Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

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Bearing Capacity (bearing load) of Soil (1)

- Review of shear strength,
- Definition
- Terzaghi equation (approach)
  - Bearing capacity factors
  - Assumptions

Review of shear strength of soil

The shear strength of a soil determined according to Mohr–Coulomb failure criterion , *defined in terms of effective stress*, as :

$$s = c' + \sigma' \tan \emptyset'$$

where

σ' = effective normal stress on plane of shearing
c' = cohesion, or apparent cohesion
ϕ' = effective stress angle of friction

- ✓ the shear strength parameters of a soil (i.e., c' and $\emptyset'$) are determined by two standard laboratory tests:
  - ➢ the direct shear test and
  - ➢ the triaxial test.

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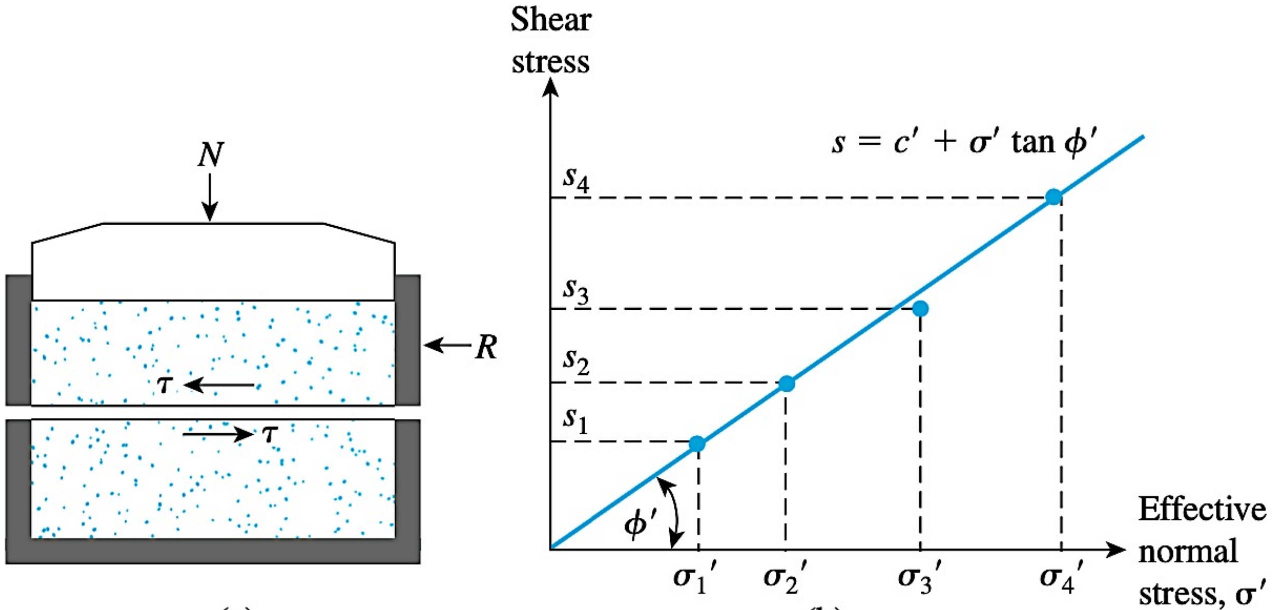
Direct shear test

Dry sand can be conveniently tested by direct shear tests. The sand is placed in a shear box that is split into two halves (Figure a). First a normal load is applied to the specimen. Then a shear force is applied to the top half of the shear box to cause failure in the sand. The normal and shear stresses at failure are (Figure b):

$$\sigma' = \frac{N}{A} \quad \& \quad s = \frac{R}{A}$$

where A = area of the failure plane in soil—that is, the cross-sectional area of the shear box.



(a) (b)

$$\emptyset' = tan^{-1}\left(\frac{s}{\sigma'}\right)$$

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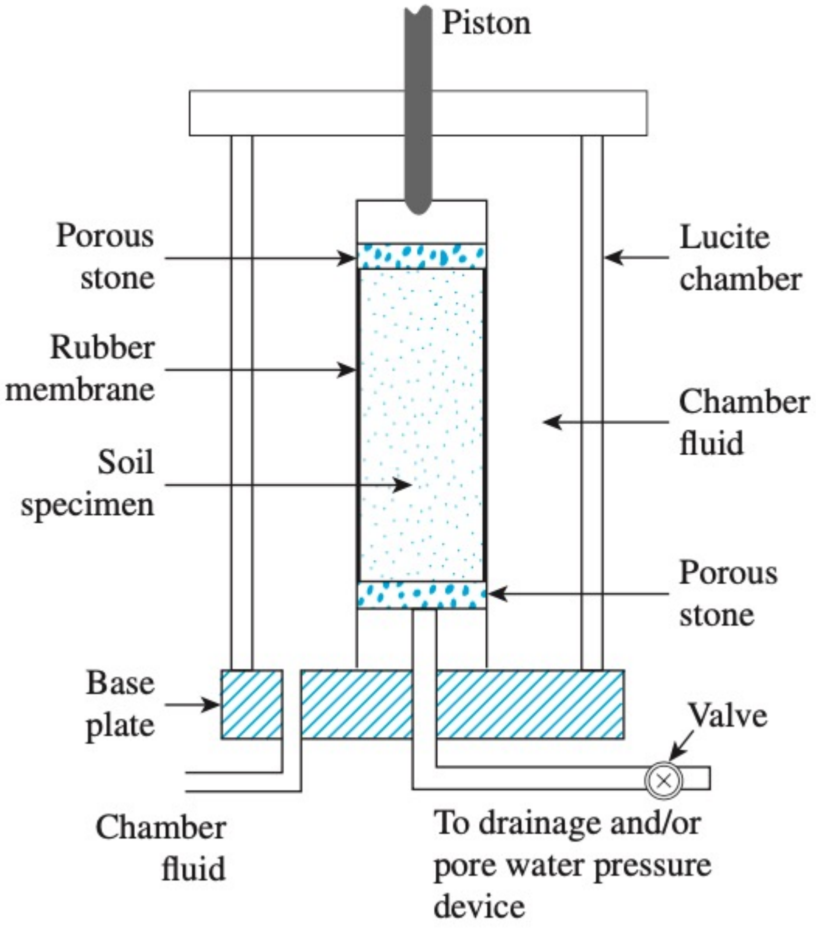
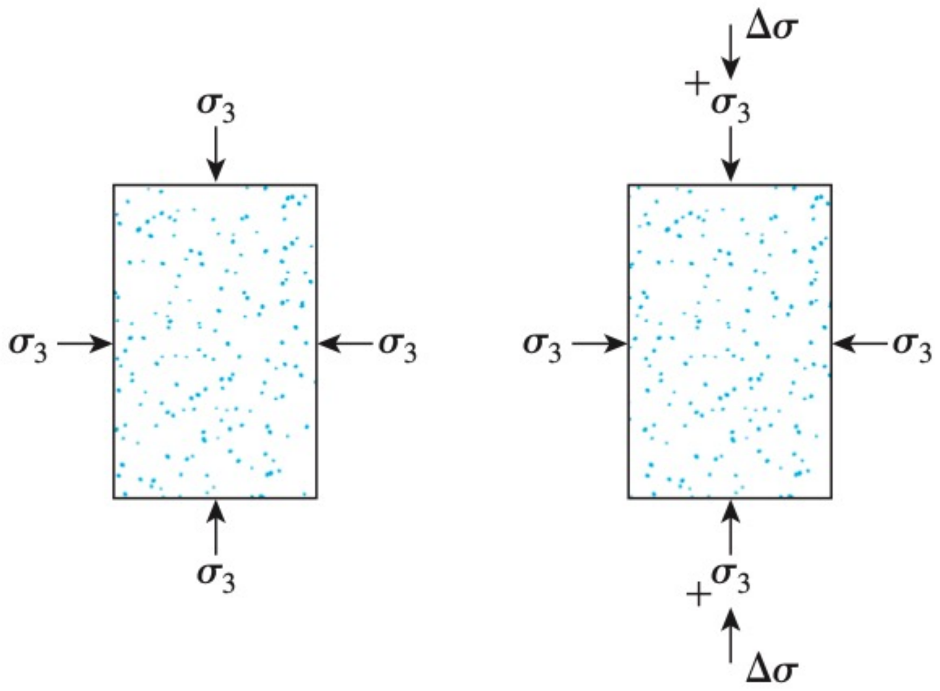
Triaxial tests

Triaxial compression tests can be conducted on sands and clays (Figure a).



Schematic diagram of triaxial test equipment

(a)

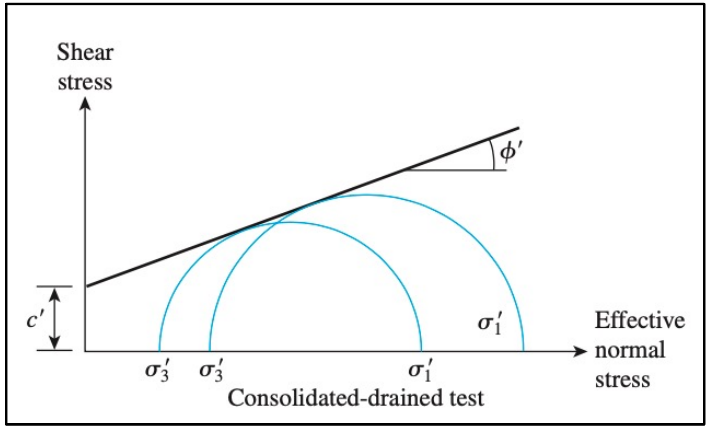


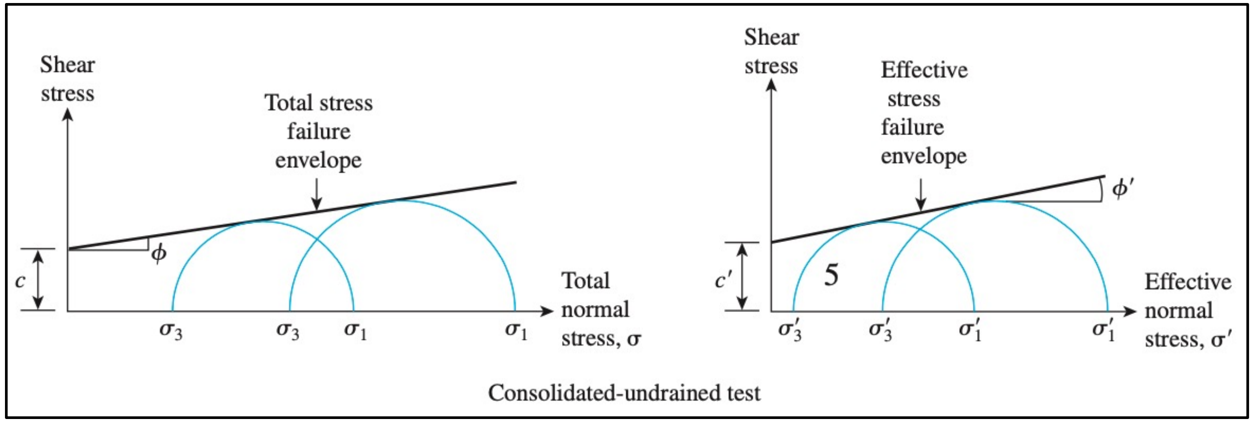
Sequence of stress application in triaxial test

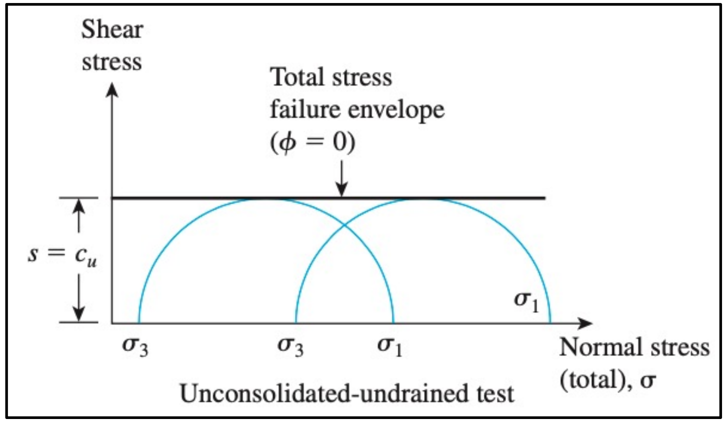
Triaxial tests, cont'd

Test consists of placing a soil specimen confined by a rubber membrane into a lucite chamber and then applying an all-around confining pressure σ_3 to the specimen by means of the chamber fluid (generally, water or glycerin). An added stress $\Delta\sigma$ can also be applied to the specimen in the axial direction to cause failure ($\Delta\sigma = \Delta\sigma_f$ at failure). Drainage from the specimen can be allowed or stopped, depending on the condition being tested. For clays, three main types of tests can be conducted with triaxial equipment (see Figures below):

1) Consolidated-drained test (CD test)
2) Consolidated-undrained test (CU test)
3) Unconsolidated-undrained test (UU test).







Unconfined Compression Test

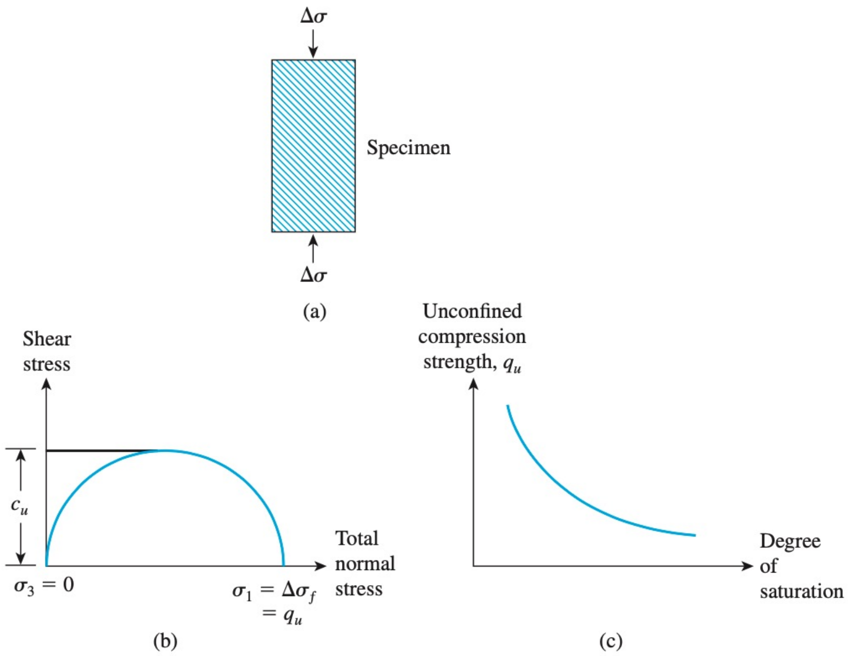
The *unconfined compression test* (Fig. a) is a special type of unconsolidated- undrained triaxial test in which the confining $pressure\ \sigma_3 = 0$, as shown in (Fig. b). In this test, an axial stress $(\Delta\sigma)$ is applied to the specimen to cause failure ($($i.e., $\Delta\sigma = \Delta\sigma_f)$. The corresponding Mohr's circle is shown in Fig. b. Note that, for this case,

Major principal total stress = $(\Delta\sigma_f = q_u)$

Minor principal total stress = 0

The axial stress at failure, $\Delta\sigma_f = q_u$, is generally referred to as the *unconfined compression strength*. The shear strength of saturated clays under this condition $(\emptyset = 0)$ is :

$$s = c_u = \frac{q_u}{2}$$



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Final comments on shear strength parameters (∅ and c)

- For sands, the angle of friction usually ranges from 26° to 45°, increasing with the relative density of compaction. A general range of the friction angle, $\emptyset'$, for sands is given in the Table.

State of packing	Relative density (%)	Angle of friction, ϕ' (deg.)
Very loose	<15	<28
Loose	15–35	28–30
Compact	35–65	30–36
Dense	65–85	36–41
Very dense	>85	>41

- The value of c':
  - ✓ for sands and normally consolidated clays is equal to zero.
  - ✓ for overconsolidated clays, $c' > 0$.

- Sensitivity:

  For many naturally deposited clay soils, the unconfined compression strength is much less when the soils are tested after remolding without any change in the moisture con- tent. This property of clay soil is called *sensitivity*. The degree of sensitivity is the ratio of the unconfined compression strength in an undisturbed state to that in a remolded state, or :

$$S_t = \frac{q_{u\,(undisturbed\ soil)}}{q_{u\,(remolded\ soil)}}$$

The sensitivity ratio of most clays ranges from about 1 to 8

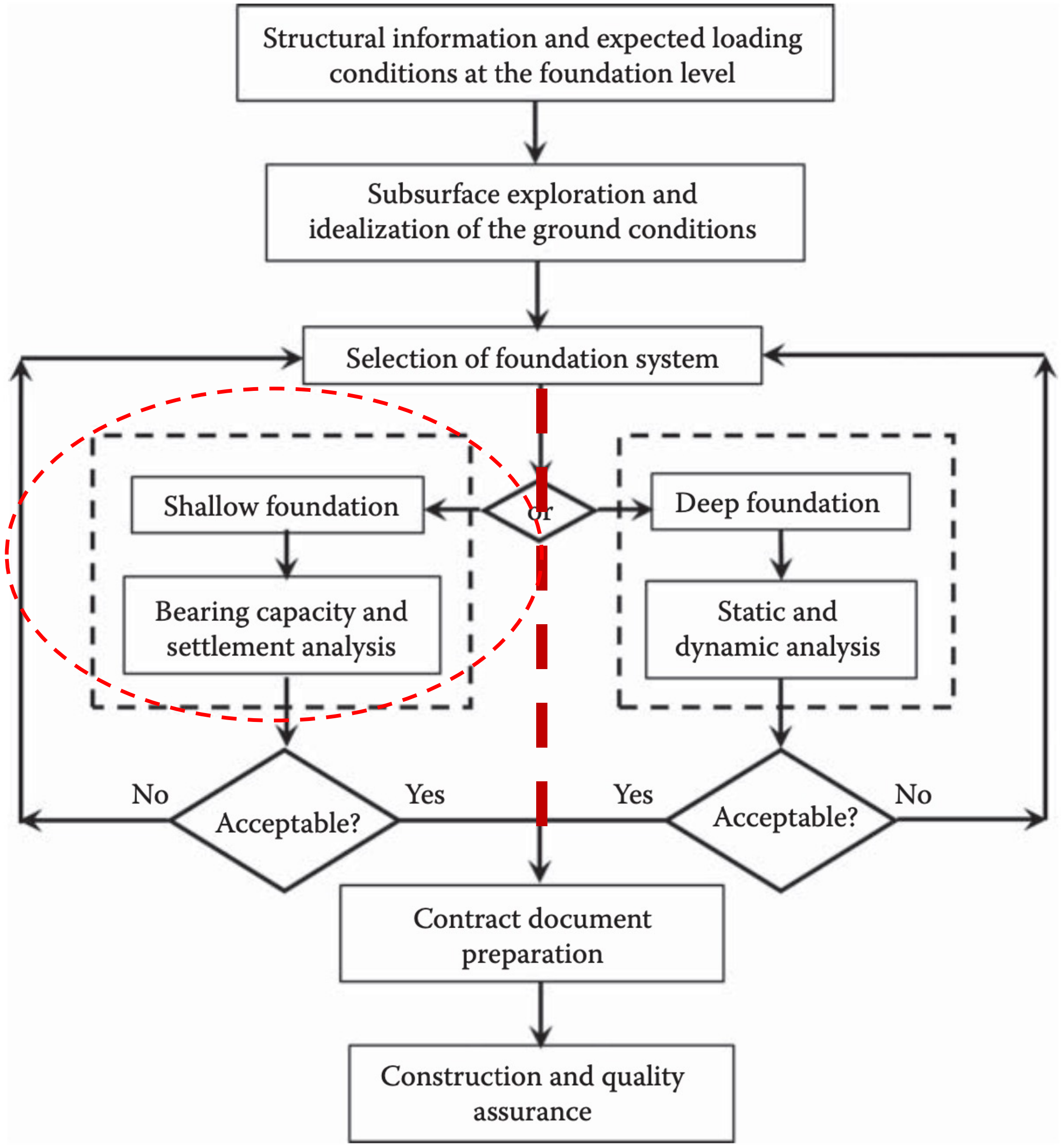
Bearing Capacity (Allowable stress) of the soil for shallow foundation

The most important considerations in foundation design are to ensure:
1.The safety of the foundation against soil failure (ultimate limit state), i.e. shear failure.
2. The functionality of the foundation and the structure above by minimizing the foundation movement and distortion (serviceability limit state), i.e. failure due to settlement.
3. The safety of the foundation against structural failure.
The items (1) and (2) above are of the geotechnical concept, while the 3^{rd} item is of structural concern.

Mainly, two types of foundation:
1) Shallow Foundation (Spread, strip and wall footings, combined and raft (mat) foundation).
2) Deep Foundation (Pile foundation)

Here, in this course, the bearing capacity of shallow foundation will be covered.
The flow chart in the following slide shows the steps that should be followed for this purpose.

