Solutions Manual - Geotechnical Engineering and Soil Testing

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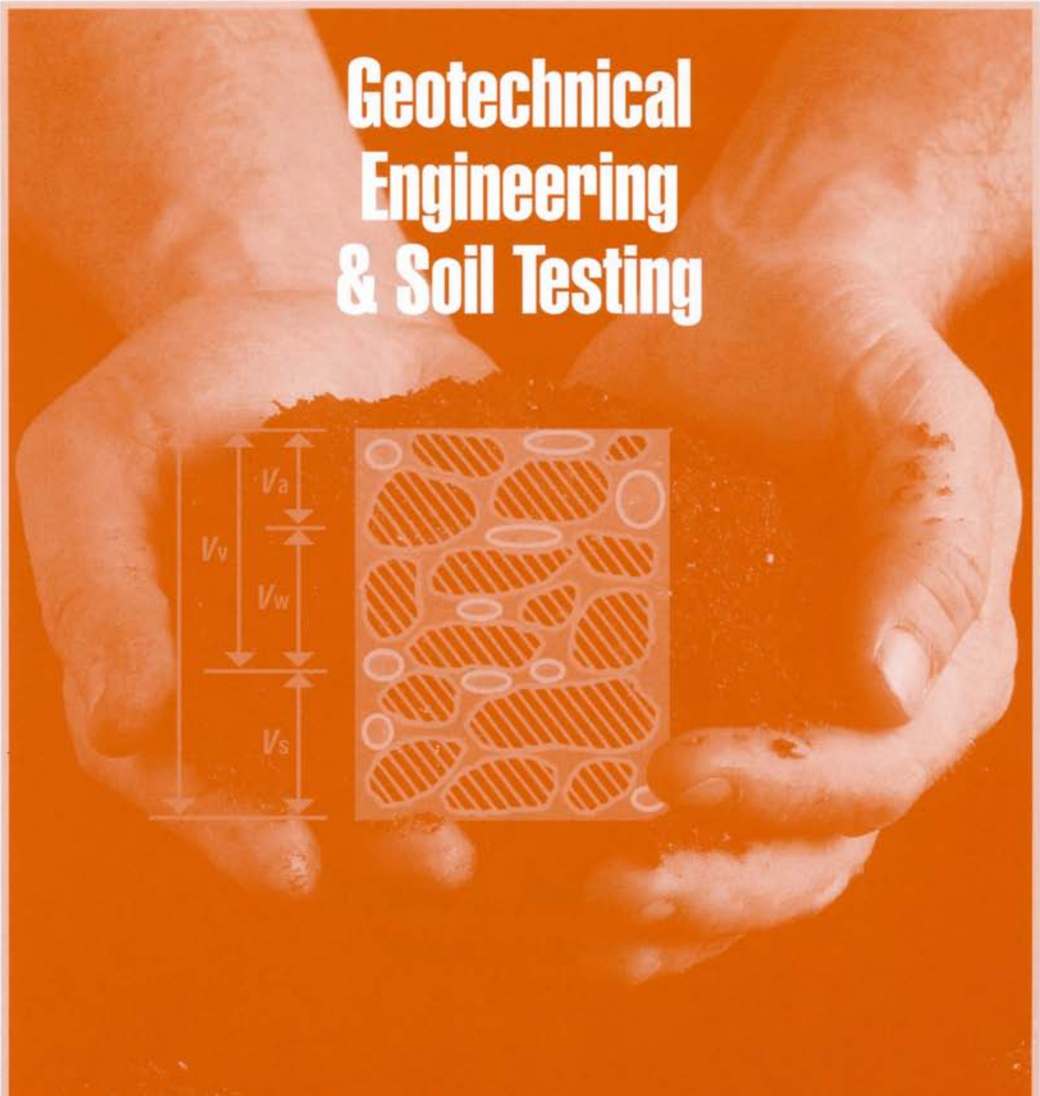

Al-Khafaji & Andersland

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PREFACE

The solutions manual for *Geotechnical Engineering and Soil Testing* is intended for use by instructors and is designed to provide solution to the all of the problems at the end of the chapters. The solutions were developed with the help of several computer programs to insure accuracy. With a few exceptions, the solutions are outlined step by step to maximize the manual value and to reduce instructors effort. However, it is expected that differences in approach may cause questions to be raised relative to some problems. Since this is the first edition, we would appreciate receiving your comments and suggestions for improvement.

