82} < 1.5$ $\boxed{\text{FAIL}}$

$e = \frac{B}{2} - \frac{\Sigma M_R - \Sigma M_O}{\Sigma V} = 1.8 > \frac{B}{6} = 1.17$

$e > \frac{B}{6}$ $\boxed{\text{Tesile Forces exist}}$

-1

$q_{toe} = \frac{\Sigma V}{B}\left[1 + \frac{6e}{B}\right) = 4359$ & $3,335.9\ \text{lb/ft}^2$

$F.S. = \frac{q_u}{q_{all}} \rightarrow 3 = \frac{q_u}{3\text{ksf}} \rightarrow q_u = 9\ \text{ksf}$

$F.S._{B.C.} = \frac{q_u}{q_{toe}} = \frac{9000}{4359} = 2.06 < 3$ $\boxed{\text{FAIL}}$

4) ⓐ $l_1 = l_2 = 5'$; $L_1 = 10'$; $L_2 = 14'$; $c' = 0$; $\phi' = 30°$; $\gamma = 113\ pcf$; $\gamma_{sat} = 125\ pcf$

$\sigma_{all} = 25000\ psi$; $q = 200\ psf$; $\gamma' = \gamma_{sat} - \gamma_w = \boxed{62.6\ pcf}$

+20

-3

$K_a = \tan^2(45 - \frac{\phi'}{2}) = \underline{\frac{1}{3}}$; $K_p = \tan^2(45 + \frac{\phi'}{2}) = \underline{3}$

$\cancel{K_o} = 1 - \sin\phi' = \boxed{0.5}$; $K_p - K_a - \cancel{K_o} = \underline{2.167}$

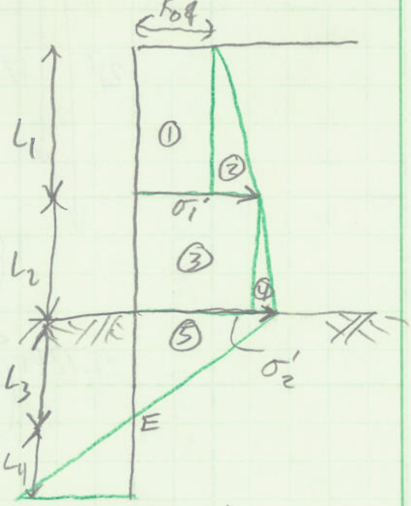
$K_o(q) = \boxed{100}$ $\qquad K_p - K_a$

Be neat

Hard to follow Calcs

$\sigma_1' = \gamma L_1 K_a + K_o q = \boxed{476.6\ psf}$ $\qquad \sigma_2' = (\gamma L_1 + \gamma' L_2) K_a + K_o q = \boxed{769\ psf}$

$L_3 = \frac{\sigma_2'}{\gamma' (K_p - K_a - K_o} = \boxed{5.67\ ft}$



$P = \Sigma A = \overset{1000}{(L_1 K_o q)} + \overset{1883}{(\frac{1}{2} L_1 (\sigma_1' - (Kq)))}$

$+ \overset{6672.4}{L_2(\sigma_1')} + \overset{2047}{(\frac{1}{2} L_2 (\sigma_2' - \sigma_1'))} + \overset{2180}{\frac{1}{2} \sigma_2' L_3}$

$= \boxed{13782\ lb}$

$\overline{Z} = \frac{\Sigma M_E}{P} = \{ \overset{24670}{(1000(5.67 + 14 + \frac{10}{2}))} + \overset{43315}{1883(5.67 + 14 + \frac{10}{3})}$

181923

$+ \overset{84539}{6672.4(5.67 + \frac{14}{2})} + \overset{21159}{2047(5.67 + \frac{14}{3})} + \overset{8240}{2180(5.67(\frac{2}{3}))} \} \frac{1}{13782}$

$= \boxed{13\ ft}$

[9.67] P.464

$$L_4^3 + 1.5 L_4^2 \underbrace{(l_2 + L_2 + L_3)}_{24.67} - \frac{\overbrace{3P[\overset{29.67}{(L_1 + L_2 + L_3)} - \overset{18}{(\overline{Z} + l_1)}]}^{482508}}{\underbrace{\gamma' (K_p - K_a - K_o)}_{135.7}} = 0$$

$L_4^3 + 37 (L_4^2) - 3555.7 = 0 \rightarrow L_4 = \begin{Bmatrix} -33.9 \\ 8.8 \\ -11.9 \end{Bmatrix}$ choose 9 ft

$D_{THEORY} = L_3 + L_4 = 14.67$

$D_{actual} = 1.4\, D_{THEORY} \cong 20$

$F = P - \frac{1}{2} \gamma' (K_p - K_a - K_o) L_4^2 = 8288\ lb$

OVER

4b) $\gamma = 113$; $\gamma_{sat} = 125$; $\gamma' = \gamma_{sat} - \gamma_w = 62.6$ psf $c' = 0$ $\phi' = 30$

$$\gamma'_{av} = \frac{\gamma L_1 + \gamma' L_2}{L_1 + L_2} = \frac{113(10) + 62.6(14)}{24} = 83.6 \text{ psf} ✓$$

$K_a = 0.3333$ $K_p = 3$ $K_o = 0.5$ $K_o q = 100$ $K_a + K_o = 0.833$

$C = 0.70$ ✓ $R = 0.55$ ✓

b/c of surcharge

$$\bar{\sigma}_a' = C(K_a + K_o)\gamma'_{av} L_{1+2} = 0.70(0.833)(83.6)(24) = 1170.4 \text{ psf}$$

$$\bar{\sigma}_p' = R\,\bar{\sigma}_a' = 0.55(1170.4) = 643.7 \text{ psf}$$

$$D^2 + 2DL\left[1 - \left(\frac{l_1}{L}\right)\right] - \frac{L^2}{R}\left[1 - 2\left(\frac{l_1}{L}\right)\right] = 0$$

$$D^2 + 38D - 610.9 = 0 \qquad D = 12.18$$

$$F = \bar{\sigma}_a'(L - RD) = 20249$$

$$M_{max} = 0.5\,\bar{\sigma}_a' L^2\left[\left(1 - \frac{RD}{L}\right)^2 - \left(\frac{2 l_1}{L}\right)\left(1 - \frac{RD}{L}\right)\right]$$

$$= 337075\left[0.52 - 0.3\right] = 74033 \text{ lb-ft}$$

-1

---

c) $H' = L_1 + L_2 + D_{actual} = 10 + 14 + 20 = \underline{44 \text{ ft}} = 13.41$ m