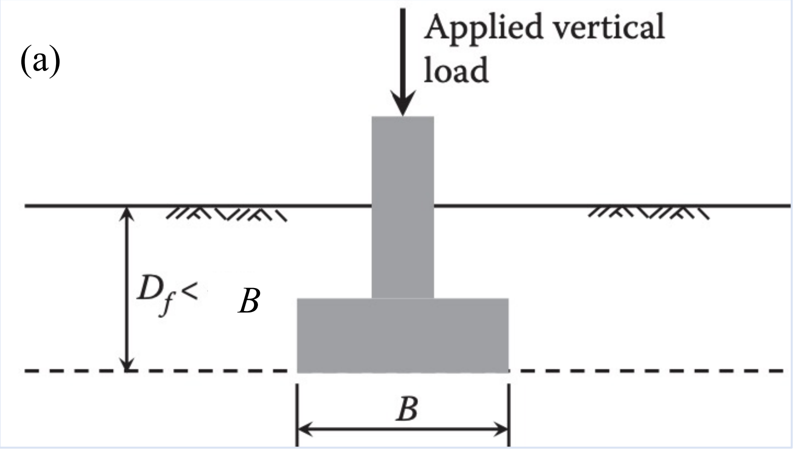
Bearing Capacity (Allowable stress) of the soil for shallow foundation, cont'd

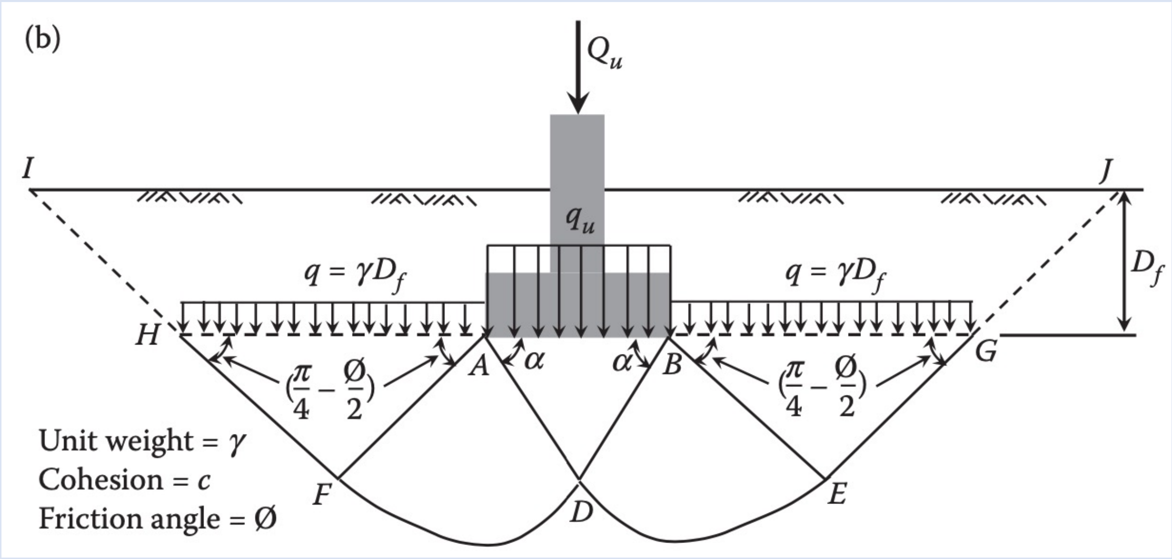


What is the Ultimate Bearing Capacity (q_u) ?

Ultimate Bearing Capacity: the load per unit area of the foundation at which ***shear failure*** in soil occurs.

In the Figure (a) shows a cross-sectional view of a shallow strip foundation subjected to a vertical load. It is obvious that the settlement of the foundation will increase with the applied vertical load. When the vertical load is increased to certain level, the foundation will collapse due to shear failure of the soil supporting it. To ensure stability in foundation design, it is most important that for a given soil condition, it should to predict or estimate the level of load, Q_u, at which the foundation collapse would occur, and the corresponding pressure (Q_u divided by the foundation area) is referred to as the **ultimate bearing capacity q_u** (Figure b).



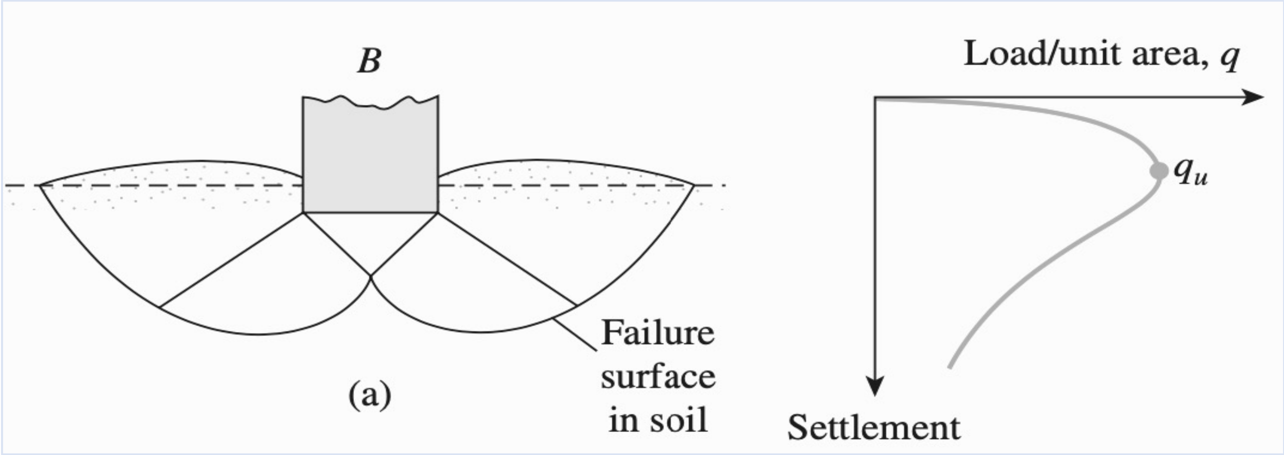


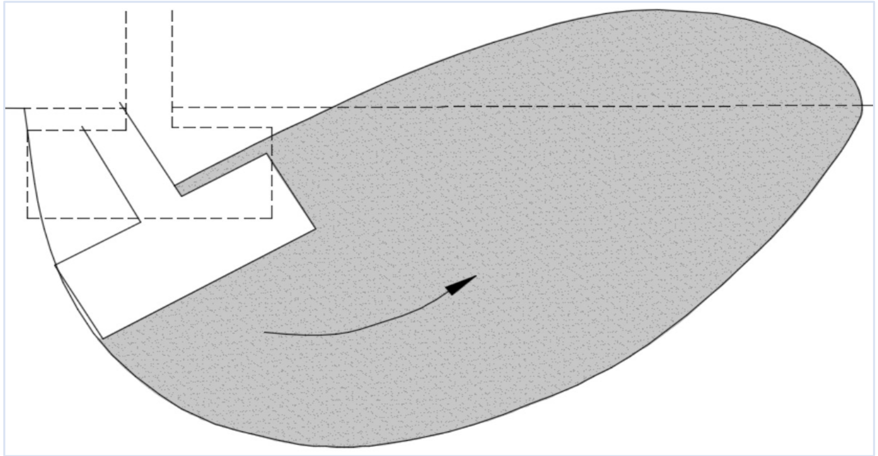
Types of shear failure:

1) General Shear Failure:

Characteristics of general shear failure:

- Occurs over dense sand or stiff cohesive soil.
- Involves total rupture of the underlying soil.
- There is a continuous shear failure of the soil from below the footing to the ground surface (solid lines on the figure).
- The ultimate bearing capacity has been defined as the bearing stress that causes **a sudden** failure of the foundation.
- As shown in the figure, a general shear failure ruptures occur and pushed up the soil surface on both sides of the footing.



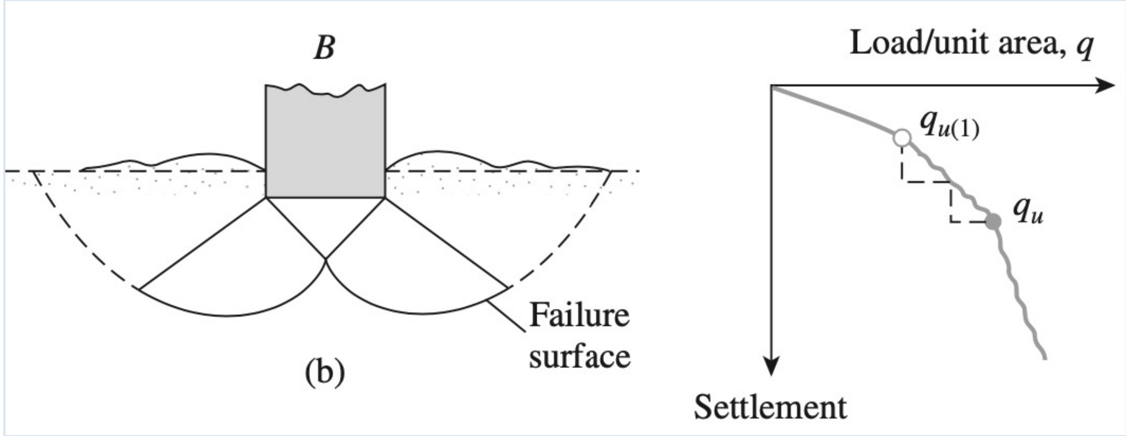


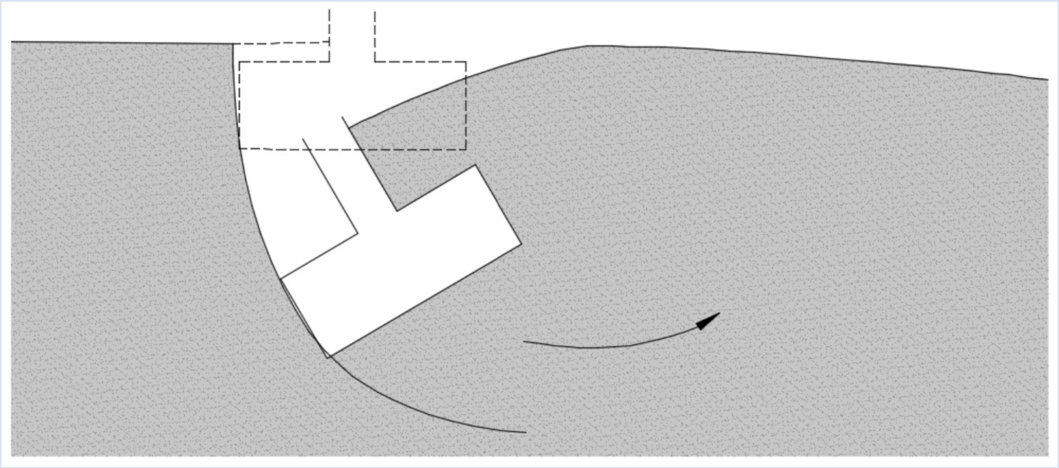
2) Local Shear Failure:

characteristics of local shear failure:

- Occurs over sand or clayey soil of medium compaction.
- Involves rupture of the soil only immediately below the footing.
- There is soil bulging on both sides of the footing, but the bulging is not as significant as in general shear.
- The failure surface of the soil will gradually (not sudden) extend outward from the foundation (not the ground surface) as shown by solid lines in the figure.
- So, local shear failure can be considered as a transitional phase between general shear and punching shear.

  Because of the transitional nature of local shear failure, the ultimate bearing capacity could be defined as the firs failure load ($q_{u,1}$) which occurs at the point which have the first measure nonlinearity in the load/unit area- settlement curve (open circle), or at the point where the settlement starts rabidly increase (q_u) (closed circle).
- This value of (q_u) is the required (load/unit area) to extends the failure surface to the ground surface (dashed lines in the above figure).
- In this type of failure, the value of (q_u) is not the peak value so, this failure called (Local Shear Failure).
- The actual local shear failure in field is proceed as shown in the figure:





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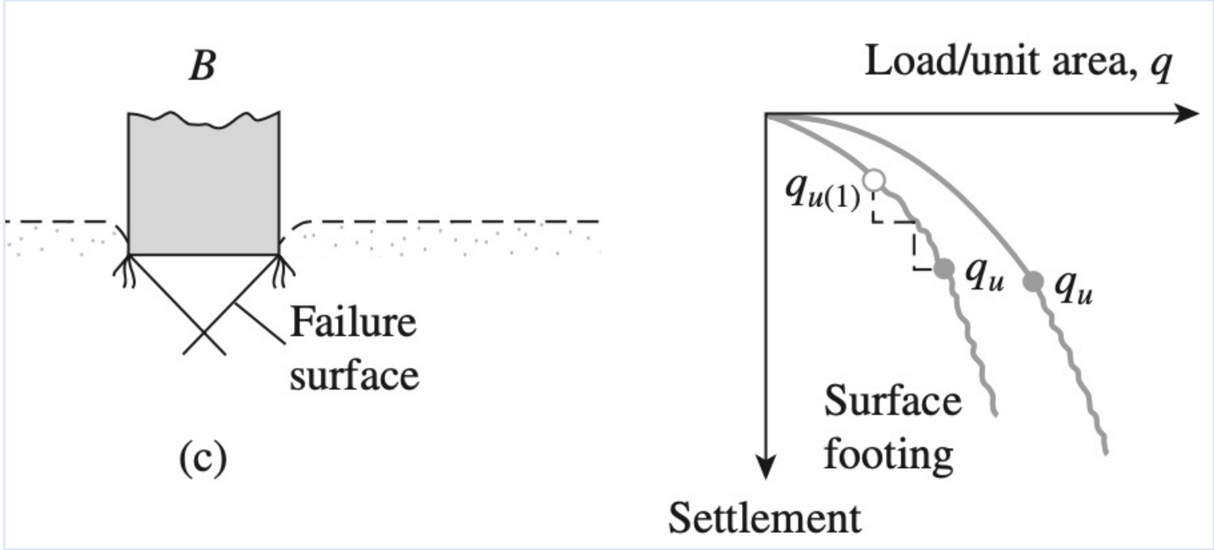
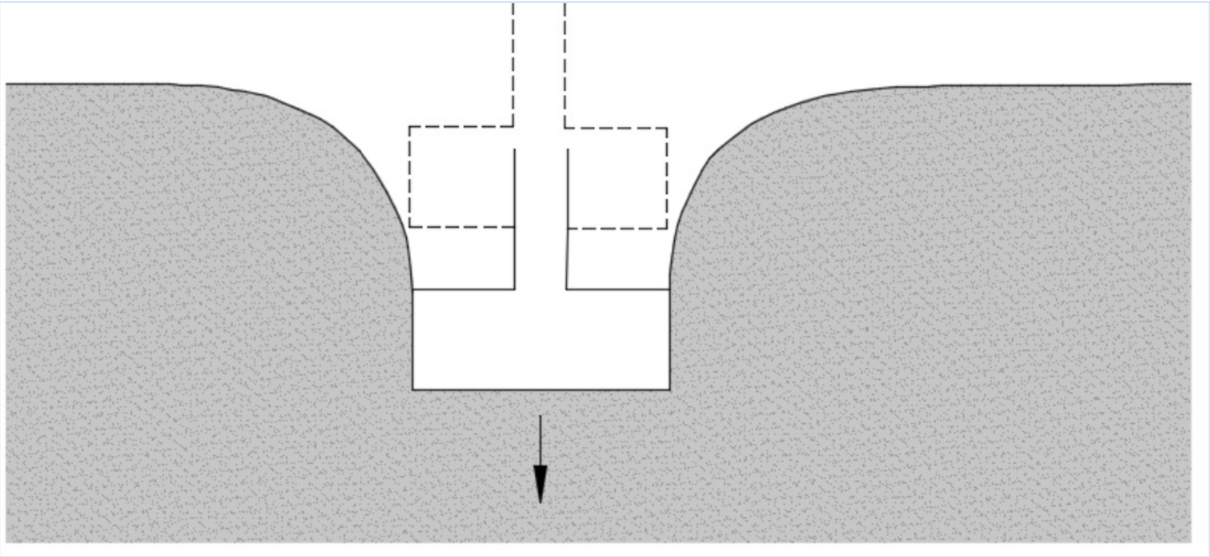
3) Punching Shear Failure:

characteristics of punching shear failure:

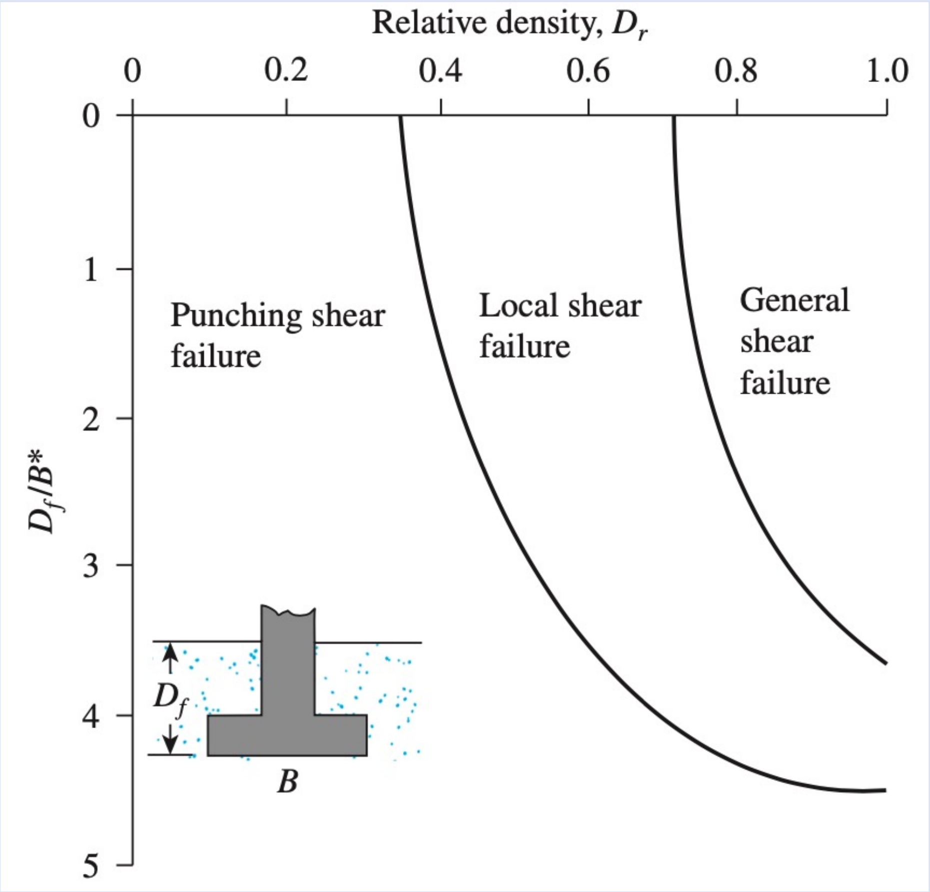
- Occurs over fairly loose soil.
- Punching shear failure does not develop the distinct shear surfaces associated with a general shear failure.
- The soil outside the loaded area remains relatively uninvolved and there is a minimal movement of soil on both sides of the footing.
- The process of deformation of the footing involves compression of the soil directly below the footing as well as the vertical shearing of soil around the footing perimeter.
- As shown in the figure, the (q_u)-settlement curve does not have a dramatic break and the bearing capacity is often defined as the first measure nonlinearity in the (q)-settlement curve($q_{u,1}$).
- Beyond the ultimate failure (load/unit area) ($q_{u,1}$), the (load/unit area)- settlement curve will be steep and practically linear.
- The actual punching shear failure in field is proceed as shown in the figure:





Types of shear failure, cont'd

A proposed relationship for the mode of bearing capacity failure of foundations resting on sands. The Figure shows this relationship, which involves the notation:



D_r = relative density of sand
D_f = depth of foundation measured from the ground surface

$$B^* = \frac{2BL}{B + L}$$

where

B = width of foundation
L = length of foundation

(*Note:* L is always greater than B.)