Approach ■ The solutions to the problems are designed to provide the instructor with utmost flexibility. It is possible to assign variations of the problems by changing certain parameters and following the same steps to produce additional solutions to additional problems. Since both US customary units and SI units are used, it is suggested that solutions to a given problem may be used as a guide to familiarize students with both systems. That is, the solutions to certain problems may be handed out so that students are exposed to a variety of problems and approaches. It is extremely important that the solution to each problem be examined before the problem is assigned. This will help eliminate misunderstandings and confusion relative to assumptions made in arriving at a particular answer.

Organization: ■ The degree of difficulty with a given problem is independent of its number and/or its location at the end of a given chapter. Some problems are fairly straightforward while others require significant amount of effort and understanding of the subject matter. Several problems may be assigned as group projects and/or be given more weight. Therefore, it is suggested that the introduction to each of the chapters be carefully read before assigning problems.

Acknowledgments ■ The authors are grateful to the many colleagues and students who have contributed significantly and often indirectly to the development of this manual. The quality of this book has been and will continue to be judged by our students and colleagues whose comments and suggestions are greatly appreciated.

Our thanks to Mrs. Emily Barrosse, Senior Acquisition Editor, Ms. Laura Shur, Assistant Editor, and Martha Brown, Project Manager at Saunders College Publishing for their effort. All the people at Saunders College Publishing who were involved with the production of this book deserve a special acknowledgment for their dedication and hard work.

A.W.A.
O.B.A.

To my brother Faris

Whose love of family and traditions has passed the test of time and whose existence lexemplifies the ultimate meaning of life.

Dr. Amir Wadi Al-Khafaji

Chapter 1

Introduction: The solution of problems in this chapter may require literature search and/or additional help from the instructor.

1.1 kaolinite: specific surface = 10 to 20 m^2/g; exchange capacity = 5 me/100 g soil solids; stable structural unit with no swelling or shrinkage.

montmorillonite: specific surface = 800 m^2/g; exchange capacity = 100 me/100 g soil solids; high swelling and shrinkage characteristics.

1.2 Some silica ions in tetrahedral sheets (usually < 15%) may be replaced by aluminum ions with lower valence. These replacements result in the clay particle having a residual negative charge.

1.3 Isomorphous substitution involves substitution of ions of one kind by ions of another type, with the same or different valence, but with retention of the same crystal structure.

1.4 Montorillonite units with comparatively weak bonds, allow water molecules to enter between the sheets causing them to expand. Soil containing substantial amounts of montmorillonite minerals will exhibit high shrinkage and swelling characteristics.

1.5 a) Wind, b) Water, c) Water, d) Ice, e) Wind, f) Gravity

1.6 a) Downwind from large sandy areas, b) Lake shore, c) River deposit, d) Soil deposits at the front of a glacier, e) Glaciated areas f) Downwind from large silt deposits.

1.7 a) Stabilized by vegetation.
b) Sprinkling with pebbles or covering with an asphalt layer.

1.8 Surface area = (specific surface) x (grams of solids).
The amount of water attracted to clay particles is dependent on their surface area.

Activity = (soil plasticity index) / (weight percent particles finer than 2μm).
Soil activity is useful for correlation with certain soil properties.

1.9 a) Foundations: sinkholes or solution cavities will provide very poor bearing capacity and should be avoided.

b) Dams: sinkholes or solution cavities may allow leakage around the dam.

1.10 a) Deep soft clay deposits: piles are required for load transfer to a deeper firm strata.

b) Deep peat or muck deposits: piles are required for load transfer to a deeper firm strata.

Chapter 2

Introduction: The solution of problems in this chapter may require literature search and/or additional help from the instructor.

2.1 The first stipulates that boring depth should be at a point where the stress increment is 5% of the effective overburden. The second is to take boring down to a depth where the stress increment is reduced to 10% of the contact pressure.