1 ft·lb = 1.356 N-m

1 ft = 0.3048m

$M_{max} = \underline{76000 \text{ lb-ft}}$ = 103056 N-m

*Assume E_{steel} = 29000000 psi

section	$I\left[\text{in}^4/\text{ft}\right]$	H'[ft]	$\rho = \frac{H'^4}{EI}$	$\log \rho$	S in³/ft	$M_d = S\sigma_{all}(\cancel{12})\left(\frac{1}{12}\right)$ lb-ft/ft	$\frac{M_d}{M_{max}}$
PZ-35	361.2	44	0.000358	-3.446	48.5	101042	1.33
PZ-27	184.2	44	0.0007017	-3.154	30.2	62917	0.839
PZ-22	84.4	44	0.001531	-2.815	18.1	37708	0.503

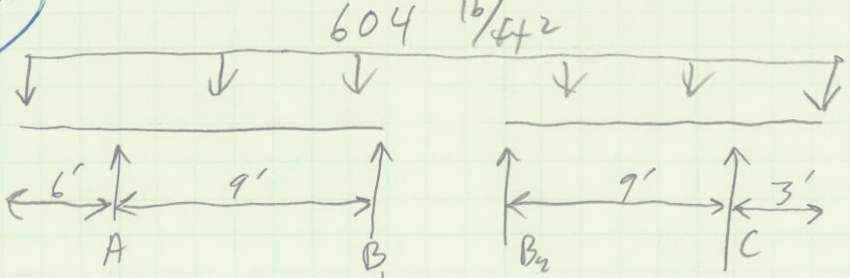
assume [illegible] again [illegible] to mult by 12 All section are OKAY

compare to Fig 9.25p. 472   PZ22 = No good

PZ-27 = OKAY ∴ choose $\boxed{\text{PZ-27}}$

+25

5) $\sigma_a = 0.65 \gamma H K_a = 0.65 \gamma H \tan^2(45 - \frac{32}{2}) = 603.9 = 604$



$\circlearrowright + \Sigma M_{B_1} = 0$

$= +604(15)(7.5) - A_y(9)$

$\underline{A_y = 7550 \text{ lb/ft} \uparrow}$ ✓

$\Sigma F_y = 0 = -604(15) + A_y + B_{y_1}$

$\underline{B_{y_1} = 1510 \text{ lb/ft} \uparrow}$ ✓

$\circlearrowright + \Sigma M_{B_2} = 0$

$= -604(12)(6) + C_y(9)$

$\underline{C_y = 4832 \text{ lb/ft} \uparrow}$ ✓

$\Sigma F_y = 0 = -604(12) + B_{y_2} + C_y$

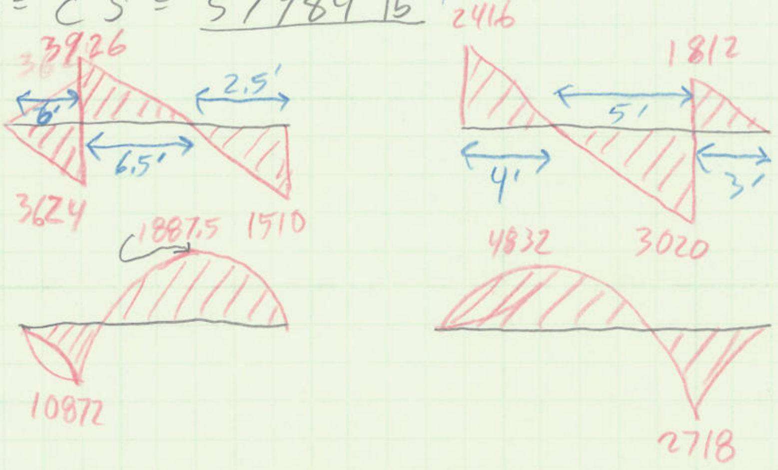
$B_{y_2} = \underline{2416 \text{ lb/ft} \uparrow}$ ✓

$S = 12 \text{ ft}$

$P_A = A S = 7550 \text{ lb/ft} (12 \text{ ft}) = \underline{90600 \text{ lb}}$ ✓

$P_B = (B_1 + B_2) S = \underline{47112 \text{ lb}}$ ✓

$P_C = C S = \underline{57984 \text{ lb}}$ ✓



max Moment

$= 10872$ ✓

$S = \frac{M_{max}}{\sigma_{all}} = \frac{10872 \text{ lb-ft/ft} \left(\frac{12''}{1 \text{ ft}}\right)}{25000 \text{ lb/in}^2}$

$S = 5.22 \text{ in}^3/\text{ft}$ ✓

Table 9.1 → select PZ-22 ($S = 18.1 \text{ in}^3/\text{ft}$)

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1(A, C), 3, 5, 7, 10, 11, 14

3.1) (a) p. 138 equ. 3.3 continuous → $q_u = c' N_c + \overset{\gamma D_f}{q} N_q + \frac{1}{2} \gamma B N_\gamma$

From Table 3.1 ($\phi' = 28°$) → $N_c = 31.61$, $N_q = 17.81$, $N_\gamma = 13.70$

$q_u = 400\ \mathrm{lb/ft^2}(31.61) + 3\mathrm{ft}(110\ \mathrm{lb/ft^3})(17.81) + (0.5)(110\ \mathrm{lb/ft^3})(3\mathrm{ft})(13.7) = \underline{20782}$

$q_{all\ (gross)} = \frac{q_u}{FS} = \boxed{5195\ \mathrm{lb/ft^2}}$

(c) Table 3.1 ($\phi = 30°$) → $N_c = 37.16$, $N_q = 22.46$, $N_\gamma = 19.13$

$q_u = (1.3)(0)(37.16) + (16.5\ \mathrm{kN/m^3})(2\mathrm{m})(22.46) + (0.4)(16.5\ \mathrm{kN/m^3})(3\mathrm{m})(19.13)$

$q_u = 1120\ \mathrm{kN/m^2}$ $\boxed{q_{all} = 280\ \mathrm{kN/m^2}}$

square Foundation → $q_u = 1.3(c')(N_c) + q(N_q) + 0.4(\gamma)(B)(N_\gamma)$

3.3) (a) (c)

(a) equ. 3.19 p. 143 Table 3.3 ($\phi = 28°$) $\underline{N_c = 25.80}$, $\underline{N_q = 14.72}$, $\underline{N_\gamma = 16.72}$