For square foundations, $B = L$; for circular foundations, $B = L =$ diameter, so

$$B^* = B$$

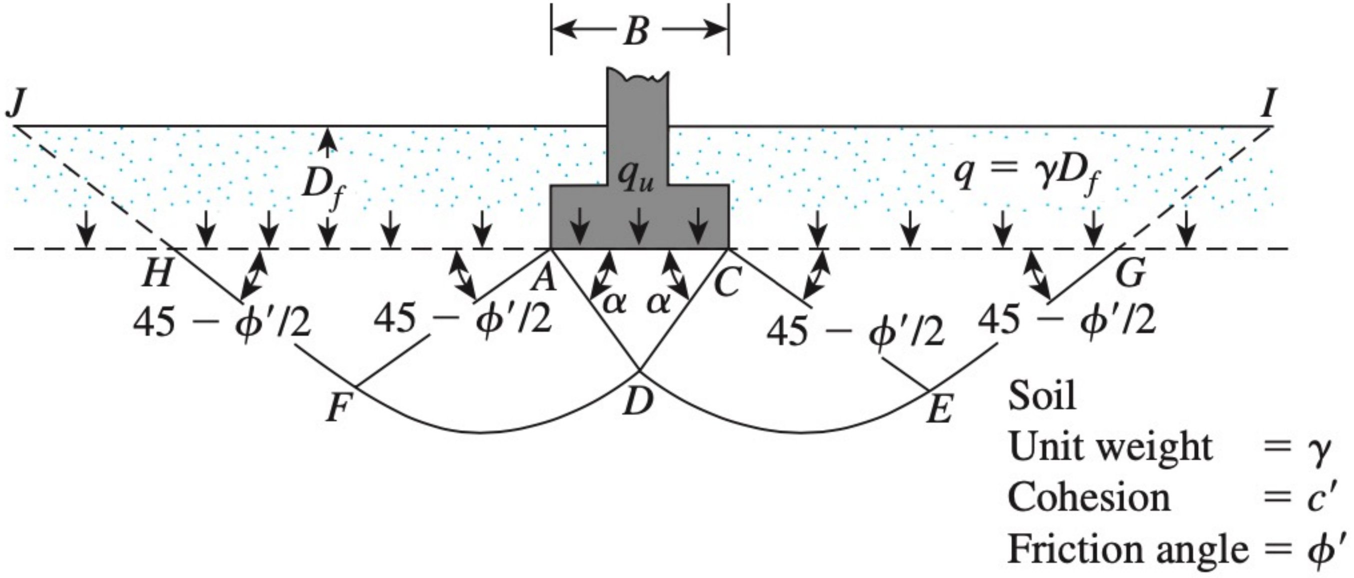
Terzaghi's Approach (Solution) for Bearing Capacity

Terzaghi (1943) first analyzed the problem of determination of bearing capacity of the soil using the limit equilibrium method in which the contributions from:

- ✓ soil cohesion (c'),
- ✓ Surcharge ((γD_f) or q), and
- ✓ soil unit weight (γ)

are superimposed.

According to Terzaghi, the failure surface under loading subjected to a shallow foundation is as shown in the Figure:



Terzaghi's Approach (Solution) for Bearing Capacity, cont'd

The failure zone under the foundation can be separated into three parts (see the Figure above):

1. The *triangular zone ACD* immediately under the foundation,

2. The *radial shear zones ADF* and *CDE,* with the curves *DE* and *DF* being arcs of a logarithmic spiral, and

3. Two triangular *Rankine passive zones AFH* and *CEG*.

The angles *CAD* and *ACD* are assumed to be equal to the soil friction angle $\emptyset'$.
The ultimate bearing capacity, q_u, of the foundation now can be obtained by considering the equilibrium of each element of the failure zones, and then it can be written as in the following expression (equation):

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma B N_\gamma$$

where, N_c, N_q, and N_γ are the bearing capacity factors

The bearing capacity factors N_c, N_q, and N_γ are, respectively, the contributions of cohesion, surcharge, and unit weight of soil to the ultimate load-bearing capacity.

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Terzaghi's Approach (Solution) for Bearing Capacity, cont'd

Bearing-capacity factors for the Terzaghi equations

Values of N_γ for ϕ of 0, 34, and 48° are original Terzaghi values and used to back-compute $K_{p\gamma}$

ϕ, deg	N_c	N_q	N_γ	$K_{p\gamma}$
0	5.7*	1.0	0.0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	
50	347.5	415.1	1153.2	800.0

*$N_c = 1.5\pi + 1$. [See Terzaghi (1943), p. 127.]

$$N_q = \frac{a^2}{a\cos^2(45 + \phi/2)}$$

$$a = e^{(0.75\pi - \phi/2)\tan\phi}$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = \frac{\tan\phi}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)$$

Terzaghi's Approach (Solution) for Bearing Capacity, cont'd

Table 6.1 Terzaghi's Bearing Capacity Factors—Eqs. (4.15), (4.13), and (4.11).[a]

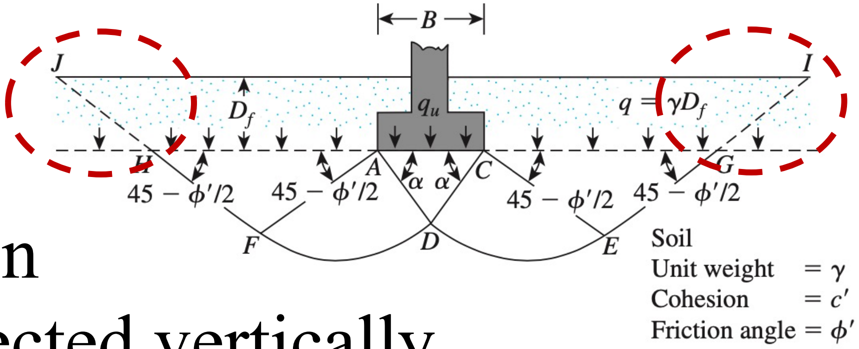
ϕ'	N_c	N_q	N_γ[a]	ϕ'	N_c	N_q	N_γ[a]
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

[a]From Kumbhojkar (1993)

Terzaghi's Approach (Solution) for Bearing Capacity, cont'd

Terzaghi's theory is based on **the following assumptions**:

1. The foundation is considered to be shallow if ($D_f \leq B$).
2. The foundation is considered to be strip or continuous if ($B/L \rightarrow 0.0$). (Width to length ratio is very small and goes to zero), and the derivation of the equation is to a strip footing.
3. The effect of soil above the bottom of the foundation may be assumed to be replaced by an equivalent surcharge ($q = \gamma D_f$). So, the shearing resistance of this soil along the failure surfaces is neglected (Lines GI and HJ in the figure)
4. The failure surface of the soil is similar to general shear failure (i.e. equation is derived for general shear failure) as shown in the figure.
5. The foundation is rigid enough to resist the structural failure.
6. The base of foundation is rough so that it insures the interaction between soil and foundation



7. The applied load on foundation should be subjected vertically

Terzaghi's Approach (Solution) for Bearing Capacity, cont'd

Note:

1. In recent studies, investigators have suggested that, foundations are considered to be shallow if [$D_f \leq (3 \rightarrow 4)B$], otherwise, the foundation is deep.
2. Always the value of (q) is the effective stress at the bottom of the foundation.

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Bearing Capacity (bearing load) of Soil (5)

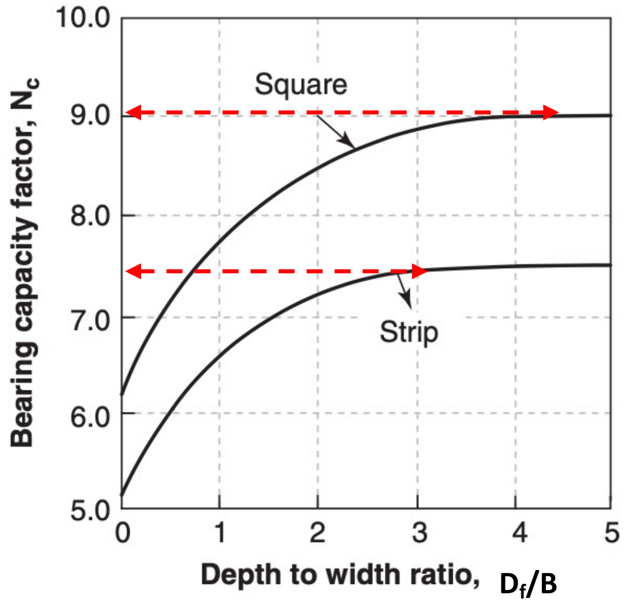
- Skempton’s equation
- Eccentricity:
  - One-way (eccentric in one direction or one axis)

Skempton's equation

The first direct strength equation was proposed by Skempton (1951);

- ✓ The problem of the undrained ultimate bearing capacity of a shallow foundation on a fine-grained soil.
- ✓ The equation makes use of the average undrained shear strength s_u within the depth of influence below the footing.

The equation is: $q_u = N_c\, S_u + \gamma D_f$



Where:

N_c is the bearing capacity factor (the Figure) proposed by Skempton,

γ is the total unit weight of the soil above the foundation depth, and

D_f is the depth of embedment (depth of foundation).

Note that N_c is higher for square footings than for strip footings.

The N_c values for the square footing and the strip footing are related by:

$$N_{c(\text{square})} = 1.2\, N_{c(\text{strip})}$$

Skempton's equation

1) Why does N_c of square footing greater than that of strip footing?
2) From the figure, it is noticed that N_c increases gradually with embedment ratio, why?
3) S_u in Skempton equation for undrained fine soil, is it equals to C_u?
4) C_u determined by uniaxial compression (unconfined compression) test = $q_{unconfined}/2$, so is it correct that the net allowable bearing capacity when FS = 3 equals, approximately, the unconfined compression pressure for square footing with embedded ratio =0?

Example (5-1)

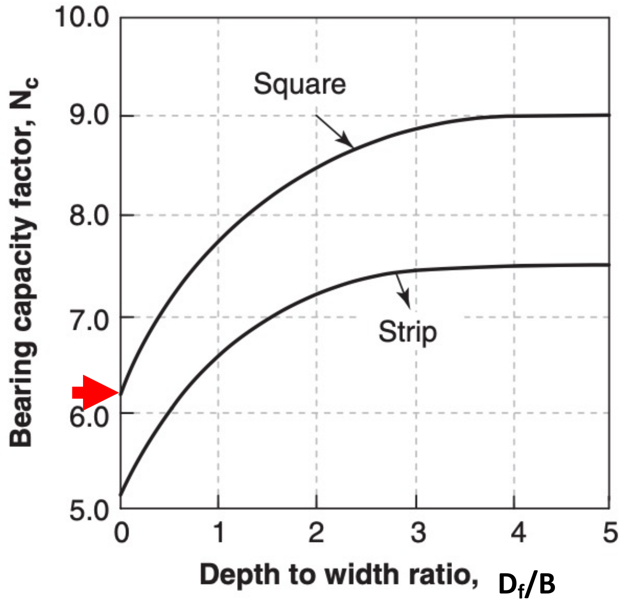
A column load of 2000 kN is to be supported by a square spread footing on a very stiff clay. Recommend the size of the footing after addressing the issue of bearing capacity. s_u = 100 kPa and γ = 18 kN/m^3.

Solution:

Here, it can be assumed that the depth of foundation equals to 0.5 m, so, $D_f/B \cong 0$

So, N_c = 6.3 from the figure of Skempton finding.

$$q_u = N_c\, S_u + \gamma D_f = 6.3 \times 100 + 18 \times 0.5 = 639\ \mathrm{kN/m^2}$$



$q_a = q_u/\ FS = 639/3 = 213$ kPa

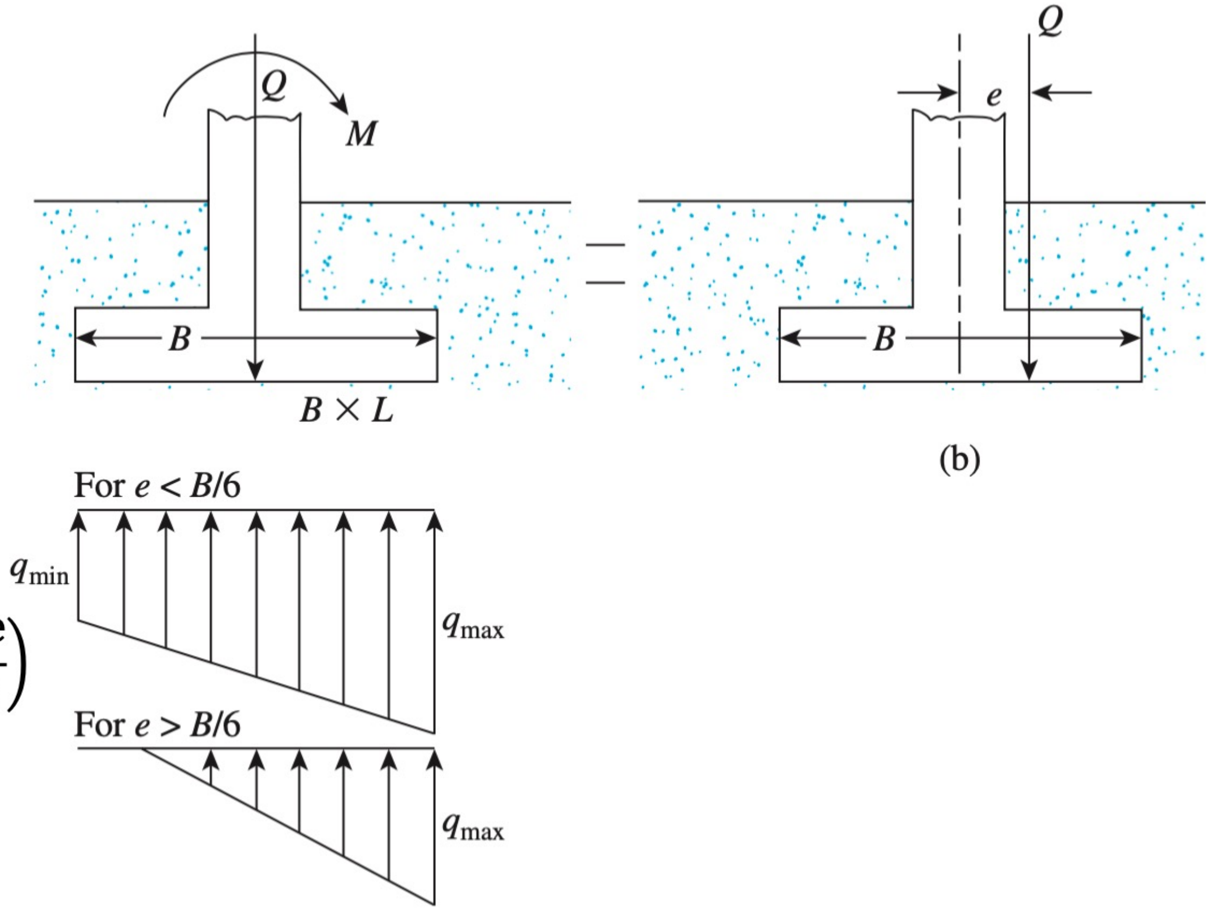
$A = Q_a/q_a = 2000\text{kN}/213\ \text{kN/m}^3 = 9.4\ \text{m}^2 = B \times B$

$B = \sqrt[2]{A} = \sqrt[2]{9.4} = 3.07\ \text{m} \approx 3.1\ m$

Eccentrically Loaded Foundations

In several instances, as it was shown with the base of a retaining wall, foundations are subjected to moments in addition to the vertical load, as shown in the Figure. In such cases, the distribution of pressure by the foundation on the soil is not uniform. The stress will be due to compression exerted by the concentrated load (Q) and the stress due to moment so that the stress equation will be in the form: $q = \frac{Q}{A} \pm \frac{MC}{I}$.

Here, M = Q*e; c = B/2 or c = L/2, depending on the direction of moment and on the axis that affects on.; I is the moment of inertia and also it may be about B axis or about L axis. For rectangular, $I_{x(b)} = B^3L/12$ or $I_{y\ (L)} = L^3B/12$



$$q = \frac{Q}{B \times L} \pm \frac{Q \times e \times B}{\frac{2\,B^3 \times L}{12}} \rightarrow q = \frac{Q}{B \times L} \pm \frac{6eQ}{B^2L} \rightarrow q = \frac{Q}{B \times L}\left(1 \pm \frac{6e}{B}\right)$$

$$\boxed{q = \frac{Q}{B \times L}\left(1 \pm \frac{6e}{B}\right) \text{ General Equation}}$$

The nominal distribution of pressure is:

$$q_{\max} = \frac{Q}{BL} + \frac{6M}{B^2L}$$

and

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

where

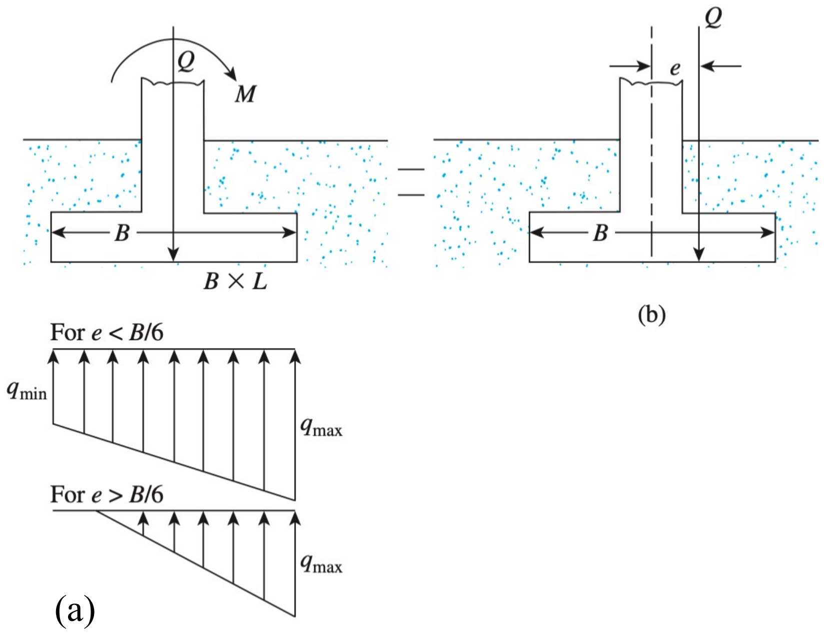
Q = total vertical load

M = moment on the foundation

In the Figure below shows a force system applied on the foundation. The distance is the eccentricity and it equals:

$$e = \frac{M}{Q}$$

So that Fig. (b) is the equivalent loading system of that loading due to Moment and Q.



Substituting Equation of (e) into Eqs. of q_{max} and q_{min} gives

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

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

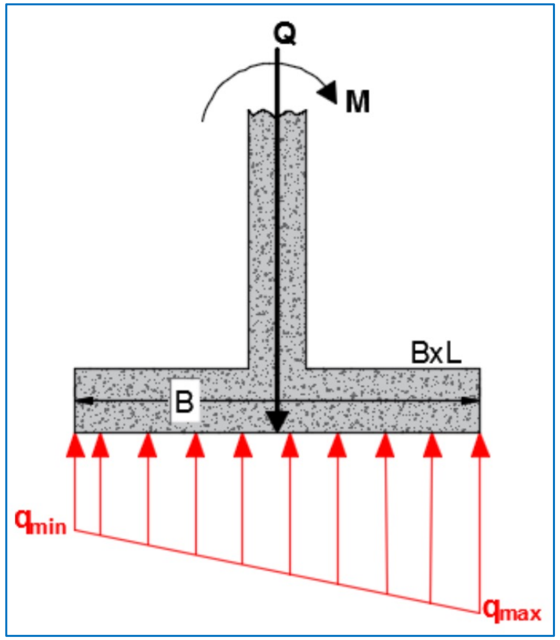
Note (See the next slide)

1) These two equations are valid when the eccentricity $e \leq B/6$.
2) q_{min} is zero for $e = B/6$.
3) q_{min} will be negative when $e > B/6$, which means that tension will develop. Because soil cannot take any tension, there will then be a separation between the foundation and the soil underlying it. .