2.2 Contact pressure, building type, drainage conditions, accessibility, foundation type (mat, spread, or deep), site geology, cost, ... etc.

2.3 Continuous sampling involves extraction of soil samples without using augers. That is, split spoons and/or shelby tubes are used to extract soils samples throughout the entire depth of a given boring.

2.4 Site geology provides valuable information relative to existing soil formations and type. Site hydrology may alert the geotechnical engineer to potential problems such as artesian conditions and makes it possible to specify the type of equipment needed for the particular exploration program.

2.5 To make sure that utilities are not damaged. The proposed foundations should not cause any settlement or other types of damage to existing structures.

2.6 Because they are less expensive and could provide valuable data on soil types and layers extents for extremely large areas without the need for drilling.

2.7 Seismic surveys are performed at the ground surface and provide information relative to soil and rock materials situated at lower depths.

Resistivity exploration are used to locate or outline gravel deposits and buried aquifers, find depths to a water table, or to locate a change in soil conditions.

2.8 The direct methods provide detailed information relative to soil deposits and permit evaluation of soil properties.

The indirect methods provide qualitative information relative to soil formations and extents. These are cheaper to use than the direct methods.

2.9 The exploratory survey is generally used to obtain preliminary data relative to soils types, layers thicknesses, and water table location. Such survey is relatively inexpensive and may involve use of augers and/or standard tubes. The detailed survey may involve use of shelby tubes and/or split spoon samples. Borings samples are taken at predetermined depths and taken continuously to better assess problematic soil layers found in the exploratory survey.

2.10 The standard penetration test involves advancing a standard sampler (split spoon) using a 140 lb weight dropped a distance of 30 inches. The number of blows required to advance the sample a distance of 12 inches is reported as the blow count. Note that the sampler is advanced a distance of 18 inches and the number of blows required to advance the sampler a distance of 6 inches is re-

Chapter 3

Introduction: Note that P 3.12 requires the assumption that soil particles are perfectly round and may be difficult to solve; P 3.22 assumes a LLR = 1.0; and P 3.23 is designed to test students comprehension of soil classification methods because it can not be solved without information relative to %passing the number 4, 40, and 200 sieve. Also, for P3.24 the student should be advised to assume a LLR =1.0 or any other value deemed appropriate by the instructor.

3.1 Free water can be removed from soil without altering the geological makeup of the soil. Hydration water is part of soil structure and its removal will cause a significant change in soil behavior.

3.2 Capillary rise is estimated using Equation 3.3 as follows:

$$h_c = \frac{1.5}{0.0055} = 272.7 \text{ cm} = 2.7 \text{ m.}$$

For a porosity n = 0.5, the time required is computed as follows:

$$t = \frac{272.7\, n}{10^{-6}}\left[\frac{136.35}{272.7} - \ln\left(1 - \frac{136.35}{272.7}\right)\right] = -26{,}335{,}620 \text{ s} = -305 \text{ days}$$

The pore water pressure at that height is calculated as 2.7(9.81)/2 = 13.3 kN/m^2.

3.3 Capillary rise is estimated using Equation 3.3 as follows:

$$h_c = \frac{1.5}{0.0065} = 230.8 \text{ cm} = 2.3 \text{ m.}$$

For a porosity n = 0.5, the time required is computed as follows:

$$t = \frac{230.8\, n}{10^{-5}}\left[\frac{115.4}{230.8} - \ln\left(1 - \frac{115.4}{230.8}\right)\right] = -13{,}768{,}919 \text{ s} = -159 \text{ days}$$

The pore water pressure at that height is calculated as 2.3(9.81)/2 = 11.3 kN/m^2.

3.4 Fine grained soil particles are platy and extremely small when compared with coarse grained particle. Coarse grained particles are generally rounded and their packing can be classified as dense, loose, or honeycombed.