$q_u = c' N_c F_{cs} F_{cd} F_{ci} + \overset{\gamma(D_f)}{q} N_q F_{qs} F_{qd} F_{qi} + 0.5(\gamma) B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$

Table 3.4 p. 145 $\left(\frac{D_f}{B} = \frac{3}{3} = 1\right)$ → $\underline{F_{cs}, F_{qs}, F_{\gamma s} \sim 1}$ b/c continuous

→ $(\phi' > 0)$ $F_{cd} = 1 + 0.4\left(\frac{D_f}{B}\right) = \underline{1.4}$ $\underline{F_{\gamma d} = 1}$

$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right) = \underline{1.299}$

$q_u = (400\ \mathrm{lb/ft^2})(25.8)(1)(1.4)F_{ci} + (110\ \mathrm{lb/ft^3})(3\mathrm{ft})(14.72)(1)(1.299)(F_{qi}) + 0.5(110\ \mathrm{lb/ft^3})(3\mathrm{ft})(16.72)(1)(1)(F_{\gamma i})$

$= 14448(F_{ci}) + 6310(F_{qi}) + 2759(F_{\gamma i}) = 23517\ \mathrm{lb/ft^2}$

Table 3.4
since $\beta = 0$ $F_{ci}, F_{qi}, F_{\gamma i} = 1$ }

$\boxed{q_{ALL} = 5879\ \mathrm{lb/ft^2}}$

(c) Table 3.3 ($\phi = 30°$) → $N_c = 30.14$, $N_q = 18.40$, $N_\gamma = 22.4$

Table 3.4 $\left(\frac{B}{L} = 1\right)$ → $F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = \underline{1.61}$ $F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi' = \underline{1.577}$ $F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = \underline{0.6}$

$\left(\frac{D_f}{B} = \frac{2}{3} \leq 1\right)$ $F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right) = \underline{1.192}$ $F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} = \underline{1.204}$

$F_{\gamma d} = 1$ since $\beta = 0$ $F_{ci} = F_{qi} = F_{\gamma i} = 1$

$q_u = 0(30.14)(1.61)(1.204)(1) + 16.5\ \mathrm{kN/m^3}(2\mathrm{m})(18.4)(1.577)(1.192)(1) + 0.5(16.5\ \mathrm{kN/m^3})(3\mathrm{m})(22.4)(0.6)(1)(1)$

$= 1474\ \mathrm{kN/m^2}$ $\boxed{q_{all} = 369\ \mathrm{kN/m^2}}$

NOT clear which is B or L

3.5) B = 2m, L = 3m, D_f = 2m, $\phi' = 25°$, $c' = 50$ kN/m², FS = 4, $\frac{B}{L} = \frac{2}{3}$, $\frac{D_f}{B} = 1$

$\gamma_{Tob} = 16.8$ kN/m², $\gamma_{sat} = 19.4$ kN/m²

use. eqn 3.23 Find $\sigma_{All(net)}$

Equ. 3.14

$$q_u = c' N_c F_{cs} F_{cd} \overset{(1)}{\cancel{F_{ci}}} + \overset{\Sigma\gamma(D_f)}{q} N_q F_{qs} F_{qd} \overset{(1)}{\cancel{F_{qi}}} + 0.5\gamma B N_\gamma F_{\gamma s} \overset{(1)}{\cancel{F_{\gamma d}}} \overset{(1)}{\cancel{F_{\gamma i}}}$$

* Did not cover 2 γs yet with these methods how do we address?

Table 3.3 ($\phi' = 25°$) $N_c = 20.72$ $N_q = 10.66$ $N_\gamma = 10.88$

$F_{cs} = 1 + \left(\frac{2}{3}\right)\left(\frac{10.66}{\cancel{10.88}\ 20.72}\right) = 1.343$ $F_{qs} = 1.311$ $F_{\gamma s} = 0.733$

$F_{cd} = 1.343$ $F_{qd} = 1.311$

$$q_u = 50\ \text{kN/m}^2(20.72)(1.343)(1.343) + \Big(\underbrace{\left[16.8\ \text{kN/m}^2(1\text{m}) + 19.4\ \text{kN/m}^2(1\text{m})\right]}_{\text{guess}}(10.66)(1.311)(1.311) + \rightarrow$$

$$(0.5)\underbrace{\left(\frac{16.8\ \frac{\text{kN}}{\text{m}^2}(1\text{m}) + 19.4\ \frac{\text{kN}}{\text{m}^2}(1\text{m})}{2\text{m}}\right)}_{\text{guess}}(2\text{m})(10.88)(0.733)(1)(1) = 2676\ \text{kN/m}^2$$

$$q_{All(net)} = \frac{q_u - q}{FS} = \frac{2676 - (1\text{m}(16.8) + 1\text{m}(19.4))}{4} = \boxed{650\ \text{kN/m}^2}$$

3.7) B = 8', L = 8', grainular soil, D_f = 5', $\gamma = 110$ lb/ft³ = 0.06366 lb/in³

z/N_{60} : 5'/11, 10'/14, 15'/16, 20'/21, 25'/24

(a) Find ϕ', $p_a = 14.7$ lb/in² where is method/Formula in 7th ed?

→ eqn 2.27 (mybook) $\phi' = \tan^{-1}\left(\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma_o'}{p_a}\right)}\right)^{0.34}$

$\frac{D_f}{B} = \frac{5}{8} < 1$

z(in)	z(ft)	N_{60}	$\sigma_o' = z(\gamma)$ [lb/in²]	p_a [lb/in²]	ϕ'
60	5'	11	3.82	14.7	40.5
120	10'	14	7.64		40.3
180	15'	16	11.46		39.6
240	20'	21	15.28		40.5
300	25'	24	19.10	↓	40.4

Avg. $\phi' = 40.3°$

(b) eqn 3.14 mybook (assum $c' = \emptyset$) (FS not given)

$$q_u = \cancel{c'} N_c F_{cs} F_{cd} \cancel{F_{ci}} + \overset{[D_f(\gamma)]}{q} N_q F_{qs} F_{qd} \cancel{F_{qi}} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} \cancel{F_{\gamma d}} \cancel{F_{\gamma i}}$$

Table 3.3 ($\phi = 40°$) $N_c = \cancel{75.31}$, $N_q = 64.2$, $N_\gamma = 109.41$

$F_{cs} = 1 + 1\left(\frac{64.2}{75.31}\right) = \cancel{1.852}$ $F_{qs} = 1 + 1(\tan(40)) = 1.839$ $F_{\gamma s} = 0.6$