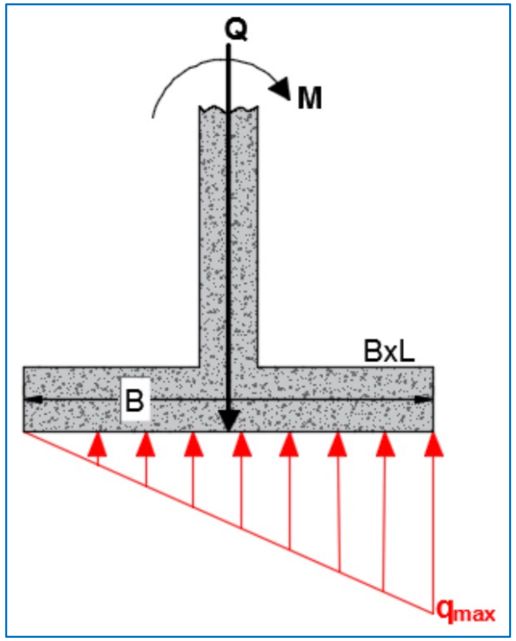
The factor of safety for such type of loading against bearing capacity failure can be evaluated as:

$$\boxed{\mathrm{FS} = \frac{Q_u}{Q}}$$

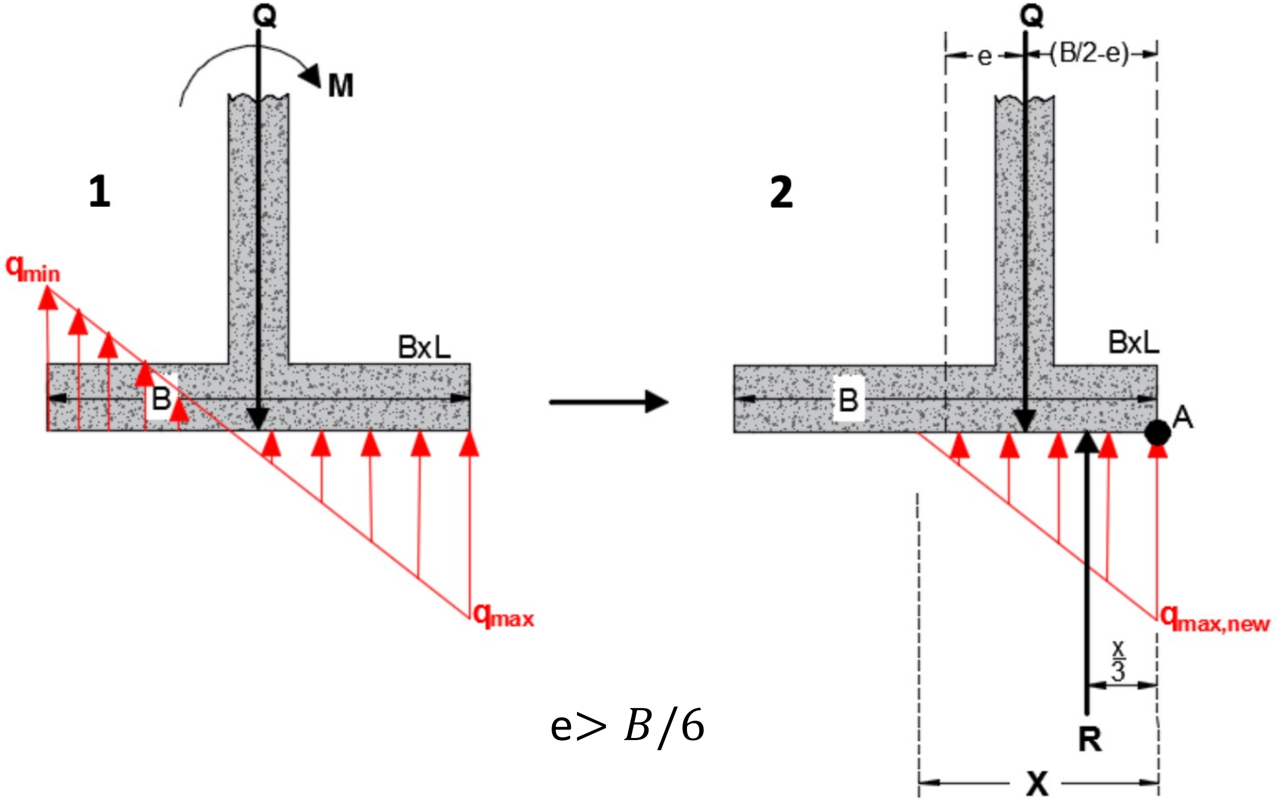
where Q_u = ultimate load-carrying capacity.



e< $B/6$



e = $B/6$



e> $B/6$

When e $\leq \frac{B}{6}$, this is accepted

When e $> \frac{B}{6}$, this is unaccepted

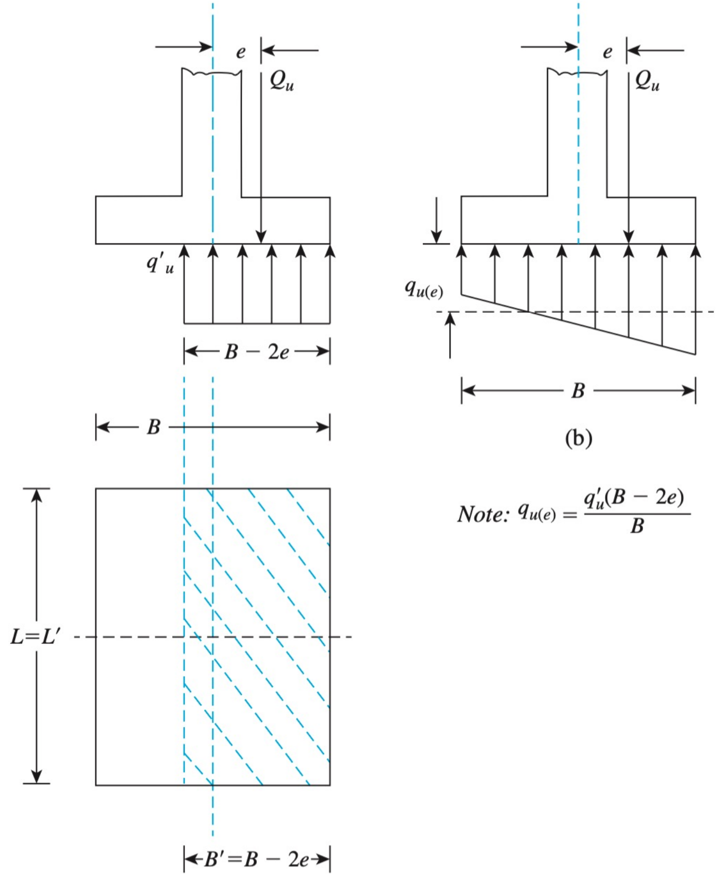
Ultimate Bearing Capacity under Eccentric Loading:

One-Way Eccentricity : Effective Area Method

step-by-step procedure for determining the ultimate load that the soil can support and the factor of safety against bearing capacity failure:

1) Determine the effective dimensions of the foundation as shown in the Figure : B' = effective width = B' = $B- 2e$

L' = effective length = L .



Note that if the eccentricity were in the direction of the length of the foundation, the value of L' would be equal to (L - 2e). The value of B' would equal B. The smaller of The two dimensions (i.e., L' and B') is the effective width of the foundation.

2) Use the same equation for determination of ultimate bearing capacity by Meyerhof, But, only substitute B by B' that was found by step (1) in above.

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

3) The total ultimate load that the foundation can sustain is

$$Q_u = q'_u \overbrace{(B')(L')}^{A'}$$

where A' = effective area.

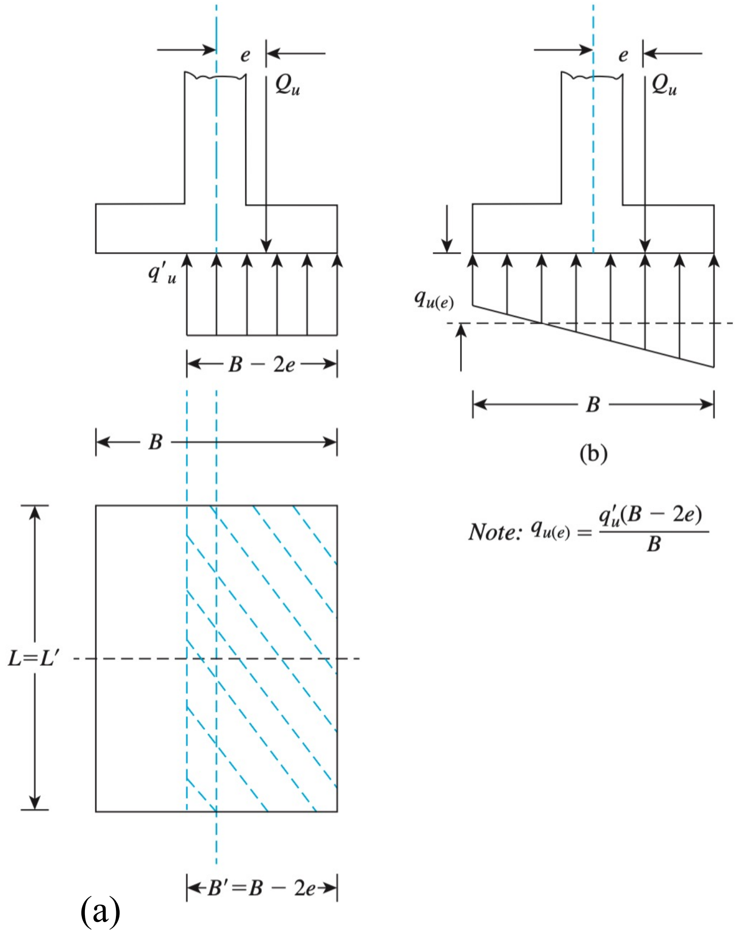
4) The factor of safety against bearing capacity failure is: $\mathrm{FS} = \dfrac{Q_u}{Q}$

Important Note: It is important to note that q'_u is the ultimate bearing capacity of a foundation of width $B' = (B\text{-}2e)$ with a centric load as in the Figure (a). However, the actual distribution of soil reaction at ultimate load will be of the type shown in Figure (b).

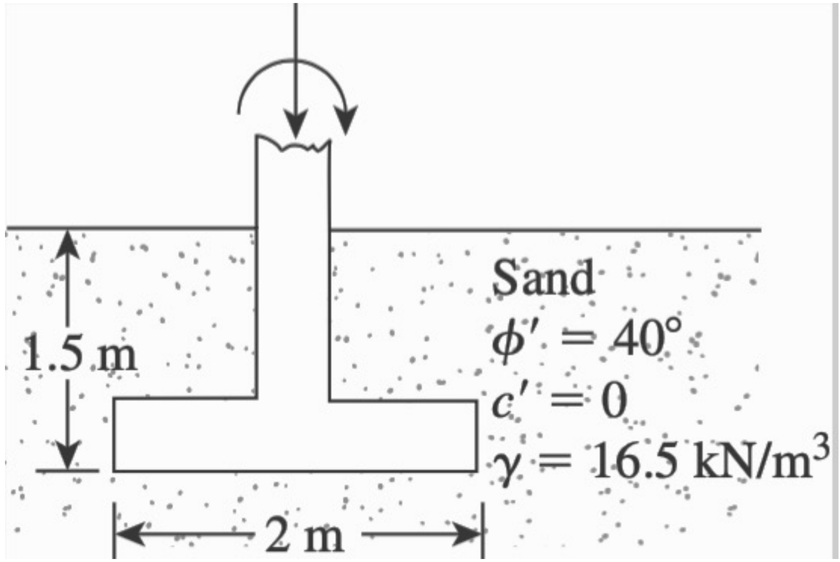
In Figure (b), $q_{u(e)}$ is the average load per unit area of the foundation

Thus:

$$q_{u(e)} = \frac{q'_u(B - 2e)}{B}$$



Example: A continuous (strip) foundation is shown in the Figure. If the load eccentricity is 0.2 m, determine the ultimate load, Q_u, per unit length of the foundation. Use Meyerhof's effective area method.



Solution

For $c' = 0$, Eq. gives:

$$q'_u = qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma' B' N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

where $q = (16.5)\,(1.5) = 24.75\ \text{kN/m}^2$

For $\phi' = 40°$, $N_q = 64.2$ and $N_\gamma = 109.41$. Also,

$$B' = 2 - (2)(0.2) = 1.6\ \text{m}$$

Because the foundation in question is a continuous foundation, B'/L' is zero. Hence, $F_{qs} = 1$, $F_{\gamma s} = 1$.

$$F_{qi} = F_{\gamma i} = 1$$

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\frac{D_f}{B} = 1 + 0.214\left(\frac{1.5}{2}\right) = 1.16$$

$$F_{\gamma d} = 1$$

and

$$q'_u = (24.75)(64.2)(1)(1.16)(1)$$

$$+ \left(\frac{1}{2}\right)(16.5)(1.6)(109.41)(1)(1)(1) = 3287.39\ \text{kN/m}^2$$

Consequently,

$$Q_u = (B')(1)(q'_u) = (1.6)(1)(3287.39) \approx \mathbf{5260\ kN}$$

Thank you

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Bearing Capacity (bearing load) of Soil (6)

- Eccentricity:
  - Two-way (eccentric in two directions or two axes)
- Answers of questions of Skempton's equation

Ultimate Bearing Capacity under Eccentric Loading:
<u>Two-Way Eccentricity :</u>

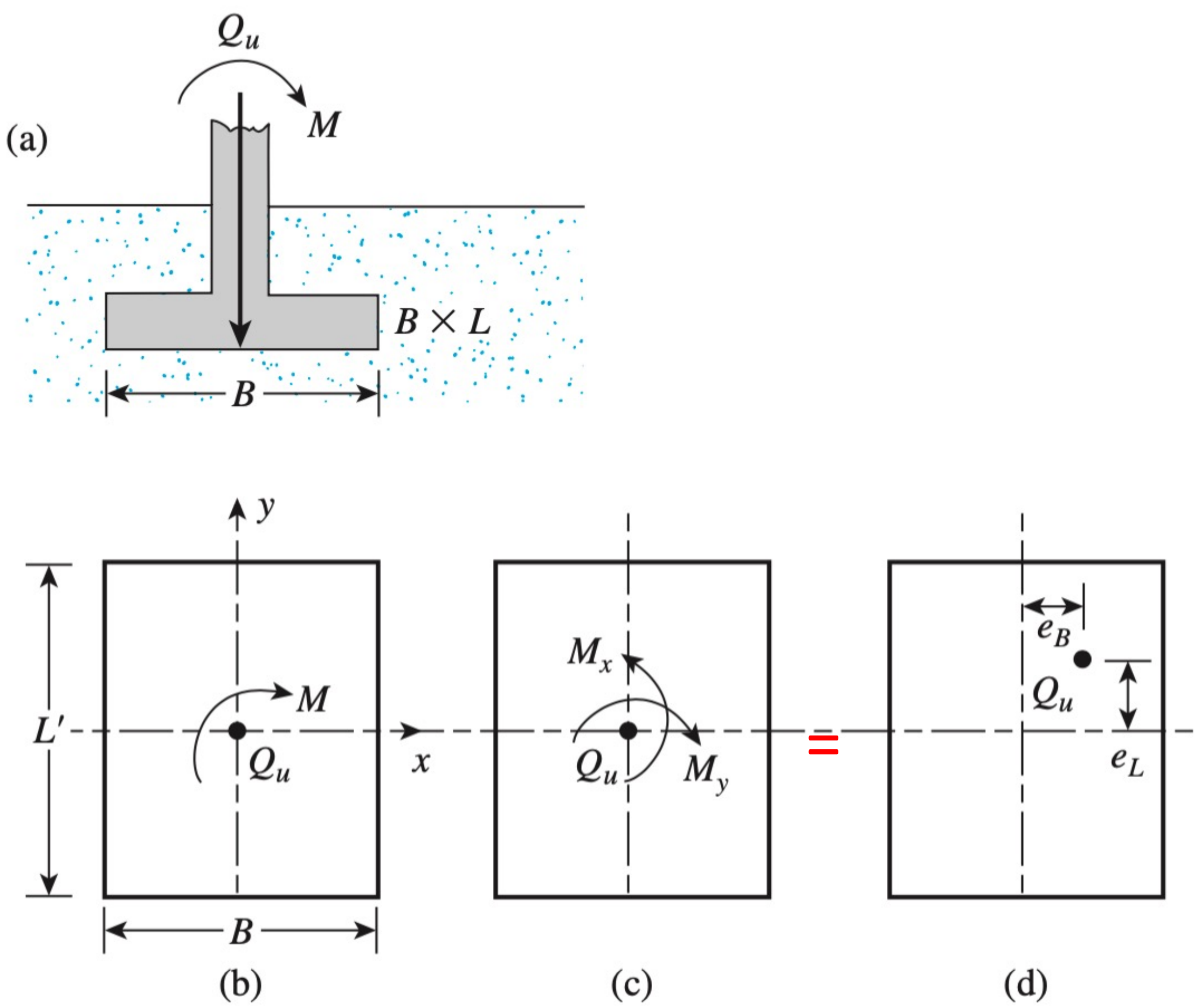


Figure 4.24 Analysis of foundation with two-way eccentricity

M about the x- and y-axes can be determined as M_x and M_y, respectively. (See Figure 4.24c.) This condition is equivalent to a load Q_u placed eccentrically on the foundation with $x = e_B$ and $y = e_L$ (Figure 4.24d). Note that

$$e_B = \frac{M_y}{Q_u} \tag{4.65}$$

and

$$e_L = \frac{M_x}{Q_u} \tag{4.66}$$

If Q_u is needed, it can be obtained from Eq. (4.52); that is,

$$Q_u = q'_u A'$$

where, from Eq. (4.51),

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

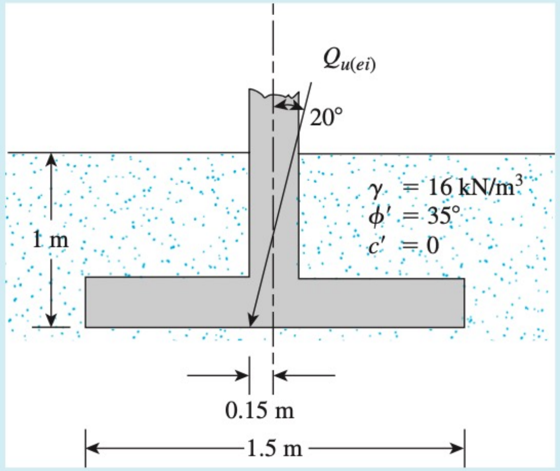
and

$$A' = \text{effective area} = B'L'$$

As before, to evaluate F_{cs}, F_{qs}, and $F_{\gamma s}$ (Table 4.3), we use the effective length L' and effective width B' instead of L and B, respectively. To calculate F_{cd}, F_{qd}, and $F_{\gamma d}$, we do not replace B with B'. In determining the effective area A', effective width B', and effective length L', five possible cases may arise (Highter and Anders, 1985).

Example (6-1)

A continuous foundation is shown in Figure Estimate the inclined ultimate load, $Q_{u(ei)}$ per unit length of the foundation.



Solution

From Eq. (4.81) with $c' = 0$, we have

$$q'_u = qN_qF_{qd}F_{qi} + \frac{1}{2}\gamma B'N_\gamma F_{\gamma d}F_{\gamma i}$$

$$q = \gamma D_f = (16)(1) = 16 \text{ kN/m}^2$$

and

$$B' = B - 2e = 1.5 - (2)(0.15) = 1.2 \text{ m}$$

From Table 4.2 for $\phi' = 35°$, $N_q = 33.3$, and $N_\gamma = 48.03$, we have

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right) = 1 + 2\tan 35(1 - \sin 35)^2\left(\frac{1}{1.5}\right) = 1.17$$

$$F_{\gamma d} = 1$$

$$F_{qi} = \left(1 - \frac{\beta°}{90°}\right)^2 = \left(1 - \frac{20}{90}\right)^2 = 0.605$$

$$F_{\gamma i} = \left(1 - \frac{\beta°}{\phi'}\right)^2 = \left(1 - \frac{20}{35}\right)^2 = 0.184$$

$$q'_u = (16)(33.3)(1.17)(0.605) + \left(\frac{1}{2}\right)(16)(1.2)(48.03)(1)(0.184) = 461.98 \text{ kN/m}^2$$

and

$$Q_{u(ei)} = \frac{q'_u(B - 2e)}{\cos\beta} = \frac{(461.98)(1.2)}{\cos 20} = 589.95 \text{ kN} \approx \mathbf{590\ kN/m}$$

Skempton's equation

1) Why does N_c of square footing greater than that of strip footing?

The reason is that the square footing can develop a relatively larger failure surface, because the failure surface can develop in four directions, whereas the failure surface for the strip footing is confined to only two directions.

1) From the figure, it is noticed that N_c increases gradually with embedment ratio, why?

Nc gradually increases with the relative depth of embedment, due to the gradual increase in the length of the failure surface with embedment.

1) S_u in Skempton equation for undrained fine soil, is it equals to C_u?

Yes, in Mohr envelope equation ($S_u = c' + \sigma' \tan \phi$), $\tan \phi = 0$, *where* $\phi = 0$ *for fine-grained undrained conditions, so* $S_u = C_u$.

1) C_u determined by uniaxial compression (unconfined compression) test = $q_{unconfined}/2$, so is it correct that the net allowable bearing capacity when FS = 3 equals, approximately, the unconfined compression pressure for square footing with embedded ratio =0 ?

$C_u = q_{uncon}/2$, $q_{a\,(net)} = q_u/3 = (CN_c + q - q)/3 = (q_{uncon}/2 * 6.3)/3 = \frac{q_{uncon} * 6.3}{2*3} = \frac{q_{uncon} * 6.3}{6} \approx q_{uncon}$

Thank You

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Bearing Capacity (bearing load) of Soil (2)

- Terzaghi equation (approach), cont'd
  - Bearing capacity for Square and Circular Foundations
  - Allowable bearing capacity (gross and net)
  - Examples

Terzaghi's Approach, cont'd

Terzaghi solution has been extended to obtain the ultimate bearing capacity (q_u) of square foundation and circular foundation by adding factors known as shape factors. This extension presented in the table:

Type of Foundation	Equation of Ultimate bearing capacity (q_u)	Notes
Strip	$q_u = c'N_c + qN_q + 0.5\,\gamma B N_\gamma$	**B/L → 0**
Square	$q_u = 1.3\,c'N_c + qN_q + 0.4\,\gamma B N_\gamma$	**B = Side length of square**
Circular	$q_u = 1.3\,c'N_c + qN_q + 0.3\,\gamma B N_\gamma$	**B = Diameter of circle**

Allowable bearing capacity (q_a)

The gross allowable load-bearing capacity of shallow foundations is the gross ultimate bearing capacity divided by factor of safety (FS), i.e.:

$$q_{a\,(g)} = \frac{q_{u\,(g)}}{FS}$$

Usually the net allowable bearing capacity is almost used for checking of stability of foundation against shear failure, so, the net ultimate bearing capacity defined as the gross ultimate stress in excess of the surcharge pressure (γD_f) or (q).

$$q_{a\,(net)} = \frac{q_{u\,(net)}}{FS} = \frac{q_u - q}{FS}$$

For strip footing, $q_{a\,(net)} = \dfrac{cN_c + qN_q + 0.5\gamma N_\gamma - q}{FS}$

$$= \frac{cN_c + q(N_q - 1) + 0.5\gamma N_\gamma}{FS}$$

Mostly, the factor of safety (FS) equals (3).