3.5 Since only ratios are specified, then assume that the volume of solids is 1.0 m^3.

$$e = \frac{V_v}{V_s} = 0.7 \quad \Rightarrow \quad V_v = 0.70 \text{ m}^3$$

$$S_r = \frac{V_w}{V_v} = 0.80 \quad \Rightarrow \quad V_w = 0.80\,(0.70) = 0.56 \text{ m}^3$$

Chapter 4

Introduction: Note that the term "density" and "unit weight" are used interchangeably in the problems. However, they both refers to unit weight of the soil in question. This is consistent with the terminology normally used by practicing geotechnical engineers in the U.S.A. It is suggested that if such terminology is deemed inappropriate, then alert your students that the word "density" appearing in the problem statements refers to unit weight.

4.1 Note that the total weight of 10 lbs is equal to the Weight of water and weight of solids. That is

$$W_w + W_s = 10 \text{ lbs}$$

Since the initial water content is known, then

$$w = 0.112 = \frac{W_w}{W_s} \Rightarrow W_w = 0.112W_s$$

Now solve the above two equations for the weight of solids and weight of water as follows:

$$0.112W_s + W_s = 10 \Rightarrow W_s = 8.993 \text{ lb} \text{ and } W_w = 1.007 \text{ lb}$$

The total amount of water required to achieve a water content of 18% is calculated as

$$W_w = 0.18\,(8.993) = 1.619 \text{ lb}$$

Finally, the net amount of water required is given as $1.619 - 1.007 = 0.611$ lb.

4.2 The percent field compaction achieved is simply the ratio of the field dry density to the laboratory optimum dry density. Hence

$$\%\text{ Compaction} = \frac{111.2}{112.7}(100) = 98.7$$

Clearly, the field % compaction value is greater than the specified value. Therefore, the test is successful.

4.3 From the figure, the optimum dry density is approximately 125 pcf. Specifying 95% compaction implies that the field dry density must be larger than 0.95(125) = 118.75 pcf. Draw a horizontal line corresponding to this value. This line should intersect the curve at water contents of 9.9% and 21.5%.

4.4 (a) The wet unit weight is computed as the ratio of total weight to volume of the soil in the compaction mold. The corresponding dry unit weight is computed as the ratio of the wet unit weight to (1+w). The computed values are listed below:

Chapter 5

Introduction: Several problems require the student to calculate and plot the results. In such problems, it is suggested that one or the other be stipulated. Otherwise, the instructor should take that factor into account when deciding on a suitable number of problems to be solved.

5.1 Measure depth from the ground surface, then

Depth = 0 feet

$\sigma = 0$
$u = 0$
$\overline{\sigma} = 0$

Depth = 15 feet

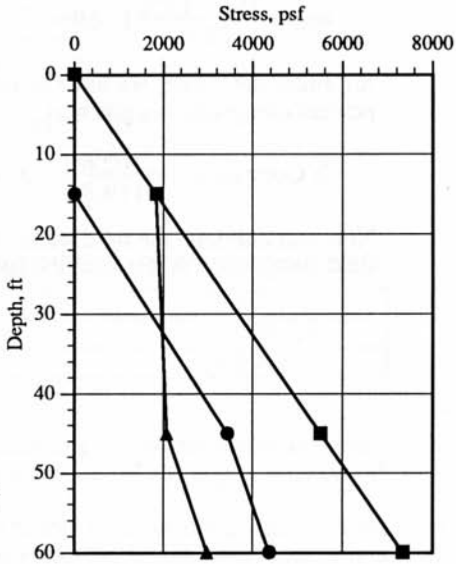
$\sigma = 15\ (122.4) = 1836$ psf
$u = 0$
$\overline{\sigma} = 1836 - 0 = 1836$ psf

Depth = 45 feet

$\sigma = 1836 + 122.4\ (30) = 5508$ psf
$u = 62.4\ (10+15+30) = 3432$ psf
$\overline{\sigma} = 5508 - 3432 = 2076$ psf

Depth = 60 feet

$\sigma = 5508 + 122.4\ (15) = 7344$ psf
$u = 62.4\ (10+15+30+15) = 4368$ psf
$\overline{\sigma} = 7344 - 4368 = 2976$ psf