$F_{cd} = 1.1338 - \frac{1 - 1.1338}{75.31 \tan 40} = \cancel{1.136}$ $F_{qd} = 1 + 2(\tan 40)(1 - \sin 40)^2\left(\frac{5}{8}\right) = 1.1338$ $F_{\gamma d} = 1$

B/c $\beta = \emptyset$ $F_{ci} = F_{qi} = F_{\gamma i} = 1$

$$q_u = 5'(110\ \text{lb/ft}^3)(64.2)(1.839)(1.1338) + 0.5(110\ \text{lb/ft}^3)(8')(109.41)(0.6) = \boxed{102.5\ \text{K/ft}^2}$$

3.10) p.159 $B' = B - 2e$, $L' = L$, $A' = B'L'$; $\phi = 26°$, $\gamma_{sat} = 122\ lb/ft^3$, $\gamma = 110\ lb/ft^3$
$c' = 500\ lb/ft^2$, $e = 0.65 ft$ ∴ $B' = 6.7 ft$, $L' = 8 ft$

$D_f/B \leq 1$

$$Q_{ult} = q'_u A' = \left[c' N_c F_{cs} F_{cd} F_{ci} + \{D_f \gamma\} N_q F_{qs} F_{qd} F_{qi} + 0.5(\gamma) B' N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}\right] B'L'$$

Table 3.3 ($\phi = 26°$) $N_c = 22.25$ $N_q = 11.85$ $N_\gamma = 12.54$

$F_{cs} = 1 + \frac{6.7 ft}{8'}\left(\frac{11.85}{22.25}\right) = 1.446$ $F_{qs} = 1.408$ $F_{\gamma s} = 0.665$

$F_{cd} = 1.273$ $F_{qd} = (\text{use } B \text{ not } B') = 1.25$ (All other Factors = 1)

$$Q_{ult} = \left[500(22.25)(1.446) + \left[(3')(110\ lb/ft^3) + (3.5')(122\ lb/ft^3)\right](11.85)(1.408)(1.25) + (0.5)(122 - 62.4)(6.7)(12.54)(0.665)\right](6.7)(8) = \boxed{1798^k}$$

3.11) $L' = ?$? can not be done 'L' is not given.

If L Assumed ∞ ; $c' = 400\ lb/ft^2$, $\phi' = 25°$, $\gamma = 105\ lb/ft^3$, $\gamma_{sat} = 118\ lb/ft^3$

$$Q_{ult} = B\left[c' N_{c(e)} + q N_{q(e)} + \frac{1}{2}\gamma B N_{\gamma(e)}\right]$$ (equ. 3.42 p.159)

$\frac{e}{B} = \frac{2}{5} = 0.4$ ∴ $N_{q(e)} \cong 4$ (Figure 3.15 p. 160); $N_{q(e)} \overset{(Fig\ 3.16)}{\cong} 4$; $N_{\gamma(e)} \overset{(Fig.\ 3.17)}{\cong} \emptyset$

$$Q_{ult} = 5 ft\left[400\ lb/ft^2(4) + \left[2'(105\ lb/ft^3) + 2'(118\ lb/ft^3)\right](4) + \emptyset\right] = 3384 = \boxed{3.4^K}$$

*Does not seem right

3.14) $e_L = 0.06 ft$, $e_B = 1.5 ft$, $B = 4 ft$, $L = 6 ft$, $D_f = 3 ft$, S.F. = 4, $\gamma = 115\ lb/ft^3$, $\phi = 35°$, $c' = \emptyset$

$\frac{e_B}{B} = \frac{1.5}{4} > \frac{1}{6}$ $= 0.375 < \frac{1}{2}$ $\frac{e_L}{L} = \frac{0.06}{6} < \frac{1}{6}$ $= 0.01$ ∴ CASE III p.166 $\frac{D_f}{B} < 1$

$A' = \frac{1}{2}(B_1 + B_2)L$; p.168 Fig 3.22 $\left(\frac{e_B}{B}\right)0.375$, $\left(\frac{e_L}{L}\right)0.01 \rightarrow \frac{B_2}{B} = 0.225$, $\frac{B_1}{B} = 0.235$

$\rightarrow B_2 = 0.9 ft$, $B_1 = 0.94 ft$ $\rightarrow$ $A' = 5.52 ft^2$, $B' = \frac{A'}{L} = 0.92 ft$, $L' = L = 6 ft$

$$Q_{ult} = A'\left[\emptyset + (3'(115\ lb/ft^3))(N_q F_{qs} F_{qd})(1) + 0.5(115\ lb/ft^3)(0.92)(F_{\gamma s})(1)(1)(N_\gamma)\right.$$

Table 3.3 ($\phi = 35°$) $\Rightarrow$ $N_q = \underline{33.3}$, $N_\gamma = \underline{48.03}$ Table 3.4 $F_{qs} = 1 + \left(\frac{0.92}{6}\right)(\tan 35) = \underline{1.107}$

$F_{\gamma s} = 1 - 0.4\left(\frac{0.92}{6}\right) = \underline{0.939}$ $F_{qd} = 1 + (2\tan\phi(1 - \sin\phi)^2\left(\frac{D_f}{B}\right) = \underline{1.191}$

$Q_{ult} = 96.8^K$ $\boxed{Q_{All} = 24.2^K}$

67/100

etgarci1@uno.edu
-org - contact - room/capacity needed
- Date time

NAME: Donald Jerolleman

SHOW ALL COMPUTATIONS; LIST ANY ASSUMPTIONS YOU MAKE; BE NEAT. SOLVE THE FOLLOWING PROBLEMS BY HAND COMPUTATIONS.

P.76
$\frac{D_b}{S^{0.7}} = b$ 20'
7 stories

1. A 7-story concrete building will be constructed in new Orleans. The building will have plan dimensions of 200-ft by 275-ft. a) How many borings are needed? b) What are the depths of the borings? **10 points**

78.1 ft
Cost of failure high or don't know area @ all ~ 80 to 12 Table 2.4 ~ If area very well known

2. An electronic Cone Penetrometer Test has a tip resistance of 15 tsf. This tip reading is 20-ft below the ground surface. The soil has a moist unit weight of 115 pcf. What is the cohesive strength of the soil at the 20 foot depth? **10 points**

q_c
$\sigma = \sigma' = 2300$ lb/ft

$\frac{200}{30} = 6.67 \approx 8$

$\frac{275}{30} = 9.17 = 10$

3. A 5-ft square footing is to carry an **eccentric** load of 100 kips. The depth of the footing is 4-ft below the ground surface. The eccentricity is 0.6-ft in the x-direction. The density of the soil is 120 pcf. The cohesion is 2 ksf and the angle of shearing resistance is 30 degrees. Compute the ultimate bearing capacity using Hansen's method. How much is the ultimate column load? How much is the factor of safety **15 points**

4. A square foundation carries 400 kips of load. The SPT results from field are given below. The footing is to be 4-ft below the ground surface. The angle of shearing resistance is 30 degrees and density of sand is 110 pcf. a) determine the width of foundation such that the settlement **does not exceed** one inch. b) what is the allowable bearing capacity for this footing as per Terzaghi's Theory? c) Since you are an engineer and selected the footing size, state the appropriate method and verify the settlement is still limited to one inch.