Examples: 1):

A square foundation is 2 m × 2 m in plan. The soil supporting the foundation has a friction angle of $\phi' = 25°$ and $c' = 20$ kN/m^2. The unit weight of soil, γ, is 16.5 kN/m^3. Determine the allowable gross load on the foundation with a factor of safety (FS) of 3. Assume that the depth of the foundation (D_f) is 1.5 m and that general shear failure occurs in the soil.

Table 4.1 Terzaghi's Bearing Capacity Factors—Eqs. (4.15), (4.13), and (4.11).[a]

ϕ'	N_c	N_q	N_γ[a]	ϕ'	N_c	N_q	N_γ[a]
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

[a]From Kumbhojkar (1993)

Solution

From Eq.

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$$

From Table 4.1, for $\phi' = 25°$,

$$N_c = 25.13$$

$$N_q = 12.72$$

$$N_\gamma = 8.34$$

Solution of Ex.1, cont'd

Thus,

$$q_u = (1.3)(20)(25.13) + (1.5 \times 16.5)(12.72) + (0.4)(16.5)(2)(8.34)$$

$$= 653.38 + 314.82 + 110.09 = 1078.29 \text{ kN/m}^2$$

So, the allowable load per unit area of the foundation is

$$q_{\text{all}} = \frac{q_u}{\text{FS}} = \frac{1078.29}{3} \approx 359.5 \text{ kN/m}^2$$

Thus, the total allowable gross load is

$$Q = (359.5)\, B^2 = (359.5)\,(2 \times 2) = \mathbf{1438\ kN}$$

Example 2

Refer to Example 1 Assume that the shear-strength parameters of the soil are the same. A square foundation measuring $B \times B$ will be subjected to an allowable gross load of 1000 kN with FS = 3 and $D_f = 1$ m. Determine the size B of the foundation.

Solution

Allowable gross load $Q = 1000$ kN with FS = 3. Hence, the ultimate gross load $Q_u = (Q)(\text{FS}) = (1000)(3) = 3000$ kN. So,

$$q_u = \frac{Q_u}{B^2} = \frac{3000}{B^2} \tag{a}$$

From Eq. for square foundation:

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma BN_\gamma$$

For $\phi' = 25°$, $N_c = 25.13$, $N_q = 12.72$, and $N_\gamma = 8.34$.
Also,

$$q = \gamma D_f = (16.5)(1) = 16.5 \text{ kN/m}^2$$

Now,

$$\begin{aligned} q_u &= (1.3)(20)(25.13) + (16.5)(12.72) + (0.4)(16.5)(B)(8.34) \\ &= 863.26 + 55.04B \end{aligned} \tag{b}$$

Example (2), cont'd

Combining Eqs. (a) and (b),

$$\frac{3000}{B^2} = 863.26 + 55.04B \tag{c}$$

By trial and error, we have

$$B = 1.77 \text{ m} \approx \mathbf{1.8\ m}$$


Thank
You

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Bearing Capacity (bearing load) of Soil (3)

- ❑ Consideration of ground water table in Bearing capacity calculations
- ❑ General equation of bearing capacity (Meyerhof approach)
- ❑ Examples

Water table effect

Determination of ultimate bearing capacity of the soil for shallow foundation were done without considering the existence of ground water, i.e. the ground water table considered at deep level out of the effective zone under the base of foundation. Here, the ground water should be considered in this calculations:

Three cases:

1) **Case I:** When the level of ground water is above the level of base of the foundation and it is at depth (D_1) from the N.G.L. i.e. $0 \le D_1 \le D_f$, as in the Figure below:

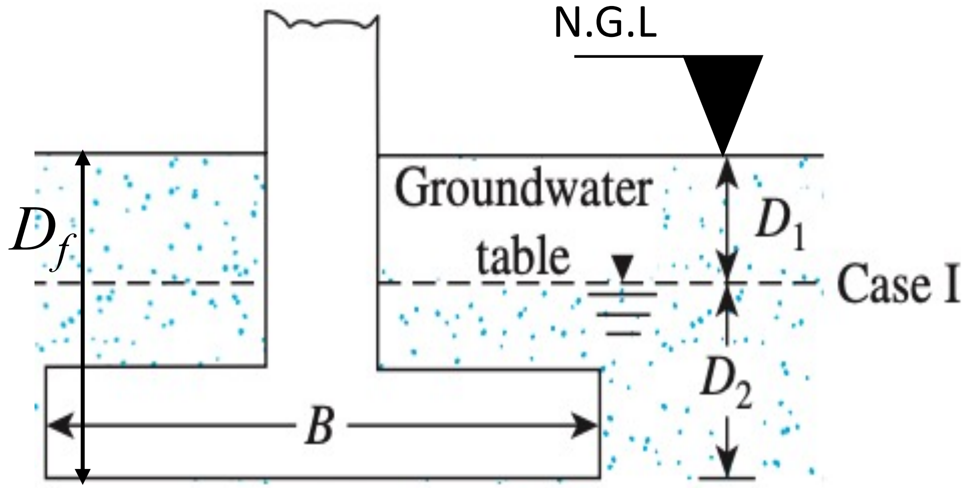
$$q = \text{effective surcharge} = D_1\gamma + D_2(\gamma_{sat} - \gamma_w)$$

where

γ_{sat} = saturated unit weight of soil
γ_w = unit weight of water

Also, the value of γ in the last term of the equations has to be replaced by $\gamma' = \gamma_{sat} - \gamma_w$.

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma$$



Water table effect, cont'd

2) Case II: When the level of ground water is below the level of base of the foundation and it is at depth (d) from the base of the foundation i.e. $D_f < d \le B$, as in the Figure below:

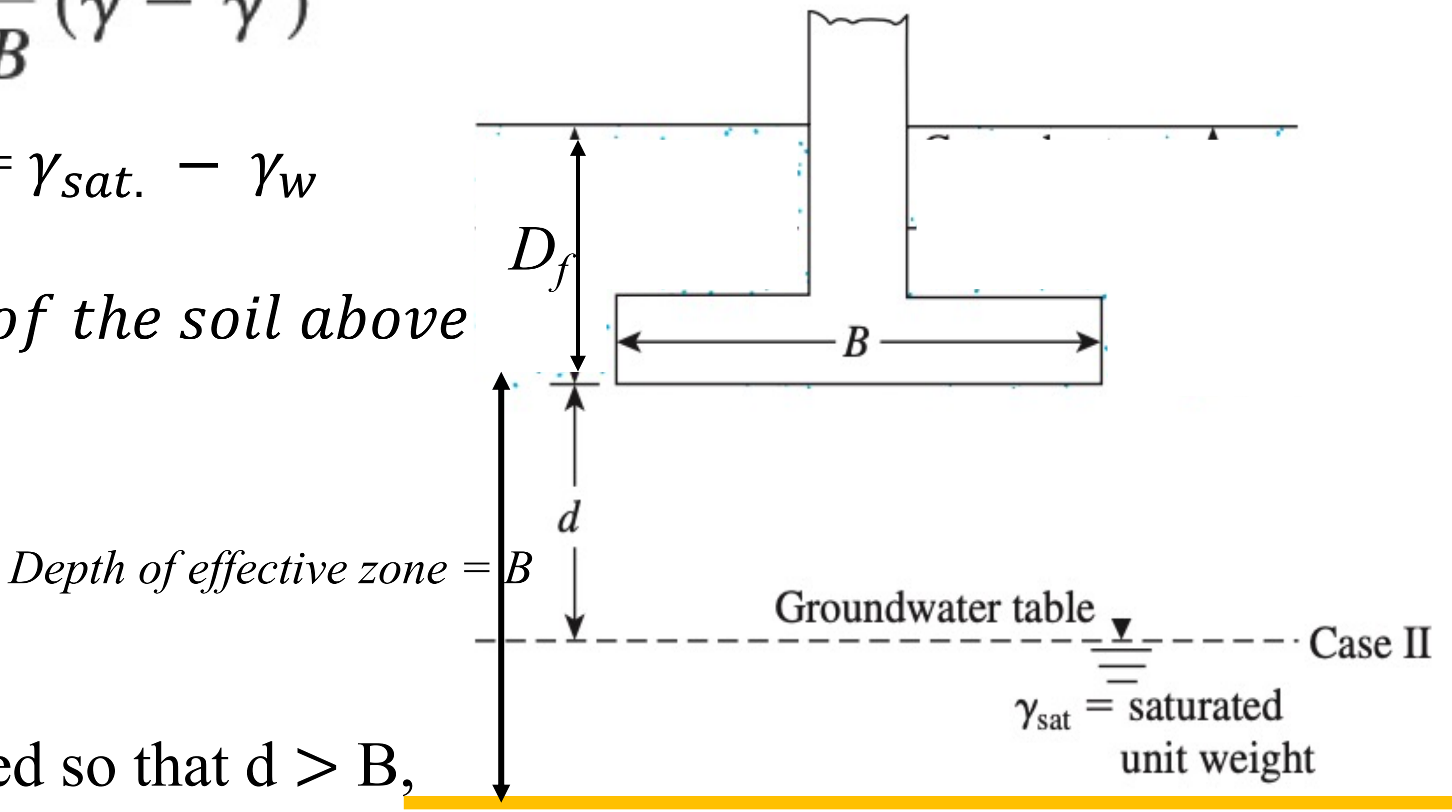
In this case, the factor γ in the last term of the bearing capacity equations must be replaced by the factor $\bar{\gamma}$ as in the equation :

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$$

Where: γ' is the submerged unit weight = $\gamma_{sat.} - \gamma_w$

and q = γD_f, here γ *is the unit weight of the soil above* The base of the foundation

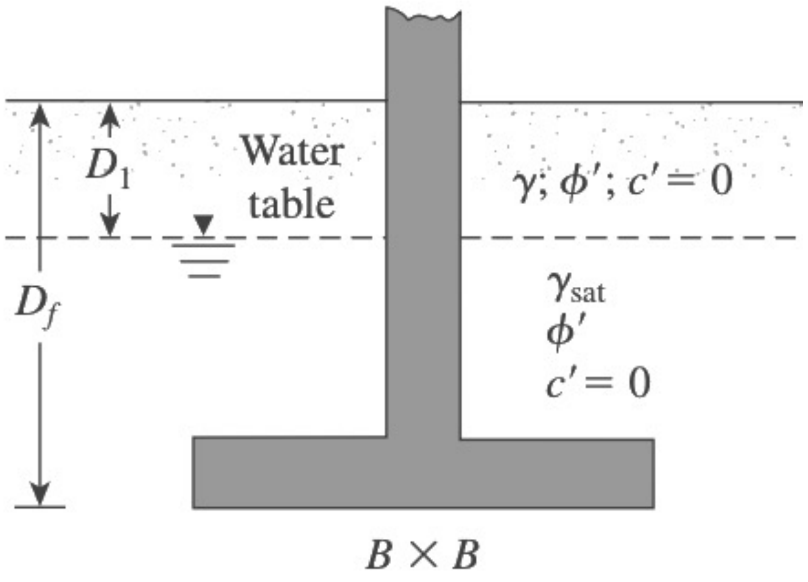
$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma$$



3) Case III: When the water table is located so that d > B, the water will have no effect on the ultimate bearing capacity.

Example -1-

A square foundation B x B has to be constructed as shown in the Figure. Assume that $\gamma = 105$ lb/ft^3, γ_{sat} =118 lb/ft^3, $\emptyset' = 34^{\circ}$, D_f =4 ft, and D_1= 2 ft. B = 4.5 ft, Determine the gross and net allowable bearing pressure, take FS=3.



Solution:

Depth of water table (D_1) = 2 ft, Depth of foundation (D_f) = 4.5 ft.

For $\emptyset' = 34^{\circ}$, and from the table: 34 52.64 36.50 38.04

- $N_q = 36.5$, $N\gamma = 38.04$

Because of c' = 0 , so the cohesion term = 0

$$q_u = 1.3\ c'N_c + qN_q + 0.4\ \gamma B N_\gamma$$

$$q_{u(g)} = [105 \times 2 + (118 - 62.4) \times 2)] \times 36.5 + 0.4\ (118 - 62.4) 4.5 \times 38.04 = 15530.8\ \frac{\text{Ib}}{\text{ft}^2}$$

$q_{all(g)} = q_u$ / FS = 15530.8/3 = 5177 Ib/ft^2

$q_{u\ (net)} = q_{u(g)} - q$ = 15530.8 - [105 x 2 + (118 – 62.4)x 2)] = 15209.6 Ib/ft^2

$q_{all\ (net)} = q_{u(net)}$ / FS = 15209.6/3 = 5070 Ib/ft^2

Example -2-

Solve the example -1-, when the level of ground water table (D_1) is at 6 ft below the ground surface, and all the remaining data are the same:

Solution:

The effective zone depth under the base of foundation = B = 4.5 ft

The depth of this zone from N.G.L = D_f + B = 4+ 4.5 = 8.5 ft.

So, the ground water table level = 6 ft is more than D_f and less than the level of effective zone, so here the case II should be applied:

d = $D_1 - D_f$ = 6– 4 = 2 m

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma') = (118\text{- }62.4) + 2/4.5\ \text{X}(105 - (118 - 62.4)) = 77.6\ \text{Ib/ft}^3$$

$\boldsymbol{q_u} = \mathbf{q}\boldsymbol{N_q} + \mathbf{0.4}\ \boldsymbol{\gamma B N_\gamma}$ = 4 X 105 X 36.5 + 0.4 x 77.6 x 4.5 x 38.04 = 20643.4 Ib/ft^2

$q_{all\ (g)}$ = 20643.4/3 = 6881.1 Ib/ft^2

$q_{u\ (net)} = q_{u\ (g)} - q$ = 20643.4 – 4 x 105 = 20223.4 Ib/ft^2

$q_{all\ (net)}$ = 20223.4/3 = 6741.1 Ib/ft^2

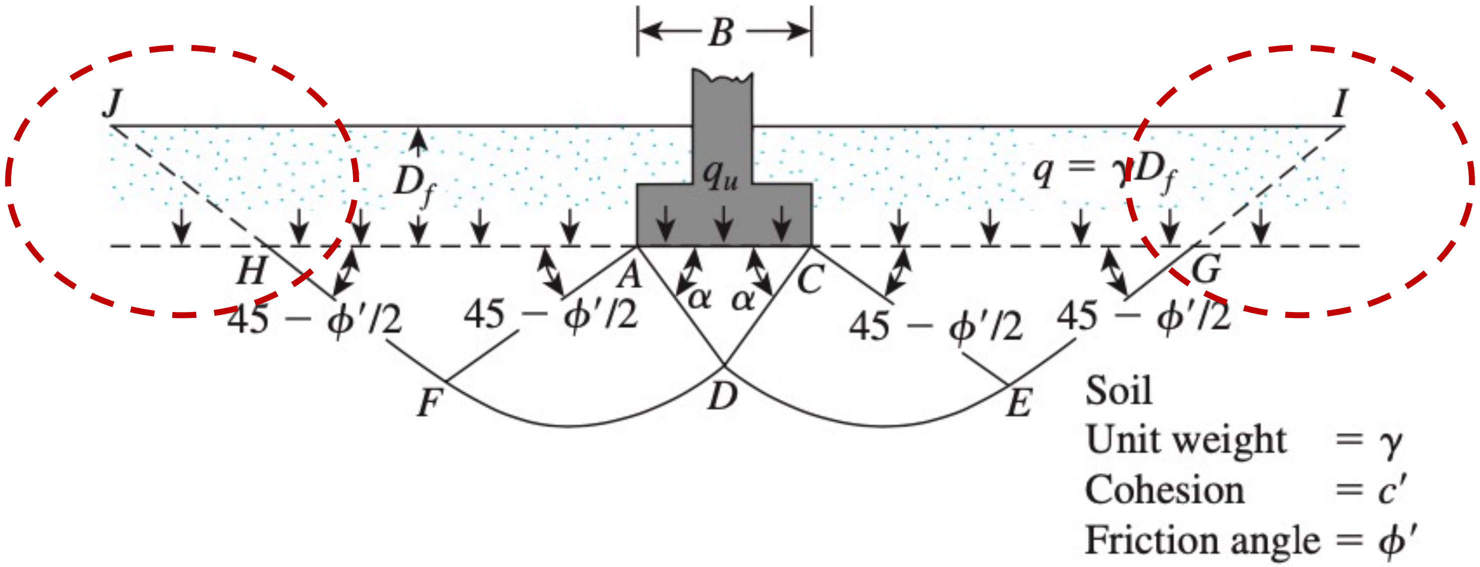
For comparison, $\frac{6741.1-5070}{5070}$ x 100 = 33 % is the increase of allowable bearing capacity when the level of W.T decreases.

General Bearing Capacity Equation (Meyerhof Approach)

To overcome the shortcomings that was appeared in Terzaghi equation, Meyerhof (1963) suggested the following form of the general bearing capacity equation.

This ultimate bearing capacity equation can be applied to:

- the case of rectangular foundations ($0 < \frac{B}{L} < 1$),
- It takes into account the shearing resistance along the failure surface in soil above the bottom of the foundation (the portion of the failure surface marked as *GI* and *HJ* in the Figure).
- Meyerhof equation includes the inclined loading may experience by foundation.



General Bearing Capacity Equation (Meyerhof Approach),cont'd

$$q_u = \boxed{c'N_c}F_{cs}F_{cd}F_{ci} + \boxed{qN_q}F_{qs}F_{qd}F_{qi} + \boxed{\tfrac{1}{2}\gamma BN_\gamma}F_{\gamma s}F_{\gamma d}F_{\gamma i} \quad \textit{(Meyerhof Equation)}$$

$$q_u = \boxed{c'N_c} + \boxed{qN_q} + \boxed{\frac{1}{2}\gamma BN_\gamma}. \quad \textit{(Terzaghi Equation)}$$

In this equation:

c' = cohesion

q = effective stress at the level of the bottom of the foundation

γ = unit weight of soil

B = width of foundation (= diameter for a circular foundation)

$F_{cs}, F_{qs}, F_{\gamma s}$ = shape factors

$F_{cd}, F_{qd}, F_{\gamma d}$ = depth factors

$F_{ci}, F_{qi}, F_{\gamma i}$ = load inclination factors

N_c, N_q, N_γ = bearing capacity factors

Table 4.2 Bearing Capacity Factors for Meyerhof Equation

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	16	11.63	4.34	3.06
1	5.38	1.09	0.07	17	12.34	4.77	3.53
2	5.63	1.20	0.15	18	13.10	5.26	4.07
3	5.90	1.31	0.24	19	13.93	5.80	4.68
4	6.19	1.43	0.34	20	14.83	6.40	5.39
5	6.49	1.57	0.45	21	15.82	7.07	6.20
6	6.81	1.72	0.57	22	16.88	7.82	7.13
7	7.16	1.88	0.71	23	18.05	8.66	8.20
8	7.53	2.06	0.86	24	19.32	9.60	9.44
9	7.92	2.25	1.03	25	20.72	10.66	10.88
10	8.35	2.47	1.22	26	22.25	11.85	12.54
11	8.80	2.71	1.44	27	23.94	13.20	14.47
12	9.28	2.97	1.69	28	25.80	14.72	16.72
13	9.81	3.26	1.97	29	27.86	16.44	19.34
14	10.37	3.59	2.29	30	30.14	18.40	22.40
15	10.98	3.94	2.65	31	32.67	20.63	25.99
32	35.49	23.18	30.22	42	93.71	85.38	155.55
33	38.64	26.09	35.19	43	105.11	99.02	186.54
34	42.16	29.44	41.06	44	118.37	115.31	224.64
35	46.12	33.30	48.03	45	133.88	134.88	271.76
36	50.59	37.75	56.31	46	152.10	158.51	330.35
37	55.63	42.92	66.19	47	173.64	187.21	403.67
38	61.35	48.93	78.03	48	199.26	222.31	496.01
39	67.87	55.96	92.25	49	229.93	265.51	613.16
40	75.31	64.20	109.41	50	266.89	319.07	762.89
41	83.86	73.90	130.22				

Table 4.3 Shape, Depth and Inclination Factors [DeBeer (1970); Hansen (1970); Meyerhof (1963); Meyerhof and Hanna (1981)]

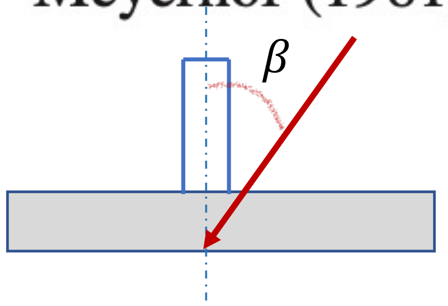
Factor	Relationship	Reference
Shape	$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$ $F_{qs} = 1 + \left(\frac{B}{L}\right)\tan \phi'$ $F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right)$	DeBeer (1970)

Table 4.3 Shape, Depth and Inclination Factors [DeBeer (1970); Hansen (1970); Meyerhof (1963); Meyerhof and Hanna (1981)]

Depth	$\dfrac{D_f}{B} \le 1$ For $\phi = 0$: $F_{cd} = 1 + 0.4\left(\dfrac{D_f}{B}\right)$ $F_{qd} = 1$ $F_{\gamma d} = 1$ For $\phi' > 0$: $F_{cd} = F_{qd} - \dfrac{1 - F_{qd}}{N_c \tan \phi'}$ $F_{qd} = 1 + 2\tan\phi'\,(1 - \sin\phi')^2\left(\dfrac{D_f}{B}\right)$ $F_{\gamma d} = 1$	$\dfrac{D_f}{B} > 1$ For $\phi = 0$: $F_{cd} = 1 + 0.4\underbrace{\tan^{-1}\left(\dfrac{D_f}{B}\right)}_{\text{radians}}$ $F_{qd} = 1$ $F_{\gamma d} = 1$ For $\phi' > 0$: $F_{cd} = F_{qd} - \dfrac{1 - F_{qd}}{N_c \tan \phi'}$ $F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\underbrace{\tan^{-1}\left(\dfrac{D_f}{B}\right)}_{\text{radians}}$ $F_{\gamma d} = 1$	Hansen (1970)

Table 4.3 Shape, Depth and Inclination Factors [DeBeer (1970); Hansen (1970); Meyerhof (196
Meyerhof and Hanna (1981)]

Inclination	$F_{ci} = F_{qi} = \left(1 - \dfrac{\beta°}{90°}\right)^2$ $F_{\gamma i} = \left(1 - \dfrac{\beta°}{\phi'}\right)^2$ β = inclination of the load on the foundation with respect to the vertical	Meyerhof (1963); Hanna Meyerhof (1981)



Example (3):

Resolve Example (1) in previous section (2), using the general equation of ultimate bearing capacity.