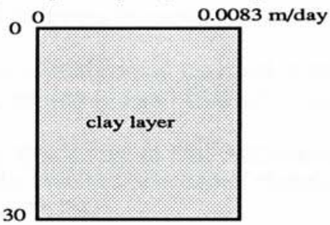
5.2 Assume a unit area of 1.0 m2, then quantity of seepage is computed as follows:

$$q = kiA = 0.01\ \frac{10 + 15}{30}\ (1.0) = 0.083\ m^3/day/m^2$$

The average seepage velocity v_e is given by Equation 5.15 as follows:

$$v_e = \frac{ki}{n}$$

Note that k = 0.01 m/day, i = 25/30, and n is the porosity of the clay layer. Since n is not given, the quantity nv_e can be plotted versus depth as follows:



Chapter 6

Introduction: Note that problems 6.4 and 6.5 were solved using normal stresses of 45 and 15 psi. Problems 6.21 through 6.24 are solved using a depth increment of 5 m. It is recommended that problem 6.28, P 6.30, P 6.33, and P 6.34 be assigned as a group project or individually. These problems are extremely difficult to solve without the aid of a computer.

6.1 Since the applied normal stresses are principal stresses, the analytical solution to this problem is determined using Equation 6.13 as follows:

$$\sigma_\theta = \left(\frac{\sigma_1 + \sigma_3}{2}\right) + \left(\frac{\sigma_1 - \sigma_3}{2}\right)\cos 2\theta$$
$$\tau_\theta = \left(\frac{\sigma_1 - \sigma_3}{2}\right)\sin 2\theta$$

Note that θ is measured counterclockwise from the direction of σ_1 to σ_θ

a) Note that $\sigma_1 = 100$, $\sigma_3 = 50$, and $\theta = 120^\circ$

$$\sigma_\theta = \left(\frac{100+50}{2}\right) + \left(\frac{100-50}{2}\right)\cos 2\,(120) = 62.5 \text{ kN/m}^2$$
$$\tau_\theta = \left(\frac{100-50}{2}\right)\sin 2\,(120) = -21.65 \text{ kN/m}^2$$

b) Note that $\sigma_1 = 100$, $\sigma_3 = 50$, and $\theta = 135^\circ$

$$\sigma_\theta = \left(\frac{100+50}{2}\right) + \left(\frac{100-50}{2}\right)\cos 2\,(135) = 75 \text{ kN/m}^2$$
$$\tau_\theta = \left(\frac{100-50}{2}\right)\sin 2\,(135) = -25 \text{ kN/m}^2$$

c) Note that $\sigma_1 = 100$, $\sigma_3 = 50$, and $\theta = 150^\circ$

$$\sigma_\theta = \left(\frac{100+50}{2}\right) + \left(\frac{100-50}{2}\right)\cos 2\,(150) = 87.5 \text{ kN/m}^2$$
$$\tau_\theta = \left(\frac{100-50}{2}\right)\sin 2\,(150) = -21.65 \text{ kN/m}^2$$

d) Note that $\sigma_1 = 100$, $\sigma_3 = 50$, and $\theta = 165^\circ$

$$\sigma_\theta = \left(\frac{100+50}{2}\right) + \left(\frac{100-50}{2}\right)\cos 2\,(165) = 96.65 \text{ kN/m}^2$$
$$\tau_\theta = \left(\frac{100-50}{2}\right)\sin 2\,(165) = -12.50 \text{ kN/m}^2$$