SPT depth (ft)	N_{field}
1	9
5	10
10	15
15	22
20	19
25	29
30	33
35	27

$\frac{164}{9}$

AVG $N_{60} = 20.5$

15 points

5. A continuous footing is 6-ft wide and is at a depth of 3-ft below the ground surface. The moist density of soil is 115 pcf, the saturated density is 125 pcf, the cohesion is 550 psf, and the angle of shearing resistance is 35 degrees. The water table is 3-ft below the ground surface. Determine the allowable bearing capacity a) by Terzaghi's general shear case and b) general bearing capacity theory. Comment on your results. **15 points**

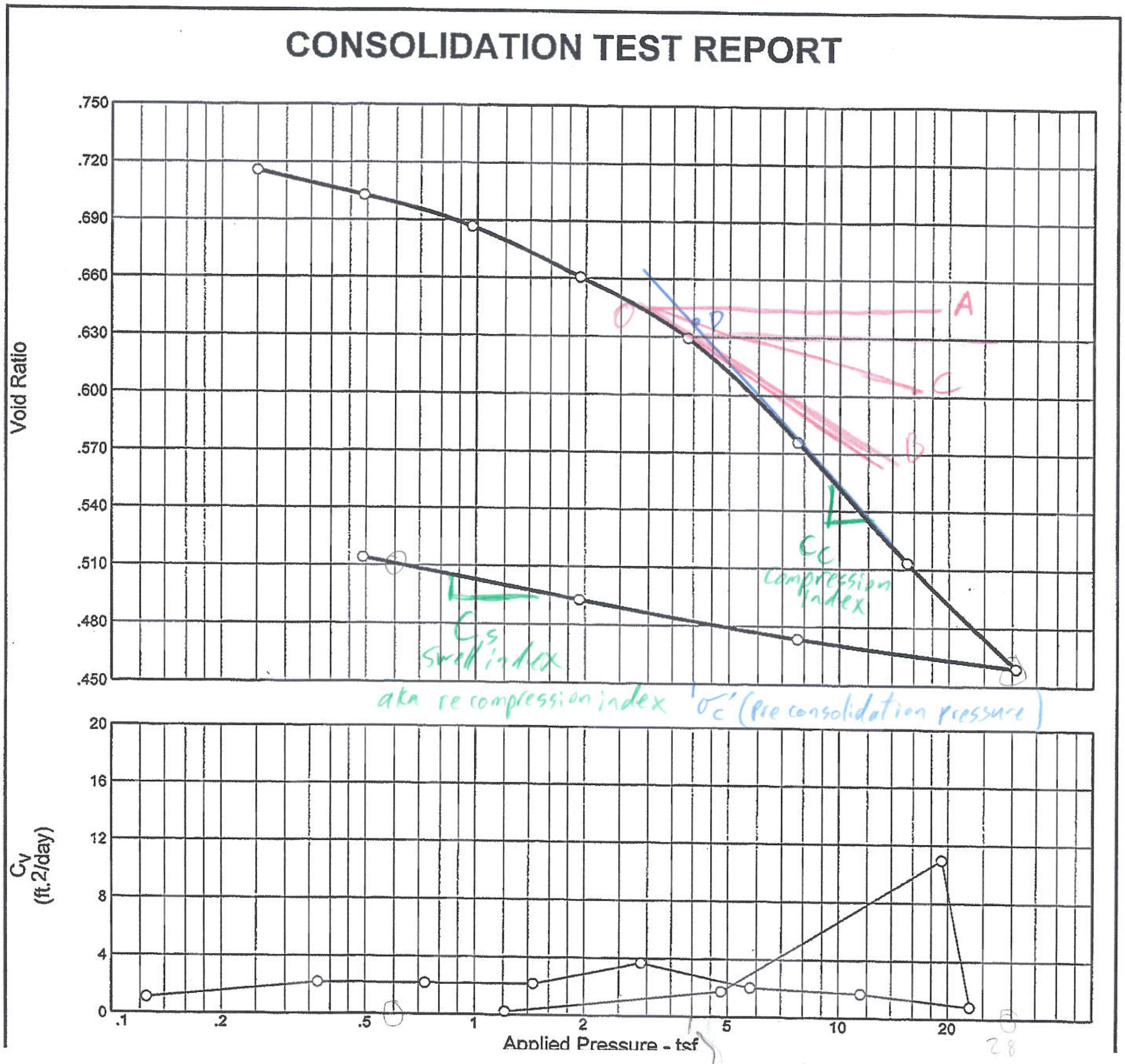
6. Given the furnished consolidation curve, use graphical methods to calculate the Compression Index, preconsolidation pressure, and Recompression Index.

10 points

7. Consider a 6-ft wide and 8-ft long foundation to support a load of 200 kips. The depth of the footing is 3-ft below the ground surface. There is dense sand for a depth of 8-ft below the ground surface, followed by 12-ft of normally consolidated clay, which rests on an impervious shale. The water table is 5-ft below the ground surface. The moist unit weight of sand is 100 pcf; the saturated unit weight of sand is 120 pcf; The properties of clay are: saturated unit weight is 115 pcf; void ratio Is 1.2; compression index = 0.2; coe. of consolidation = 0.2 sq.ft/mo. a) determine the elastic settlement at the center of the rigid footing, b) determine the consolidation settlement of the center of the footing using 2:1 method [at top, middle and bottom], c) repeat part b using Table 3.11 and compare your results, d) how long will it take for 90% of the settlement to occur? e) Determine the settlement of the corner of the footing and compute the differential settlement. Comment on your results.

25 points

YOUR SIGNATURE: _________________________ I PLEDGE ON MY HONOR THAT I HAVE NOT TAKEN HELP OR GIVEN HELP ON THIS EXAM.