Solution:

From the Meyerhof equation the solution is:

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qt} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma t}$$

Since the load is vertical, $F_{ci} = F_{qi} = F_{\gamma i} = 1$. From Table 4.2 for $\phi' = 25°$, $N_c = 20.72$, $N_q = 10.66$, and $N_\gamma = 10.88$.

Using Table 4.3,

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = 1 + \left(\frac{2}{2}\right)\left(\frac{10.66}{20.72}\right) = 1.514$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi' = 1 + \left(\frac{2}{2}\right)\tan 25 = 1.466$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{2}{2}\right) = 0.6$$

Table 4.2 Bearing Capacity Factors

ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
0	5.14	1.00	0.00	16	11.63	4.34	3.06
1	5.38	1.09	0.07	17	12.34	4.77	3.53
2	5.63	1.20	0.15	18	13.10	5.26	4.07
3	5.90	1.31	0.24	19	13.93	5.80	4.68
4	6.19	1.43	0.34	20	14.83	6.40	5.39
5	6.49	1.57	0.45	21	15.82	7.07	6.20
6	6.81	1.72	0.57	22	16.88	7.82	7.13
7	7.16	1.88	0.71	23	18.05	8.66	8.20
8	7.53	2.06	0.86	24	19.32	9.60	9.44
9	7.92	2.25	1.03	25	20.72	10.66	10.88
10	8.35	2.47	1.22	26	22.25	11.85	12.54
11	8.80	2.71	1.44	27	23.94	13.20	14.47
12	9.28	2.97	1.69	28	25.80	14.72	16.72
13	9.81	3.26	1.97	29	27.86	16.44	19.34
14	10.37	3.59	2.29	30	30.14	18.40	22.40
15	10.98	3.94	2.65	31	32.67	20.63	25.99

Example (3): cont'd

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

$$= 1 + (2)(\tan 25)(1 - \sin 25)^2\left(\frac{1.5}{2}\right) = 1.233$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'} = 1.233 - \left[\frac{1 - 1.233}{(20.72)(\tan 25)}\right] = 1.257$$

$$F_{\gamma d} = 1$$

Hence,

$$q_u = (20)(20.72)(1.514)(1.257)(1)$$

$$+ (1.5 \times 16.5)(10.66)(1.466)(1.233)(1)$$

$$+ \frac{1}{2}(16.5)(2)(10.88)(0.6)(1)(1)$$

$$= 788.6 + 476.9 + 107.7 = 1373.2 \text{ kN/m}^2$$

$$q_{all} = \frac{q_u}{FS} = \frac{1373.2}{3} = 457.7 \text{ kN/m}^2$$

$$Q = (457.7)(2 \times 2) = \mathbf{1830.8\ kN}$$

Thank you

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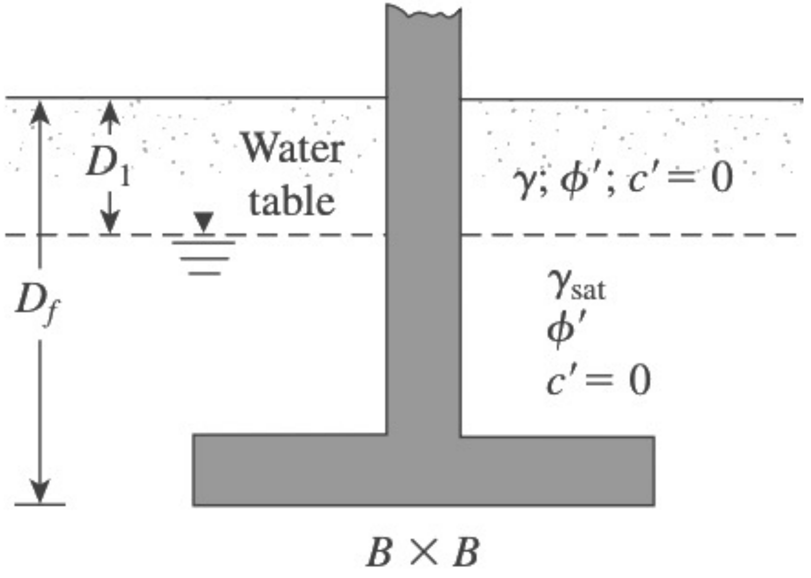
Bearing Capacity (bearing load) of Soil (4)

- Examples

Examples

1) A square foundation B x B has to be constructed as shown in the Figure. Assume that $\gamma = 105$ lb/ft^3, γ_{sat} =118 lb/ft^3, $\emptyset' = 34^\circ$, D_f =4 ft, and D_1= 2 ft. B = 4.5 ft, Determine the gross and net allowable bearing pressure using the general (Meyerhof) equation, take FS=3.



Solution:

For $\emptyset' = 34^\circ$, $N_q = 29.44$ and $N_\gamma = 41.06$

$q = [105 \times 2 + (118 - 62.4) \times 2)] = 321.2$ Ib/ft^3

$$F_{qs} = 1 + \frac{4.5}{4.5} \tan 34° = 1.7$$

$$F_{\gamma s} = 1 - 0.4\frac{4.5}{4.5} = 0.6$$

$\frac{2}{4.5} < 1$ and $\phi' > 0$:

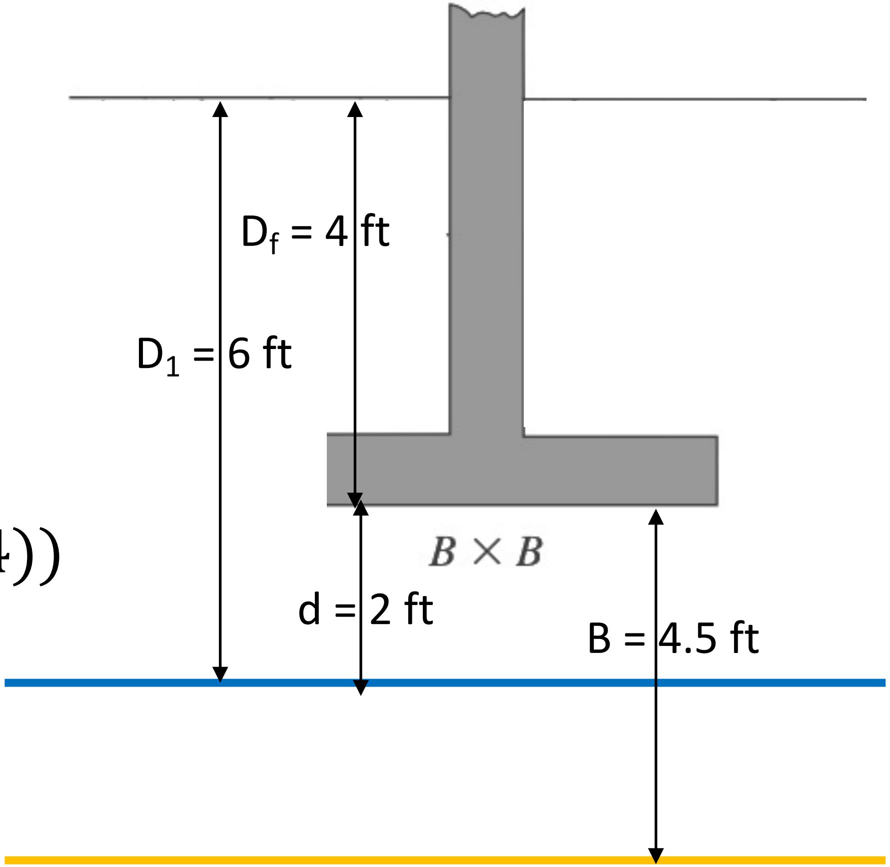
$F_{qd} = 1 + 2\tan\phi'(1-\sin\phi')^2\frac{D_f}{B} \quad = 1 + 2\tan 34^\circ (1-\sin 34^\circ)^2 \frac{2}{4.5} = 1.12 \qquad , F_{\gamma d} = 1$

$F_{\gamma I} = 1$, $F_{qi} = 1$

$q_{u(g)} = 321.2 \times 29.44 \times 1.7 \times 1.12 \times 1 + 0.5 \times 4.5 \times (118-62.4) \times 41.06 \times 0.6 \times 1 \times 1 = 21086.4$ lb/ft^2

Examples

2) A square foundation B x B has to be constructed as shown in the Figure. Assume that $\gamma = 105$ lb/ft^3, γ_{sat} =118 lb/ft^3, $\emptyset' = 34^{\circ}$, D_f =4 ft, and D_1= 6 ft. B = 4.5 ft, Determine the gross and net allowable bearing pressure using the general (Meyerhof) equation, take FS=3.



Solution:

For $\emptyset' = 34^{\circ}$, $N_q = 29.44$ and $N_\gamma = 41.06$

$q = [105 \times 4)] = 420$ Ib/ft^2

$F_{qs} = 1 + \frac{4.5}{4.5} \tan 34° = 1.7$

$F_{\gamma s} = 1 - 0.4 \frac{4.5}{4.5} = 0.6$

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$$

$= (118- 62.4) +$
$2/4.5 \text{ X}(105 - (118 - 62.4))$
$= 77.6$ Ib/ft^3

$\frac{2}{4.5} < 1 \text{ and } \phi' > 0$:

$F_{qd} = 1 + 2 \tan \phi' (1\text{-}\sin \phi')^2 \frac{D_f}{B}$ $= 1 + 2 \tan 34^{\circ} (1\text{-} \sin 34^{\circ})^2 \frac{2}{4.5} = 1.12$, $F_{\gamma d} = 1$

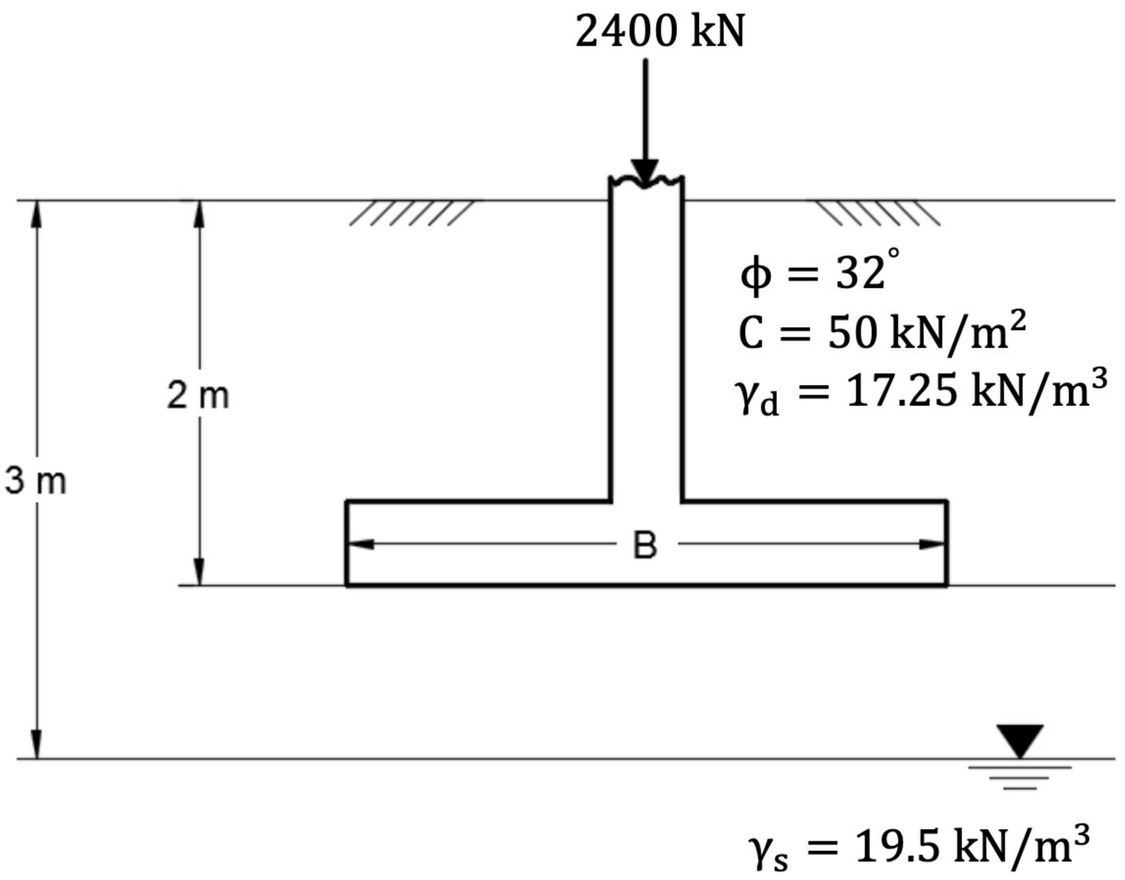
$F_{\gamma I} = 1$, $F_{qi} = 1$

$q_{u(g)} = 420 \times 29.44 \times 1.7 \times 1.12 \times 1 + 0.5 \times 4.5 \times 77.6 \times 41.06 \times 0.6 \times 1 \times 1 = 27844$ Ib/ft^2

Examples

3) The square footing shown below must be designed to carry a 2400 KN load. Use Terzaghi's bearing capacity formula and factor of safety = 3. **Determine** the foundation dimension B in the following two cases:

1. The water table is at 1m below the foundation (as shown).
2. The water table rises to the ground surface.



$q_u = 1.3cN_c + qN_q + 0.4B\gamma N_\gamma$

$q_u = q_{all} \times FS \qquad (q_{all} = \dfrac{Q_{all}}{Area}\ ,\ \ FS = 3)$

Applied load $\leq Q_{all} \rightarrow Q_{all} = 2400kN$

$q_{all} = \dfrac{Q_{all}}{Area} = \dfrac{2400}{B^2}\ , FS = 3 \rightarrow\rightarrow q_u = \dfrac{3 \times 2400}{B^2}$

$c = 50\ kN/m^2$

$q(\text{effective stress}) = \gamma \times D_f = 17.25 \times 2 = 34.5\ kN/m^2$

Since the width of the foundation is not known, assume $d \leq B$

$$\gamma = \bar{\gamma} = \gamma' + \frac{d \times (\gamma - \gamma')}{B}$$

$$\gamma' = \gamma_{sat} - \gamma_w = 19.5 - 10 = 9.5 kN/m^3 \;,\; d = 3 - 2 = 1m$$

$$\rightarrow \bar{\gamma} = 9.5 + \frac{1 \times (17.25 - 9.5)}{B} \rightarrow \bar{\gamma} = 9.5 + \frac{7.75}{B}$$

Assume general shear failure

Note:

Always we design for general shear failure (soil have a high compaction ratio) except if we can't reach high compaction, we design for local shear (medium compaction).

For $\phi = 32^{\circ} \rightarrow N_c = 44.04,\; N_q = 28.52,\; N_\gamma = 26.87$ Table of Terzaghi equation

Now substitute from all above factors on terzaghi equation:

$$\frac{7200}{B^2} = 1.3 \times 50 \times 44.04 + 34.5 \times 28.52 + 0.4 \times B \times \left(9.5 + \frac{7.75}{B}\right) \times 26.87$$

$$\frac{7200}{B^2} = 3923.837 + 102.106\ B$$

Multiply both sides by $(B^2) \rightarrow\ 102.106\ B^3 + 3923.837B^2 - 7200 = 0.0$

$\rightarrow B = 1.33m$✓.

2.

All factors remain unchanged except q and γ:

$q(\text{effective stress}) = (19.5 - 10) \times 2 = 19\ kN/m^2$

$\gamma = \gamma' = 19.5 - 10 = 9.5\ kN/m^3$

Substitute in terzaghi equation:

$$\frac{7200}{B^2} = 1.3 \times 50 \times 44.04 + 19 \times 28.52 + 0.4 \times B \times 9.5 \times 26.87$$

$$\frac{7200}{B^2} = 3404.48 + 102.106B$$

Multiply both sides by $(B^2) \rightarrow 102.106\ B^3 + 3404.48\ B^2 - 7200 = 0.0$

$\rightarrow B = 1.42m$✓.

Note that as the water table elevation increase the required width (B) will also increase to maintain the factor of safety (3).

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Bearing Capacity (bearing load) of Soil (7)

- Local shear failure
- SPT method (N blows): Part (1)

Local Shear Failure

When the soil failed by local shear failure, the shear strength parameters should be reduced. This is because the status of soil is of moderate properties between compact (dense) or cohesive soil. So, ϕ' and c' equals :

$\phi'^{*} = tan^{-1}(0.67 \tan \phi')$

$c'^{*} = 0.67\ c'$

The reduction of the values of effective cohesion and the effective angle of internal friction was proposed by Terzaghi. The same equation derived by Terzaghi used, just the new parameters used for solution.

Example:

A strip footing 1 m wide and its base is located at a depth of 0.8 m below the ground surface. The properties of the foundation soil are: $\gamma = 18\ kN/m^3$ and

$\phi = 20°$ and $C = 30\ kN/m^2$. Determine the safe bearing capacity, using a factor of safety 3. Use Terzaghi's analysis. Assume soil's local shear failure.

Solution:

1) Find $C'^* = 0.67 \times 30 = 20.1\ kN/m^2$, $\phi'^* = \tan^{-1}(0.67 \tan 20°) = 13.7°$
2) Enter the table of bearing capacity parameters to find them using $\phi'^* = 13.7°$
3) Substitute all data in the equation of bearing capacity by Terzaghi to find q_u.
4) Divide q_u by FS to get q_a .

13.7

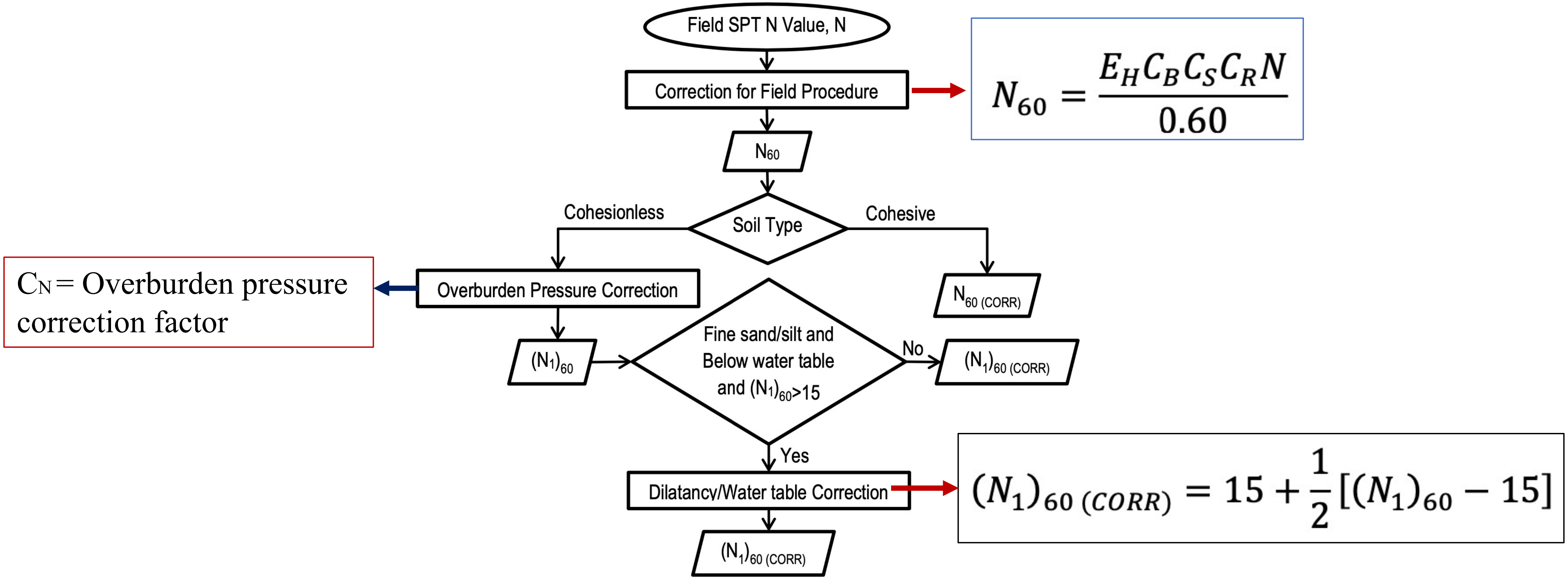
Table 4.1 Terzaghi's Bearing Capacity Factors—Eqs. (4.15), (4.13), and (4.11).[a]

ϕ'	N_c	N_q	N_γ[a]	ϕ'	N_c	N_q	N_γ[a]
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

[a]From Kumbhojkar (1993)

Bearing Capacity by Standard Penetration Test:

The Standard Penetration Test (SPT) is widely used to determine the in-situ properties of soil. The test is especially suited for cohesionless soils as the correlation between the SPT value and φ is now well established.