e) Note that $\sigma_1 = 100$, $\sigma_3 = 50$, and $\theta = 180^\circ$

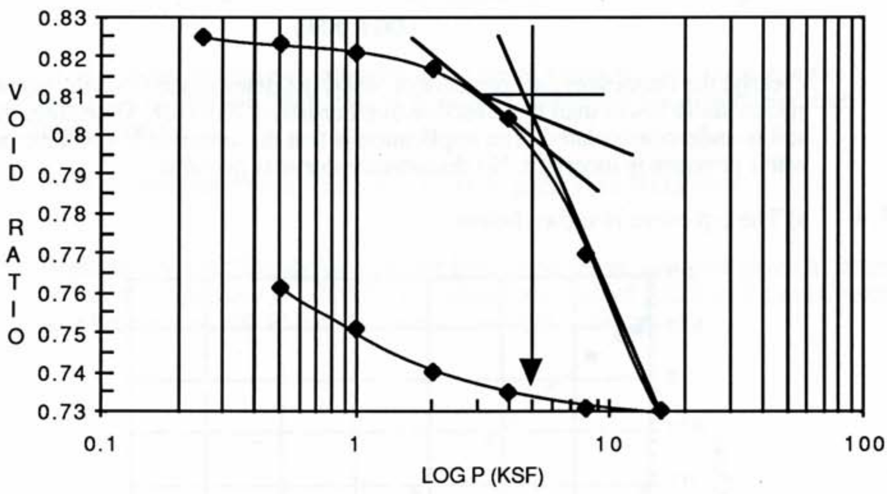
Chapter 7

Introduction: Note that P 7.3 is meant to demonstrate the concept of underconsolidated soils as it relates to settlement calculations. This problem shows that engineers can make mistakes when assessing water table conditions. In this case, the effective overburden is higher than the maximum past pressure. This is possible when the actual water pressure is assumed to be hydrostatic when it is not. The complexity of the consolidation test is demonstrated through P 7.5. In this case the student knowledge of the basic requirements for eo and the effective overburden are tested. These values are required for the consolidation test to be useful. The log t procedure is used in P 7.8. The instructor may chose to specify the square root of time using the same data. Note that a depth increment of 0.1 H is used in P7.14. Problem 7.23 requires significant amount of calculations and an $\alpha = 1/6$ is used.

7.1 Calculate the effective overburden at the given depth then compare it with the maximum past pressure determined from the e-logs as follows:

$$\gamma_{sat} = \frac{(e+G_s)}{(1+e)}\gamma_w = \frac{(1.20 + 2.65)}{1 + 1.20}(62.4) = 109.2 \text{ pcf}$$

$$\bar{\sigma}_{10} = 10\,(109.2 - 62.4) = 468 \text{ psf}$$



Clearly, the maximum past pressure is approximately 5000 psf. This is substantially greater than the effective overburden. Therefore, the soil is Overconsolidated.

7.2 Calculate the effective overburden at the given depth then compare it with the maximum past pressure determined from the e-logs as follows:

$$\bar{\sigma}_{30} = 30\,(109.2 - 62.4) = 1404 \text{ psf}$$

Chapter 8

Introduction: Some of the problems provide more information relative to soil properties than needed. This is to examine students understanding of the basic concepts. P 8.6 requires knowledge of the triaxial test and shear strength concepts.

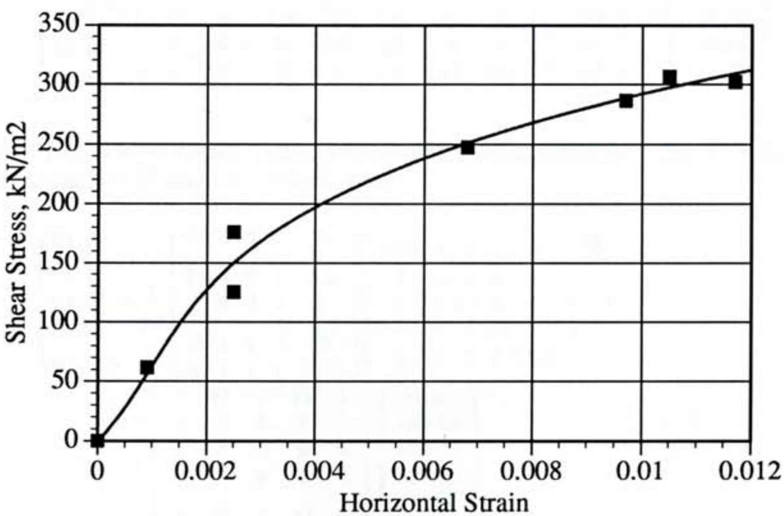
8.1 The normal stress and shear stress are computed as the ratio of applied load to the cross-sectional area as follows:

$$\sigma = \frac{P_v}{A} = \frac{P_v}{0.076\,(0.076)}\ (\text{kN/m}^2)$$

$$\tau = \frac{P_h}{A} = \frac{P_h}{0.076\,(0.076)\,(1000)}\ (\text{kN/m}^2)$$

The vertical strain values are computed as the change in vertical displacement divided by 11 mm and the horizontal strain values are computed as the change in horizontal displacement divided by 76 mm. The result is shown as follows:

Time elapsed (min)	Normal Stress (kN/m^2)	Shear Stress (kN/m^2)	Horizontal Strain (-)	Vertical Strain (-)
0	389.54	0	0	0
0.5	389.54	61.63	0.09%	-0.18%
1	389.54	124.83	0.25%	-0.18%
2	389.54	175.55	0.25%	-0.09%
3	389.54	247.23	0.68%	0.18%
4	389.54	286.53	0.97%	0.55%
5	389.54	306.44	1.05%	0.36%
6	389.54	301.94	1.17%	0.18%



Chapter 9

Introduction: The majority of the problems are straight forward. However, P 9.6 requires additional expressions not found in the textbook. These are provided with the solution and the instructor is advised to either provide them with the problems or to require students to ascertain them through literature search. Problem 9.10 through 9.15 requires access to precise drawing tools.

9.1 Note that all of the answers are given per one foot of wall.

a) $K_a = \tan^2\left(45 - \frac{30}{2}\right) = 0.33$ $E_a = 100\left(\frac{10^2}{2}\right)(0.33) = 1666.67$ lb	e) $K_a = \tan^2\left(45 - \frac{30}{2}\right) = 0.33$ $E_a = 18.2\left(\frac{4^2}{2}\right)(0.33) = 48.5$ kN
b) $K_a = \tan^2\left(45 - \frac{35}{2}\right) = 0.27$ $E_a = 100\left(\frac{10^2}{2}\right)(0.27) = 1355$ lb	f) $K_a = \tan^2\left(45 - \frac{35}{2}\right) = 0.27$ $E_a = 18.2\left(\frac{4^2}{2}\right)(0.27) = 39.3$ kN
c) $K_a = \tan^2\left(45 - \frac{30}{2}\right) = 0.33$ $E_a = 110\left(\frac{10^2}{2}\right)(0.33) = 1833$ lb	g) $K_a = \tan^2\left(45 - \frac{30}{2}\right) = 0.33$ $E_a = 17.4\left(\frac{4^2}{2}\right)(0.33) = 46.4$ kN
d) $K_a = \tan^2\left(45 - \frac{35}{2}\right) = 0.27$ $E_a = 110\left(\frac{10^2}{2}\right)(0.27) = 1485$ lb	h) $K_a = \tan^2\left(45 - \frac{35}{2}\right) = 0.27$ $E_a = 17.4\left(\frac{4^2}{2}\right)(0.27) = 37.6$ kN

9.2 Note that all of the answers are given per one foot of wall.

a) $K_p = \tan^2\left(45 + \frac{30}{2}\right) = 3.00$ $E_p = 100\left(\frac{10^2}{2}\right)(3.0) = 15000$ lb	c) $K_p = \tan^2\left(45 + \frac{30}{2}\right) = 3.00$ $E_p = 18.2\left(\frac{4^2}{2}\right)(3.00) = 436.8$ kN
b) $K_p = \tan^2\left(45 + \frac{35}{2}\right) = 3.69$ $E_p = 110\left(\frac{10^2}{2}\right)(3.69) = 20296$ lb	d) $K_p = \tan^2\left(45 + \frac{35}{2}\right) = 3.69$ $E_p = 17.4\left(\frac{4^2}{2}\right)(3.69) = 513.6$ kN

9.3 Note that all of the answers are given per one foot of wall.

Chapter 10

Introduction: The solutions to problems 10.11 and 10.12 require extremely large number of steps. These are meant to demonstrate the fallacy of using a factor of safety of 2.0 to 3.0 when dealing with bearing capacity problems. These problems show that an allowable bearing pressure must be small enough so that the settlement is within acceptable limits. Most foundations textbooks ignore the fact that the allowable bearing pressure is related to both settlement and shear strength. It is recommended that these two problems be assigned as a group project or that the handed out to further students understanding of this important concept.