0.4(4) = 1.6
2 + 1.6 = 3.6

$\sigma_c' = 3.6$ TSf

$C_c = 0.0075$

$C_s = 0.001825$

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1) p.76 $\frac{D_b}{S^{0.7}} = b = \frac{D_b}{7^{0.7}} = 20 ft \rightarrow D_b = 78.1 ft = \boxed{78 ft.}$

in N.O. USE 200'

+7

Table 2.4 100 ft spacing = 12 borings (If know area well)
30 ft spacing = 80 borings (Do Not " " well / High risk if fails)

From Notes: # borings $= \frac{55000}{2700} \simeq \boxed{20 \text{ borings}}$

---

2) $q_c = 15\ ton/ft^2$, depth = 20 ft, $\gamma = 115\ lb/ft^3$

+10

ϕ' (friction angle) $= \tan^{-1}\left[0.1 + 0.38 \log\left(\frac{q_c}{\sigma_o'}\right)\right]$

$\sigma_o' = \gamma z = 115\ lb/ft^3 (20 ft) = \underline{2300\ lb/ft^2}$

$\phi' = \tan^{-1}\left[0.1 + 0.38 \log\left(\frac{30{,}000\ lb/ft^2}{2300\ lb/ft^2}\right)\right] = \underline{27.65°}$

[2.51] C_u (shear strength) $= \frac{q_c - \sigma_o}{N_K} \rightarrow N_K$ (bearing capacity factor) = 15 for elec. cone (P.106)

$C_u = \frac{30{,}000 - 2300}{15} = \boxed{1846.7\ lb/ft^2}$ ✓

[1.81] $C_u = s = c' + \sigma' \tan\phi' \rightarrow c' = 1846.7 - (2300)(\tan(27.65)) = \boxed{641.7\ lb/ft^2}$

c' ← cohesion or apparent cohesion

---

3) * square (5x5 = 25ft²), eccentric load, $Q = 100^K$, $d_f = 4ft$, $e = 0.6 ft$ in x-direction,

+15 $\gamma = 120\ lb/ft^3$, $c' = 2000\ lb/ft^2$, $\phi' = 30°$, $q = \gamma d_f = 120(4) = 480\ lb/ft^2$

P.159 $B' = B - 2e = \underline{3.8 ft}$ ✓, $L' = L = \underline{5 ft}$

$$q_u' = c' N_c F_{cs} F_{cd} \cancel{F_{ci}}^{1} + q N_q F_{qs} F_{qd} \cancel{F_{qi}}^{1} + \frac{1}{2}\gamma B' N_\gamma F_{\gamma s} \cancel{F_{\gamma d}}^{1} \cancel{F_{\gamma i}}^{1}$$

$$q_u' = 2000\left(\left(\left(e^{\pi \tan 30} \tan^2\left(45+\frac{30}{2}\right)\right)-1\right)\cot\phi\right)\left(1+\left(1.8\tan^2 30 + 0.1\right)\left(\frac{3.8}{5}\right)^{0.5}\right)\left(1+0.4\left(\frac{4}{5}\right)\right)$$
$$+ 480\left(e^{\pi \tan 30}\tan^2\left(45+\frac{30}{2}\right)\right)\left(1+1.9\tan^2(30)\left(\frac{3.8}{5}\right)^{0.5}\right)\left(1+2\tan(30)(1-\sin 30)^2\left(\frac{4}{5}\right)\right)$$
$$+ 0.5(120)(3.8)\left(1.5\left(\left(e^{\pi\tan 30}\tan^2\left(45+\frac{30}{2}\right)\right)-1\right)(\tan 30)\right)\left(1+\left(0.6\tan^2(30) - 0.25\right)\left(\frac{3.8}{5}\right)\right)$$

$q_u' = \underline{148087\ lb/ft^2}$ $Q_{ult} = q_u' B' L' = 148087(3.8)(5) = \boxed{2{,}813{,}653\ lb}$ ✓

$FS = \frac{Q_{ult}}{Q} = \frac{2813653}{100000} = \boxed{28.1}$

check $\frac{q_u'}{q_{max}} = \frac{148087}{\frac{100000}{25}\left(1 + \frac{6(0.6)}{5}\right)} = 21.5$

q_{max} ← [3.36]

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4) * Square * $Q = 40000\,lb$, $d_f = 4'$, $\phi' = 30°$, $\overset{SAND}{\gamma} = 110\ lb/ft^3$, $S_e \le 1''$

Avg $N_{60} = 20.5$

+10

Net Bearing Pressure P.263 $q_{net} = \bar{q} - \gamma D_f = \frac{40^K}{BL} - 110(4)$ $\left\{B = L \rightarrow \frac{40^K}{B^2} - 110(4)\right\}$

Use N_{cor} −1

Assume Foundation $\overset{B}{} > 4\,ft$ & Settlement $= 1'' = 0.0833\,ft$

$\therefore$ [5.60] P.264 $q_{net}\left[\frac{K}{ft^2}\right] = \frac{N_{60}}{6}\left(\frac{B+1}{B}\right)^2 F_d S_e$

$F_d = 1 + 0.33\left(\frac{D_f}{B}\right)$

$S_e[in] = \frac{4\,q_{net}[K/ft^2]}{N_{60}\,F_d}\left(\frac{B}{B+1}\right)^2$ For $B > 4'$

See attached Excell Sheet

$$\frac{40^K}{B^2} - 440 = \frac{20.5}{6}\left(\frac{B+1}{B}\right)^2\left(1 + 0.33\left(\frac{4'}{B}\right)\right)(0.0833\,ft)$$

$$\frac{40^K}{B^2} = 36.94\left(\frac{B+1}{B}\right)^2\left(1 + 0.33\left(\frac{4}{B}\right)\right)$$

$$\emptyset = 36.94\left(\frac{B^2+2B+1}{\cancel{B^2}}\right)\left(\frac{B^2}{40} + \frac{1.32(\cancel{B^2})}{40\,B^{\cancel{3}}}\right)$$

$$= 36.94(B^2+2B+1)\left(\frac{1}{40} + \frac{1.32}{40\,B^3}\right)$$

$$= \cancel{36.94} \rightarrow 0.9235\ \frac{B^2+2B+1}{\cancel{40}} + \frac{1.32B^2 + 2.64B + 1.32}{40\,B^3}$$

$$= \frac{1.219B^4 + 2.438B^3 + 1.219B^2 + 2.43804B^3 + 4.876B^2 + 2.44B + 1.219B^2 + 2.438B}{40B^3}$$