Flow chart of different types of correction of SPT (N value)

Correlations between SPT N values and Different Parameters of Soil

Table (1): Penetration Resistance and Soil Properties on the Basis of SPT (Cohesionless Soil: Fairly reliable) (Peck et. al. 1974; Bowles, 1977)

SPT N-value		0 to 4	4 to 10	10 to 30	30 to 50	>50
Compactness		very loose	loose	medium	dense	very dense
Relative Density, D_r (%)		0 to 15	15 to 35	35 to 65	65 to 85	85 to 100
Angle of Internal Friction, φ(°)		<28	28 to 30	30 to 36	36 to 41	>41
Unit Weight (moist)	pcf	<100	95 to 125	110 to 130	110 to 140	>130
	kN/m^3	<15.7	14.9 to 19.6	17.3 to 20.4	17.3 to 22.0	>20.4
Submerged unit weight	pcf	<60	55 to 65	60 to 70	65 to 85	>75
	kN/m^3	<9.4	8.6-10.2	9.4 to 11.0	10.5 to 13.4	>11.8

Table (1): Penetration Resistance and Soil Properties on the Basis of SPT (Cohesive Soil: rather unreliable) (Peck et. al. 1974; Bowles, 1977)

SPT N-value		0 to 2	2 to 4	4 to 8	8 to 16	16 to 32	>32
Consistency		very soft	soft	medium	stiff	very stiff	hard
Unconfined Comp. Test	lb/ft^2	0 to 250	250 to 500	500 to 1000	1000 to 2000	2000 to 4000	>4000
	kPa	0 to 25	25 to 50	50 to 100	100 to 200	200 to 400	>400
Unit Weight (Saturated)	pcf	<100	100 to120	110 to 125	115 to130	120 to 140	>130
	kN/m^3	<15.7	15.7 to 18.8	17.3 to 19.6	18.1 to 20.4	18.8 to 22.0	>20.4

Example on correction of SPT (N values)

Find out the corrected SPT N-value.

Given: Field N-value = 15, Depth = 6 m below ground level, Soil Type: Fine sand with trace mica, no water table was observed within this depth. Standard penetration test was performed with standard split spoon sampler and hand dropped donut hammer. Bore diameter was 100 mm.

Solution

Step 1: Correction for Field procedure is made for all types of soil

E_H = Hammer efficiency (Table 1.1) = 0.60

C_B = Borehole diameter correction (Table 1.1) = 1.00

C_S = Sampler correction (Table 1.1) = 1.00

C_R = Rod length* correction (Table 1.1) = 0.85

*total rod length = depth+legth above borehole (typically 1~2m; let 1.5m) = 6+1.5=7.5m

N = Measured SPT N-value in field = 15

$$N_{60} = \frac{E_H C_B C_S C_R N}{0.60} = \frac{0.60 \times 1.00 \times 1.00 \times 0.85 \times 15}{0.60} = 12.75 \approx 13$$

Step 2: Soil type is cohesionless, overburden pressure correction must be made. For overburden pressure correction effective overburden pressure and hence unit weight of soil must be known. Assume average unit weight of soil from N-value as follows

$$\gamma_{moist} = 16.0 + 0.1N_{60}\ (kN/m^3) = 16.0 + 0.1 \times 13 = 17.3\ kN/m^3$$

$$\sigma_0 = \gamma z = 17.3 \times 6 = 103.8\ kN/m^2$$

$$\sigma_0' = \sigma_0 - u = 103.8 - 0 = 103.8\ kN/m^2$$

Where u is pore water pressure. Here no water table is observed at 6 m.

$$(N_1)_{60} = C_N \times N_{60} = 0.77 \log\left(\frac{2000}{\sigma_0'}\right) \times N_{60}$$

$$(N_1)_{60} = 0.77 \log\left(\frac{2000}{103.8}\right) \times 13 = 0.99 \times 13 \approx 13 \ \leq 2N_{60}$$

Step 3: Since soil in not under water table and $(N_1)_{60} < 15$ hence no need for dilatancy correction.

$$N_{corr} = (N_1)_{60} = 13\ (\boldsymbol{Ans})$$

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Thank You

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Bearing Capacity (bearing load) of Soil (8)

- SPT method (N blows): Part (2)

Allowable Bearing Capacity from N-value for Cohesionless Soil

It is difficult to collect undisturbed sample in cohesionless soil hence extensive research have been made to find out the allowable bearing capacity of shallow foundation in cohesionless soil from SPT N-value.

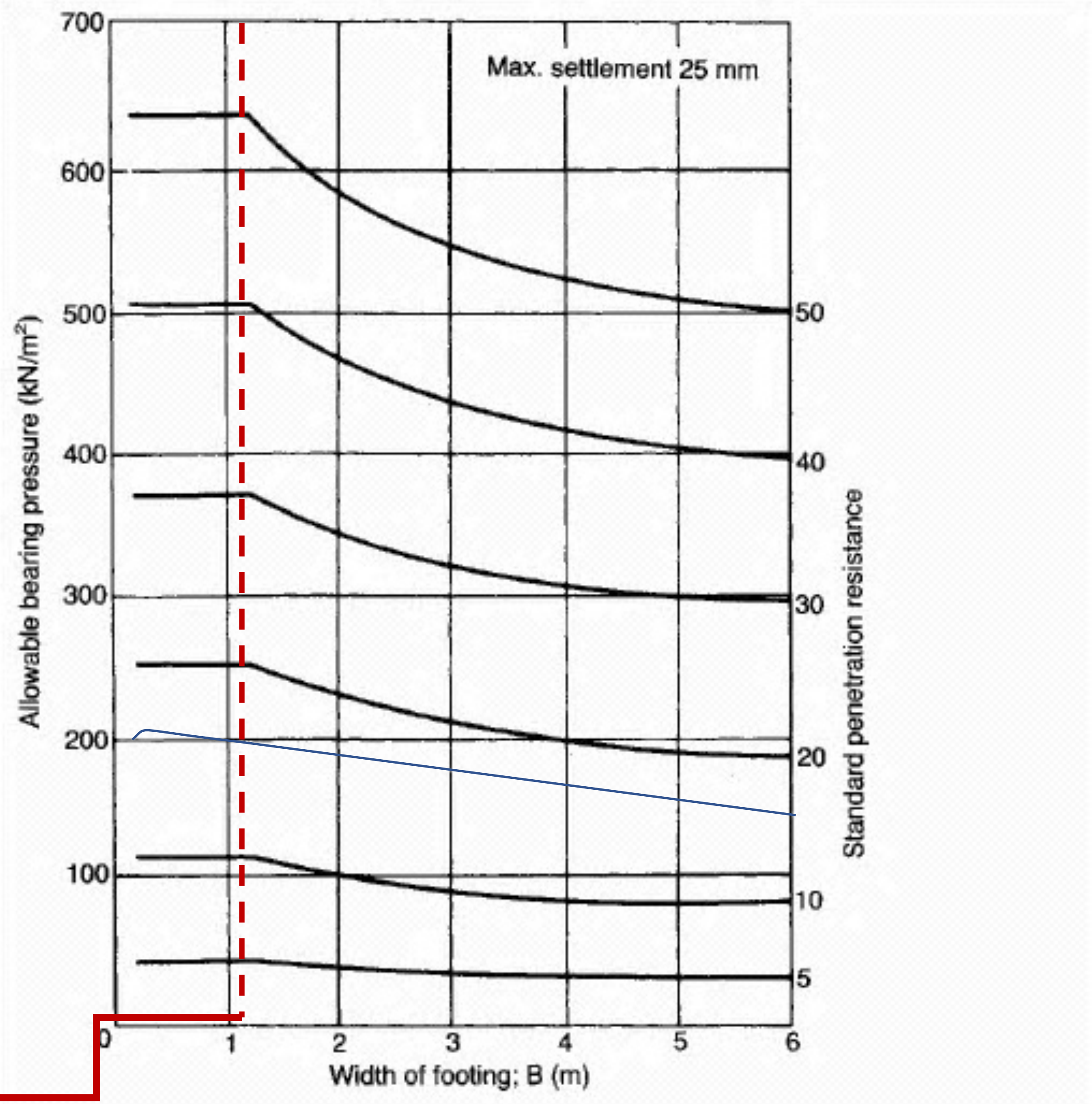
To obtain the allowable bearing capacity of sands is presented empirical correlations between standard penetration resistance, width of footing and the bearing pressure limiting maximum settlement to 25mm (and differential settlement to 75%of maximum settlement).

If the sand at foundation level is saturated, the pressures obtained from Figure should be reduced by one-half if the depth/breadth ratio of the footing is zero, and reduced by one-third if the depth/breadth ratio is unity.

Peck proposed that linear interpolation should be used between a reduction of 50% if the water table is at ground level and zero reduction if the water table is at depth B below the foundation. Thus the provisional value of allowable bearing pressure obtained from Figure should be multiplied by a factor C_w, given by:

$$C_w = 0.5 + 0.5\frac{D_w}{D + B}$$

Allowable Bearing Capacity	References
Originally in graphical form $$q_a(kPa) = 12N\frac{1}{c_w c_d}\left(\frac{s}{25.4}\right) for\ B \le 1.2m$$ $$q_a\ (kPa) = 8N\left(\frac{3.28B+1}{3.28B}\right)^2 \frac{1}{c_w c_d}\ \left(\frac{s}{25.4}\right)\ for\ B > 1.2m$$ $$q_a(kPa) = 8N\frac{1}{c_w c_d}\left(\frac{s}{25.4}\right)\ for\ raft$$ Where c_d = Depth factor $=1 + 0.25\frac{D_f}{B}$ c_w = water correction factor $=2 - \frac{D_w}{2B} \le 2$ for surface footings $=2 - \frac{D_f}{2B} \le 2$ for fully submerged footing $d_w \le d_f$ s = tolerable settlement (mm) B = width of the footing (m) D_f = depth of footing (m) D_w= depth of water (m) N = Lowest (average) uncorrected N-value from depth of footing to D_f+B every 0.76m (2.5ft). water table correction suggested.	Terzaghi and Peck (1948)



1.2 m for linear portion of curves

Figure Relationship between standard penetration resistance and allowable bearing pressure.

$q_a(kPa) = 12NF_d\left(\frac{s}{25.4}\right)\ for\ B \le 1.2m$ $q_a\ (kPa) = 8N\left(\frac{3.28B+1}{3.28B}\right)^2 F_d\left(\frac{s}{25.4}\right)\ for\ B > 1.2m$ $q_a(kPa) = 8NF_d\left(\frac{s}{25.4}\right)\ for\ raft$ Where F_d = Depth factor $=1 + 0.33\frac{D_f}{B} \le 1.33$ s = tolerable settlement (mm) B = width of the footing (m) D_f = depth of footing (m) N = Average uncorrected N-value from depth of footing to D_f+B. Only water table correction suggested.	Meyerhof (1956)

Example :

A footing of 3m in width is to be located at a depth of 1.5m in a sand deposit, the water table being 3.5m below the surface. Values of standard penetration resistance were determined as detailed in Table. Determine the allowable bearing capacity using the various design methods.

Depth (m)	N	σ'_v (kN/m^2)	C_N	N_1
0.75	8	–	–	–
1.55	7	26	2.0	14
2.30	9	39	1.6	14
3.00	13	51	1.4	18
3.70	12	65	1.25	15
4.45 = 1.5 m + B (3 m)	16	70	1.2	19
5.20	20	–	–	–
				(av. 16)

- Terzaghi and Peck recommended that N values should be determined between foundation level and a depth of approximately B below the foundation;
- in this example N blows taking between depths of 1.5 and 4.5m: the values at depths of 0.75 and 5.20m are therefore superfluous.
- The measured N values are corrected using values of effective overburden pressure are calculated (using $\gamma = 17$ kN/m^3 above the water table and $\gamma = 10$ kN/m^3 below the water table) and the corresponding values of C_N determined. The average of the corrected values $(N_1)_{60}$ is 16. Then referring to Figure, for B = 3m and N=16, the provisional value of **allowable bearing capacity is 165 kN/m^2**. For the given water table level the provisional value should be multiplied by the factor Cw ,where:

$$C_w = 0.5 + \frac{0.5 \times 3.5}{4.5} = 0.89$$

The allowable bearing capacity is given by : $q_a = 0.89 * 165 = 150$kN/m^2

Allowable Bearing Capacity from N-value for Cohesive Soil

Due unreliability of determination of bearing capacity by SPT for cohesive soil, so it will be not considered here in this course.

Thank You

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

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Bearing Capacity (bearing load) of Soil (9-a)

- **Bearing capacity for stratified soil:**
  - a- Bearing Capacity of Layered Soils: Stronger Soil Underlain by Weaker Soil ($C' - \emptyset'$ soil).
    - Bearing Capacity of Layered Soil: Weaker Soil Underlain by Stronger Soil.

Bearing capacity of stratified (non-homogenous) soil

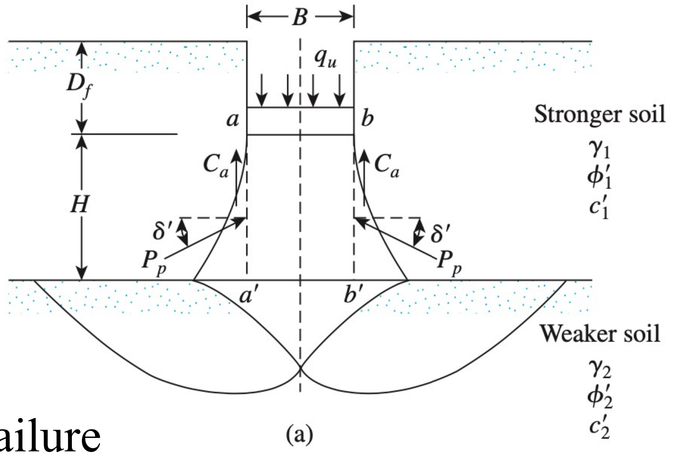
General:

- ✓ The bearing capacity equations presented in the previous presentations involve cases in which the soil supporting the foundation is **<u>homogeneous</u>** and extends to a considerable depth.
- ✓ The cohesion, angle of friction, and unit weight of soil were assumed to remain constant for the bearing capacity analysis.
- ✓ However, in practice, layered soil profiles are often encountered. In such instances, the failure surface at ultimate load may extend through two or more soil layers, and a determination of the ultimate bearing capacity in layered soils can be made in only a limited number of cases.
- ✓ This section features the procedure for estimating the bearing capacity for layered soils proposed by Meyerhof and Hanna (1978) and Meyerhof (1974) in a ($C' - \emptyset'$ soil).

Bearing Capacity of Layered Soils: Stronger Soil Underlain by Weaker Soil, ($C' - \emptyset'$ soil).

The figure shows a shallow, continuous foundation supported by a *stronger soil layer*, underlain by a weaker soil that extends to a great depth. For the two soil layers, the physical parameters are as follows:

	Soil properties		
Layer	**Unit weight**	**Friction angle**	**Cohesion**
Top	γ_1	ϕ'_1	c'_1
Bottom	γ_2	ϕ'_2	c'_2

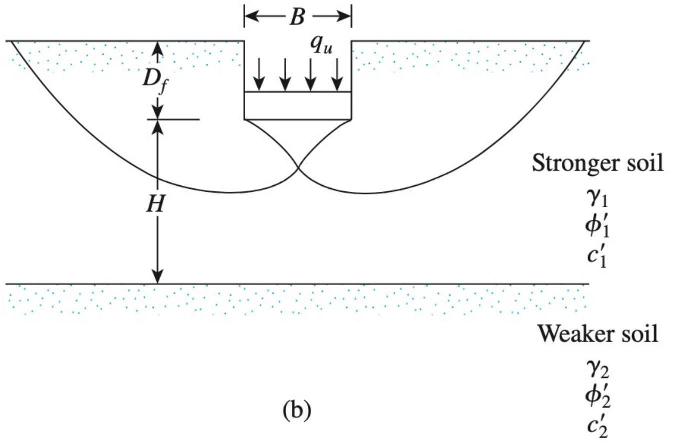


(a)

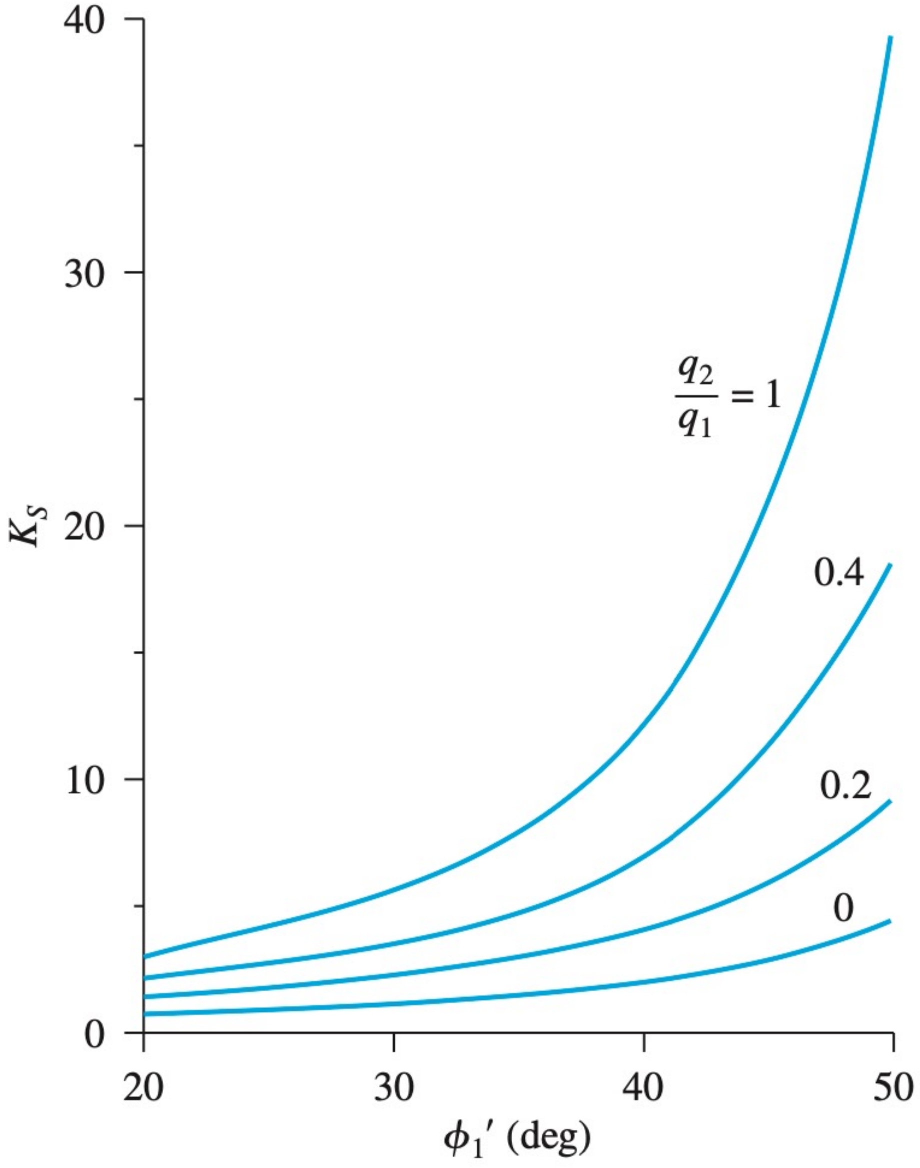
Two cases:

1- H is so small, that cause punching failure in top layer (stronger) and general shear failure in bottom layer (Fig. a)