10.1 Note that for a strip footing L/B is assumed to be infinite. Using Table 10.1, we have for general shear

$$q_{ult} = cN_c + q\,N_q + \frac{1}{2}\gamma\,BN_\gamma$$

From Figure 10.4, with $\phi = 32$, read $N_c = 40$, $N_q = 25$, and $N_\gamma = 25$

$$q_{ult} = 750\,(40) + 4\,(112)(25) + \frac{1}{2}(112)(5)(25) = 48200 \text{ psf}$$
$$q_{allowable} = \frac{q_{ult}}{3} = \frac{48200}{3} = 16{,}067 \text{ psf}$$

Note that the allowable value is high and may produce more settlement than can be tolerated.

10.2 Using Table 10.1, we have for general shear

$$q_{ult} = 1.3\ \ cN_c + q\,N_q + 0.4\,\gamma\,BN_\gamma$$

From Figure 10.4 with $\phi = 30$, read $N_c = 32$, $N_q = 20$, and $N_\gamma = 16$

$$q_{ult} = 1.3\ \ (50)(32) + 1.2\,(17.6)(20) + 0.4\,(17.6)(1.5)(16) = 2671 \text{ kN/m}^2$$
$$q_{allowable} = \frac{q_{ult}}{3} = \frac{2671}{3} = 890 \text{ kN/m}^2$$

10.3 Using Table 10.1, we have for general shear

$$q_{ult} = 1.3\ \ cN_c + q\,N_q + 0.3\,\gamma\,BN_\gamma$$

Note that 1.0 m = 0.305 ft. From Figure 10.4 with $\phi = 27$, read $N_c = 25$, $N_q = 14$, and $N_\gamma = 10.2$

$$q_{ult} = 1.3\,(73)(25) + 0.305\,(4)(17.9)(14) + 0.3\,(0.305)(17.9)(5)(10.2)$$
$$= 2762 \text{ kN/m}^2$$

$$q_{allowable} = \frac{q_{ult}}{2.5} = \frac{2762}{2.5} = 1105 \text{ kN/m}^2$$

Chapter 11

Introduction: The solution to problem 11.6 is a bit difficult and may cause students to spend much time. Problems 11.7 and 11.8 involve significant amount of number crunching and may be altered to meet instructional needs. Although the solution is given assuming a zero for the angle of internal friction, additional information is provided so that soils with both ϕ and c can be analyzed with relative ease.

11.1 The factor of safety for an infinite slope is given by Equation 11.5 as follows:

$$FS = \frac{\left(\gamma D \cos^2\beta - \gamma_w h_p\right)\tan\bar{\phi}}{\gamma D \cos\beta \sin\beta} = \frac{\tan\bar{\phi}}{\tan\beta}\left(1 - \frac{h_p \gamma_w}{D\gamma \cos{}^2\beta}\right)$$

Note that in this case, the pressure head is equal to the thickness of the soil layer. That is $h_p = D$. Substituting into the expression gives

$$FS = \frac{\tan\bar{\phi}}{\tan\beta}\left(1 - \frac{\gamma_w}{\gamma \cos{}^2\beta}\right)$$

11.2 Since water will from vertically to the blanket, then u=0 and h_p =0. Substituting into Equation 11.11 gives

$$FS = \frac{\bar{c}}{\gamma D \cos\beta \sin\beta} + \frac{\tan\bar{\phi}}{\tan\beta}$$

11.3 Note that in this case, the pressure head is equal to the thickness of the soil layer. That is $h_p = D\text{-}D_w$. Substituting into Equation 11.11 gives

$$FS = \frac{\bar{c}}{\gamma D \cos\beta \sin\beta} + \frac{\tan\bar{\phi}}{\tan\beta}\left(1 - \frac{\left(D - D_w\right)\gamma_w}{D\gamma \cos{}^2\beta}\right)$$

11.4 Note that in this case, the pressure head is equal to the thickness of the soil layer. That is $h_p = D$. Substituting into Equation 11.11 gives

$$FS = \frac{\bar{c}}{\gamma D \cos\beta \sin\beta} + \frac{\tan\bar{\phi}}{\tan\beta}\left(1 - \frac{\gamma_w}{\gamma \cos{}^2\beta}\right)$$

11.5 A cross-section is shown below for an infinite slope making a β angle with the horizontal and flow lines making α angle with the horizontal,

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