$$\emptyset = \frac{\overset{0.03}{\cancel{1.219}}B^{\cancel{4}} + \overset{0.122}{\cancel{4.88}}B^{\cancel{3}} + \overset{0.183}{\cancel{7.314}}B^{\cancel{2}\,-1} + \overset{0.122}{\cancel{4.88}}B^{-2} + \overset{0.03}{\cancel{1.219}}B^{-3}}{\cancel{40B^3}}$$

$$\emptyset = 0.03B^{-3} + 0.122B^{-2} + 0.183B^{-1} + 0.122 + 0.03B$$

$$q_{all} = \frac{q_u}{FS} = \frac{1}{3}\left(1.3\overset{\emptyset}{\cancel{c'}}N_c + \overset{\gamma D_f}{q}N_q + 0.4\gamma BN_\gamma\right) = \frac{1}{3}\left(440\left(\overset{TABLE\ 3.1}{22.46}\right) + 0.4(110)(10)\left(\overset{TABLE\ 3.1}{19.13}\right)\right)$$

$$\boxed{q_{all} = 6100\ lb/ft^2}$$ ✓

$q_{net} = \bar{q} - \gamma D_f$; $\bar{q}$ is in lb/ft³ not K/ft³

gamma	110	lb/ft3	Q	40000	lb/ft3	
Df	4	ft	N60 avg	20.5		problem #4

uses ↓

↙ uses [5.58 qnet $= \frac{N_{60}}{6}\left(\frac{B+1}{B}\right)^2$ if B > 4 ft] ; q_{net} [K/ft²]

B	q(bar) lb/ft3	q(net) lb/ft3	q(net2) 5.58 k/ft3	Fd	(B/B+1)^2	Se	Se (q(net2))
4	2500.000	2060.000	5.339	1.3300	4	1208.876	3.133
4.5	1975.309	1535.309	5.104	1.2933	4	926.513	3.080
5	1600.000	1160.000	4.920	1.2640	4	716.270	3.038
5.5	1322.314	882.314	4.772	1.2400	4	555.351	3.004
6	1111.111	671.111	4.650	1.2200	4	429.339	2.975
6.5	946.746	506.746	4.549	1.2031	4	328.748	2.951
7	816.327	376.327	4.463	1.1886	4	247.119	2.930
7.5	711.111	271.111	4.389	1.1760	4	179.931	2.913
8	625.000	185.000	4.324	1.1650	4	123.940	2.897
8.5	553.633	113.633	4.268	1.1553	4	76.768	2.883
9	493.827	53.827	4.218	1.1467	4	36.638	2.871
9.5	443.213	3.213	4.174	1.1389	4	**2.202**	2.860
10	400.000	-40.000	4.134	1.1320	4	**-27.579**	2.850
10.5	362.812	-77.188	4.098	1.1257	4	-53.517	2.842
11	330.579	-109.421	4.066	1.1200	4	-76.252	2.834
11.5	302.457	-137.543	4.037	1.1148	4	-96.297	2.826
12	277.778	-162.222	4.010	1.1100	4	-114.065	2.819
12.5	256.000	-184.000	3.985	1.1056	4	-129.893	2.813
13	236.686	-203.314	3.963	1.1015	4	-144.057	2.808
13.5	219.479	-220.521	3.942	1.0978	4	-156.784	2.802
14	204.082	-235.918	3.922	1.0943	4	-168.266	2.797
14.5	190.250	-249.750	3.904	1.0910	4	-178.663	2.793
15	177.778	-262.222	3.887	1.0880	4	-188.108	2.789
15.5	166.493	-273.507	3.872	1.0852	4	-196.716	2.785
16	156.250	-283.750	3.857	1.0825	4	-204.585	2.781
16.5	146.924	-293.076	3.843	1.0800	4	-211.799	2.777
17	138.408	-301.592	3.830	1.0776	4	-218.428	2.774
17.5	130.612	-309.388	3.818	1.0754	4	-224.537	2.771
18	123.457	-316.543	3.807	1.0733	4	-230.178	2.768
18.5	116.874	-323.126	3.796	1.0714	4	-235.400	2.765
19	110.803	-329.197	3.786	1.0695	4	-240.243	2.763
19.5	105.194	-334.806	3.776	1.0677	4	-244.745	2.760
20	100.000	-340.000	3.767	1.0660	4	-248.936	2.758
20.5	95.181	-344.819	3.758	1.0644	4	-252.846	2.756
21	90.703	-349.297	3.750	1.0629	4	-256.499	2.754
21.5	86.533	-353.467	3.742	1.0614	4	-259.919	2.752
22	82.645	-357.355	3.734	1.0600	4	-263.124	2.750

-3

Choose B = 10′

Solve for Q

B ≈ 6.5 or 7′

-1

if use qnet [K/ft³] values are too large

approaches but does not reach w/in realilistic limits

5) * continuous, $B = 6'$, $D_f = 3'$, $\gamma = 115\ \text{lb/ft}^3$, $\gamma_{sat} = 125\ \text{lb/ft}^3$

+13

$c' = 550\ \text{lb/ft}^2$, $\phi' = 35°$, WT depth = 3 ft $\quad \frac{D_f}{B} = \frac{3}{6} = 0.5$

Table 3.1

γD_f

(a) $$q_{all} = \frac{q_u}{FS} = \frac{1}{3}\left(c'N_c + q N_q + \frac{1}{2}\gamma B N_\gamma\right) = \frac{1}{3}\Big(550(57.75) + (115)(3)(41.44)$$

$$+ 0.5(\overset{626}{\cancel{125}})(6)(45.41)\Big) = \boxed{21029\ \text{lb/ft}^2}$$

(-1)

*Assume b/c continuous ~1

(b) $$q_{all} = \frac{1}{3}\left(c'N_c \overset{1^*}{\cancel{F_{cs}}} F_{cd} \overset{*}{\cancel{F_{ci}}} + (\gamma D_f) N_q \overset{1^*}{\cancel{F_{qs}}} F_{qd} \overset{*}{\cancel{F_{qi}}} + 0.5(\gamma)(B)(N_\gamma) \overset{1^*}{\cancel{F_{\gamma s}}} \overset{1^*}{\cancel{F_{\gamma d}}} \overset{2^*}{\cancel{F_{\gamma i}}}\right)$$

*$\phi > 1 = 1$

*=1 b/c no inclination

Table 3.4 p.145

$$\left(\left(1 + \left((2\tan 35)(1-\sin 35)^2\left(\frac{3}{6}\right)\right)\right) - \left(\frac{(1-F_{qd})}{\tan^2(45+\frac{35}{2})e^{\pi\tan 35}(\tan 35)}\right)\right)$$

replace

$$q_{all} = \frac{1}{3}\left(550\left(\left(\left(\tan^2\left(45+\frac{35}{2}\right)e^{\pi\tan 35}\right)-1\right)\cot 35\right)\cancel{\left(1+0.4\left(\frac{3}{6}\right)\right)} + \left(345\left(\tan^2\left(45+\frac{35}{2}\right)e^{\pi\tan 35}\right)\right.\right.$$

$$\left.\left.\left(1+\left((2\tan 35)(1-\sin 35)^2\left(\frac{3}{6}\right)\right)\right)\right)\right)$$

IGNORE

$N_c = (N_q - 1)\cot 35 = 46.13$ $\qquad$ $N_q = \tan^2\left(45+\frac{35}{2}\right)e^{\pi\tan 35} = 33.3$

$F_{cd} = F_{qd} - \frac{1-F_{qd}}{N_c \tan 35} = 1.1146$ $\qquad$ $F_{qd} = 1 + 2\tan 35(1-\sin 35)^2\left(\frac{3}{6}\right) = 1.1112$

$N_\gamma = 2(N_q + 1)\tan 35 = 48.03$

$q_{all} = \boxed{15856\ \text{lb/ft}^2}$

-1

General Bearing Capacity Theory is more conservative than the Terzaghi's general shear case under these conditions.

---

6) See chart attached to test.

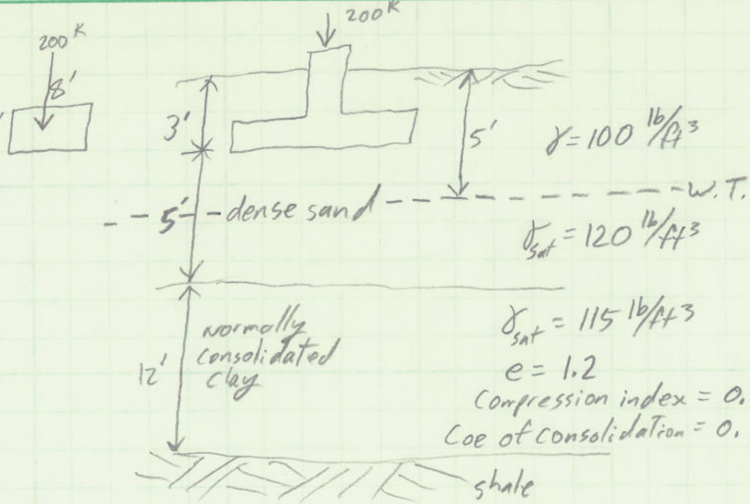
+8

✓ $\boxed{\sigma_c' = 3.6\ \text{TSF},\ C_c = 0.0075,\ C_s = 0.001825}$

$C_c = .189$ $\qquad$ $C_s = .0319$

7)

+4



$\gamma = 100\ \mathrm{lb/ft^3}$

$\gamma_{sat} = 120\ \mathrm{lb/ft^3}$

$\gamma_{sat} = 115\ \mathrm{lb/ft^3}$

$e = 1.2$

Compression index = 0.2

Coe of Consolidation = 0.2 $\mathrm{ft^3/month}$

* Assume $\mu_s = 0.325_{sand}$
$= 0.5_{clay}$

* Assume $E_s = 1.25\times10^6_{sand}$
$7\times10^5_{clay}$

See Refrences for above (attached)

(a) Find elastic settlement @ center.
(b) " consolidation settlement @ center using 2:1 method (@ top, middle, bottom) p.273
(c) Repeat (b) using Table 3.11 & compair results
(d) Find time of 90% settlement
(e) Find settlement of corner of footing & compute differential settlement.

[5.33]
p.246

(a) $S_e = q_0\,\alpha\,\beta'\ \dfrac{1-\mu_s^2}{E_s} I_s I_f$ $\quad \dfrac{D_f}{B} = \dfrac{3}{6} = 0.5 \quad \dfrac{B}{L} = \dfrac{6}{8} = 0.75$ $\quad m' = \dfrac{L}{B} = \dfrac{8}{6} = 1.3\bar{3}$ $\quad n'_s = \dfrac{H}{\frac{B}{2}} = \dfrac{8}{3} = 2.67$ H=5' (-1)

$n'_{all\,clay} = 6.67 \qquad n'_{top\,clay} = 2.67$

+4

$$S_e = \frac{200^K}{6(8)}(4)\left(\frac{6}{2}\right)\left[\left[\frac{1-0.325^2}{1.25\times10^6}\left(\overset{\text{Table 5.8}}{0.355} + \frac{1-2(0.325)}{1-0.325}\left(\overset{\text{Table 5.9}}{0.0663}\right)\right)\left(\overset{\text{Table 5.10}}{0.803}\right)\right)\right.$$

10^3 for Kip

$$+\left(\left(\frac{1-0.5^2}{7\times10^5}\right)\left(\overset{\text{Table 5.10}}{0.878}\right)\left(\left(\overset{\text{Table 5.8}}{0.5133} + \frac{1-2(0.5)}{1-0.5}\left(\overset{\text{Table 5.9}}{0.0305}\right)\right) - \left(0.355 + \frac{1-2(0.5)}{1-0.5}(0.0663)\right)\right)\right]$$

10^2

$$= 50\left[7.155\times10^{-\cancel{7}\,4}(0.3127) + \left\{9.407\times10^{-\cancel{7}\,4}(0.5133 - 0.355)\right\}\right]$$

$$= 0.0186\ \text{ft} = 0.22\ \text{inches}$$

Typical values of Poisson's ratio

Soil	**Poisson's ratio (n)**
Undrained saturated clays/silts	$n_u = 0.5$
Stiff sandy or silty clays	$n' = 0.2 - 0.4$
Medium to loose sands	$n' = 0.4$
Dense sands	$n' = 0.2 - 0.45$

Young's Modulus for Soil on the Geotechnical Information Website http://www.geotechnicalinfo.com/youngs_modulus.html

Typical Elastic Moduli of soils based on soil type and consistency/ density, (from USACE, *Settlement Analysis*).

Soil	**E_s (tsf)**
very soft clay	5 - 50
soft clay	50 - 200
medium clay	200 - 500
stiff clay, silty clay	500 - 1000
sandy clay	250 - 2000
clay shale	1000 - 2000
loose sand	100 - 250
dense sand	250 - 1000
dense sand and gravel	1000 - 2000
silty sand	250 - 2000

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