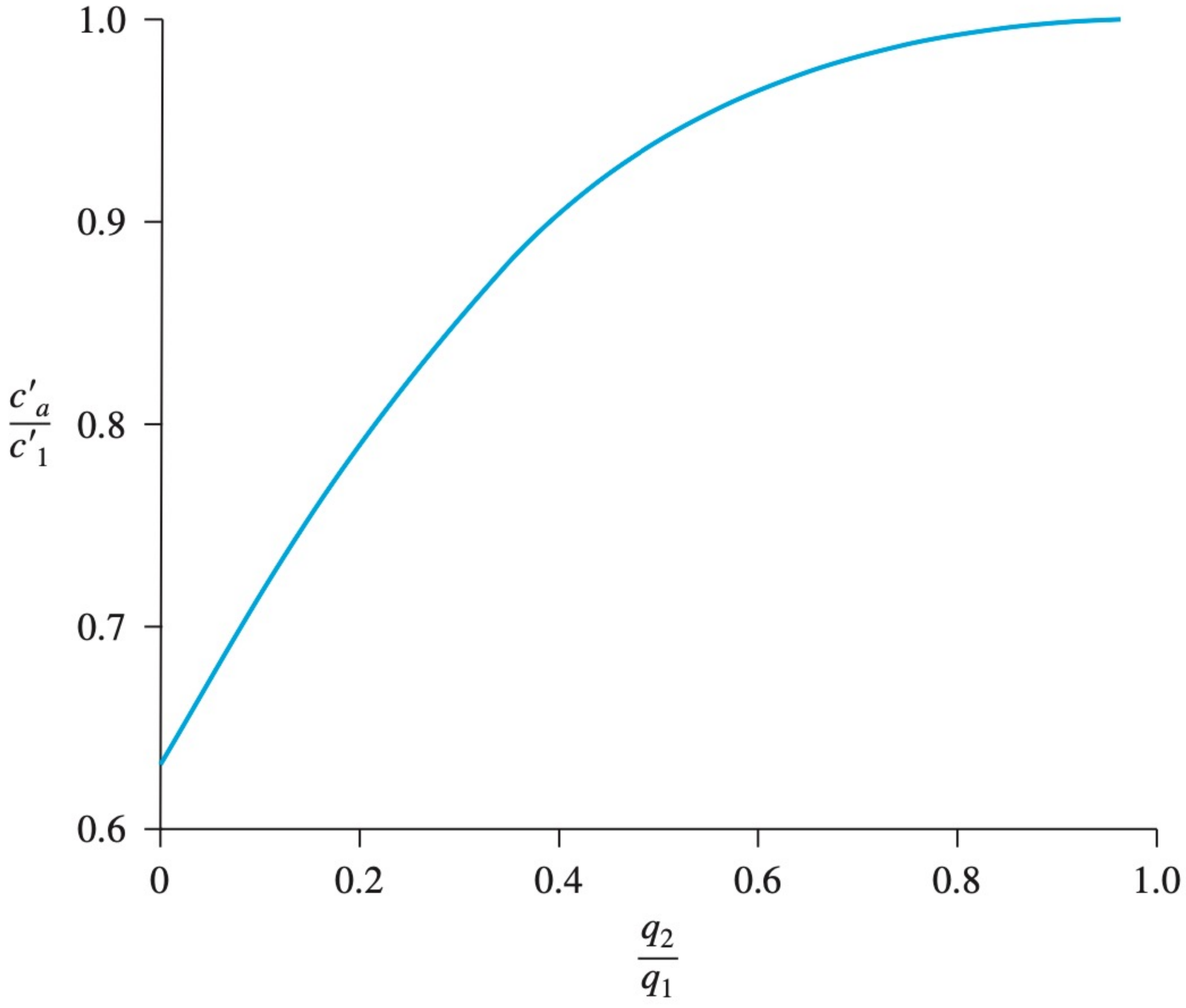
$$q_u = q_b + \frac{2c'_aH}{B} + \gamma_1 H^2\left(1 + \frac{2D_f}{H}\right)\frac{K_s \tan\phi'_1}{\mathrm{B}} - \gamma_1 H \leq q_t$$



(b)



Meyerhof and Hanna's punching shear coefficient K_s



Variation of c'_a/c'_1 with q_2/q_1 based on the theory of Meyerhof and Hanna (1978)

1. *Top layer is strong sand and bottom layer is saturated soft clay* $\emptyset_2 = 0$.

$$q_u = \left(1 + 0.2\,\frac{B}{L}\right)5.14c_{u(2)} + \gamma_1 H^2\left(1 + \frac{B}{L}\right)\left(1 + \frac{2D_f}{H}\right)\frac{K_s \tan\phi_1'}{B} + \gamma_1 D_f \leqslant \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\,\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

where $C_{u(2)}$ = undrained cohesion.

For a determination of K_s

$$\frac{q_2}{q_1} = \frac{c_{u(2)} N_{c(2)}}{\frac{1}{2}\gamma_1 B N_{\gamma(1)}} = \frac{5.14c_{u(2)}}{0.5\gamma_1 B N_{\gamma(1)}}$$

2. *Top layer is stronger sand and bottom layer is weaker sands* ($C_1' = 0$ and $C_2' = 0$)The ultimate bearing capacity can be given as:

$$q_u = \left[\gamma_1(D_f + H)N_{q(2)}F_{qs(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)}F_{\gamma s(2)}\right] + \gamma_1 H^2\left(1 + \frac{B}{L}\right)\left(1 + \frac{2D_f}{H}\right)\frac{K_s \tan\phi_1'}{B} - \gamma_1 H \leqslant q_t$$

where

$$q_t = \gamma_1 D_f N_{q(1)}F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)}F_{\gamma s(1)}$$

Then

$$\frac{q_2}{q_1} = \frac{\frac{1}{2}\gamma_2 B N_{\gamma(2)}}{\frac{1}{2}\gamma_1 B N_{\gamma(1)}} = \frac{\gamma_2 N_{\gamma(2)}}{\gamma_1 N_{\gamma(1)}}$$

3. *Top layer is stronger saturated clay* ($\emptyset_1 = 0$) *and bottom layer is weaker saturated clay* ($\emptyset_2 = 0$). The ultimate bearing capacity can be given as

$$q_u = \left(1 + 0.2\,\frac{B}{L}\right)5.14c_{u(2)} + \left(1 + \frac{B}{L}\right)\left(\frac{2c_a H}{B}\right) + \gamma_1 D_f \leqslant q_t$$

where

$$q_t = \left(1 + 0.2\,\frac{B}{L}\right)5.14c_{u(1)} + \gamma_1 D_f$$

and $c_{u(1)}$ and $c_{u(2)}$ are undrained cohesions. For this case,

$$\frac{q_2}{q_1} = \frac{5.14c_{u(2)}}{5.14c_{u(1)}} = \frac{c_{u(2)}}{c_{u(1)}}$$

Example (E 9.1)

Refer to Figure 5.9a and consider the case of a continuous foundation with $B = 2$ m, $D_f = 1.2$ m, and $H = 1.5$ m. The following are given for the two soil layers:

Top sand layer:

Unit weight $\gamma_1 = 17.5\ \text{kN/m}^3$

$\phi'_1 = 40°$

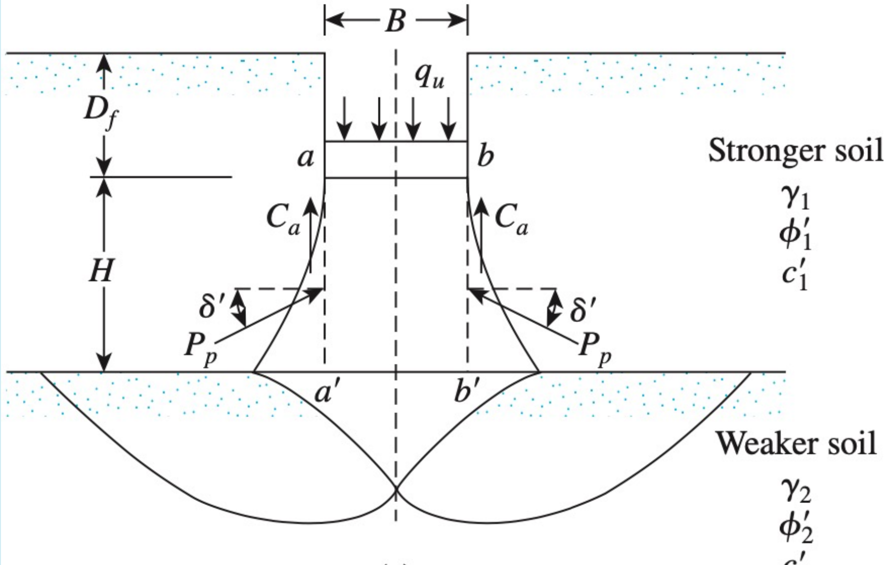
$c'_1 = 0$

Bottom clay layer:

Unit weight $\gamma_2 = 16.5\ \text{kN/m}^3$

$\phi'_2 = 0$

$c_{u(2)} = 30\ \text{kN/m}^2$



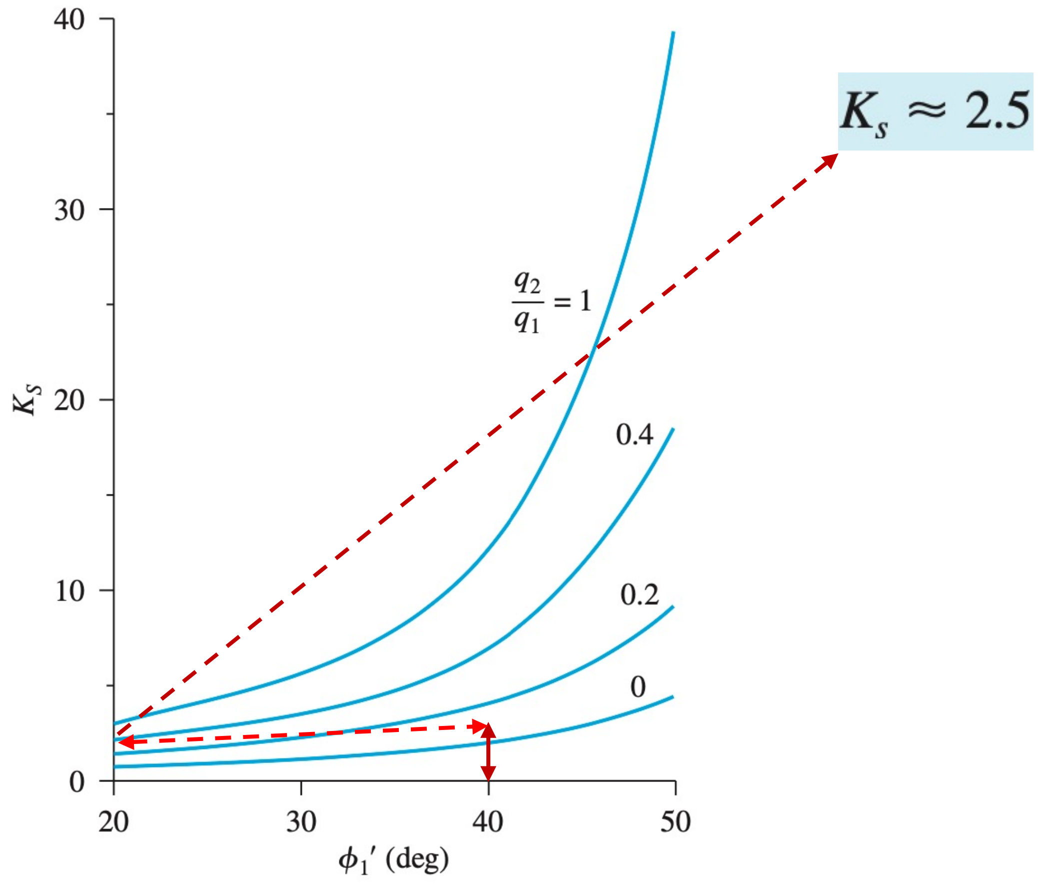
(a)

Determine the gross ultimate load per unit length of the foundation.

Solution (E 9-1)

For $\phi_1' = 40°$, from Table 4.2, $N_\gamma = 109.41$ and

$$\frac{q_2}{q_1} = \frac{c_{u(2)}N_{c(2)}}{0.5\gamma_1 B N_{\gamma(1)}} = \frac{(30)(5.14)}{(0.5)(17.5)(2)(109.41)} = 0.081$$



Meyerhof and Hanna's punching shear coefficient K_s

$$q_u = \left[1 + (0.2)\left(\frac{B}{L}\right)\right]5.14c_{u(2)} + \left(1 + \frac{B}{L}\right)\gamma_1 H^2\left(1 + \frac{2D_f}{H}\right)K_s\frac{\tan\phi_1'}{B} + \gamma_1 D_f$$

$$= [1 + (0.2)(0)](5.14)(30) + (1 + 0)(17.5)(1.5)^2$$

$$\times \left[1 + \frac{(2)(1.2)}{1.5}\right](2.5)\frac{\tan 40}{2.0} + (17.5)(1.2)$$

$$= 154.2 + 107.4 + 21 = 282.6 \text{ kN/m}^2$$

Again, from Eq. (5.26),

$$q_t = \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

From Table 4.2, for $\phi_1' = 40°$, $N_\gamma = 109.4$ and $N_q = 64.20$.
From Table 4.3,

$$F_{qs(1)} = 1 + \left(\frac{B}{L}\right)\tan\phi_1' = 1 + (0)\tan 40 = 1$$

and

$$F_{\gamma s(1)} = 1 - 0.4\frac{B}{L} = 1 - (0.4)(0) = 1$$

so that

$$q_t = (17.5)(1.2)(64.20)(1) + \left(\frac{1}{2}\right)(17.5)(2)(109.4)(1) = 3262.7 \text{ kN/m}^2$$

Hence,

$$q_u = 282.6 \text{ kN/m}^2$$

$$Q_u = (282.6)(B) = (282.6)(2) = \mathbf{565.2\ kN/m}$$

Example (E 9.2)

A foundation 1.5 m × 1 m is located at a depth, D_f, of 1 m in a stronger clay. A softer clay layer is located at a depth, H, of 1 m measured from the bottom of the foundation. For the top clay layer,

Undrained shear strength = 120 kN/m^2
Unit weight = 16.8 kN/m^3

and for the bottom clay layer,

Undrained shear strength = 48 kN/m^2
Unit weight = 16.2 kN/m^3

Determine the gross allowable load for the foundation with an FS of 4.

Solution (E 9.2)

$$q_u = \left(1 + 0.2\frac{B}{L}\right)5.14c_{u(2)} + \left(1 + \frac{B}{L}\right)\left(\frac{2c_a H}{B}\right) + \gamma_1 D_f$$

$$\leq \left(1 + 0.2\frac{B}{L}\right)5.14c_{u(1)} + \gamma_1 D_f$$

Given:

$B = 1$ m $\quad H = 1$ m $\quad D_f = 1$ m

$L = 1.5$ m $\quad \gamma_1 = 16.8$ kN/m^3

From Figure , $c_{u(2)}/c_{u(1)} = 48/120 = 0.4$, the value of $c_a/c_{u(1)} \approx 0.9$, so

$$c_a = (0.9)(120) = 108 \text{ kN/m}^2$$

$$q_u = \left[1 + (0.2)\left(\frac{1}{1.5}\right)\right](5.14)(48) + \left(1 + \frac{1}{1.5}\right)\left[\frac{(2)(108)(1)}{1}\right] + (16.8)(1)$$

$$= 279.6 + 360 + 16.8 = 656.4 \text{ kN/m}^2$$

$$q_t = \left[1 + (0.2)\left(\frac{1}{1.5}\right)\right](5.14)(120) + (16.8)(1)$$

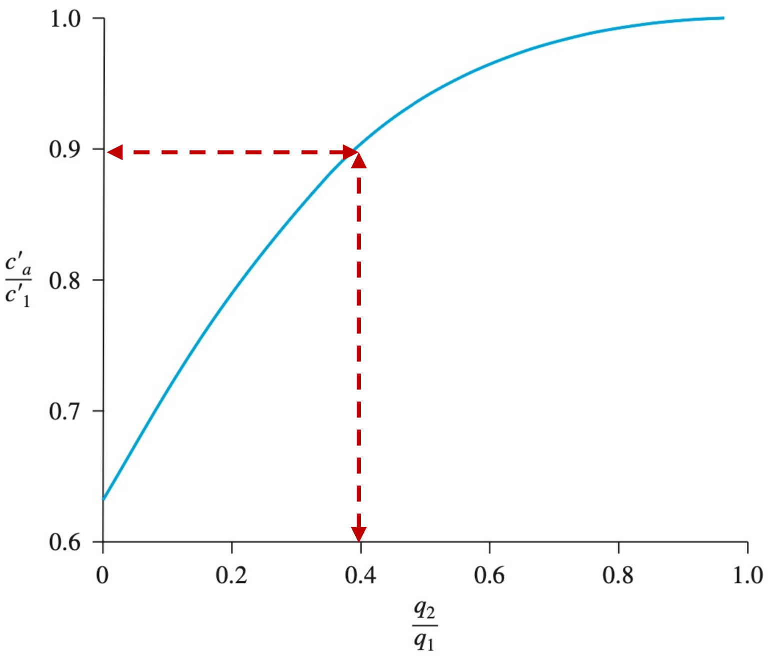
$$= 699 + 16.8 = 715.8 \text{ kN/m}^2$$

Thus $q_u = 656.4$ kN/m^2 (that is, the smaller of the two values calculated above) and

$$q_{all} = \frac{q_u}{FS} = \frac{656.4}{4} = 164.1 \text{ kN/m}^2$$

The total allowable load is

$$(q_{all})(1 \times 1.5) = \mathbf{246.15\ kN}$$



Thank you

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

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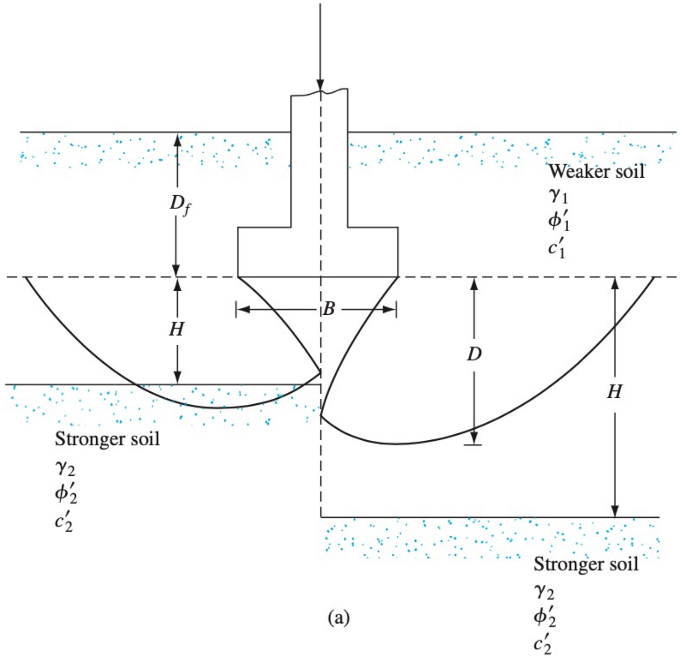
Bearing Capacity (bearing load) of Soil (9-b)

- **Bearing capacity for stratified soil:**
  - b- Bearing Capacity of Layered Soil: Weaker Soil Underlain by Stronger Soil.

Bearing Capacity of Layered Soil: Weaker Soil Underlain by Stronger Soil

When a foundation is supported by a weaker soil layer underlain by a stronger layer (The Figure a), the ratio of q_2/q_1 defined by will be greater than one. Also, if H/B is relatively small, as shown in the left-hand half of the Figure a, the failure surface in soil at ultimate load will pass through both soil layers. However, for larger H/B ratios, the failure surface will be fully located in the top, weaker soil layer, as shown in the right-hand half of Figure 2a. For this condition, the ultimate bearing capacity (Meyerhof, 1974; Meyerhof and Hanna, 1978) can be given by the empirical equation:

$$q_u = q_t + (q_b - q_t)\left(\frac{H}{D}\right)^2 \geq q_t$$



(a)

where

D = depth of failure surface beneath the foundation in the thick bed of the upper weaker soil layer
q_t = ultimate bearing capacity in a thick bed of the upper soil layer
q_b = ultimate bearing capacity in a thick bed of the lower soil layer

$$q_t = c_1' N_{c(1)} F_{cs(1)} + \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

and

$$q_b = c_2' N_{c(2)} F_{cs(2)} + \gamma_2 D_f N_{q(2)} F_{qs(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)} F_{\gamma s(2)}$$

where

$N_{c(1)}, N_{q(1)}, N_{\gamma(1)}$ = bearing capacity factors corresponding to the soil friction angle ϕ_1'
$N_{c(2)}, N_{q(2)}, N_{\gamma(2)}$ = bearing capacity factors corresponding to the soil friction angle ϕ_2'
$F_{cs(1)}, F_{qs(1)}, F_{\gamma s(1)}$ = shape factors corresponding to the soil friction angle ϕ_1'
$F_{cs(2)}, F_{qs(2)}, F_{\gamma s(2)}$ = shape factors corresponding to the soil friction angle ϕ_2'

Meyerhof and Hanna (1978) suggested that

- $D \approx B$ for loose sand and clay
- $D \approx 2B$ for dense sand

Example (E 9-b-1)

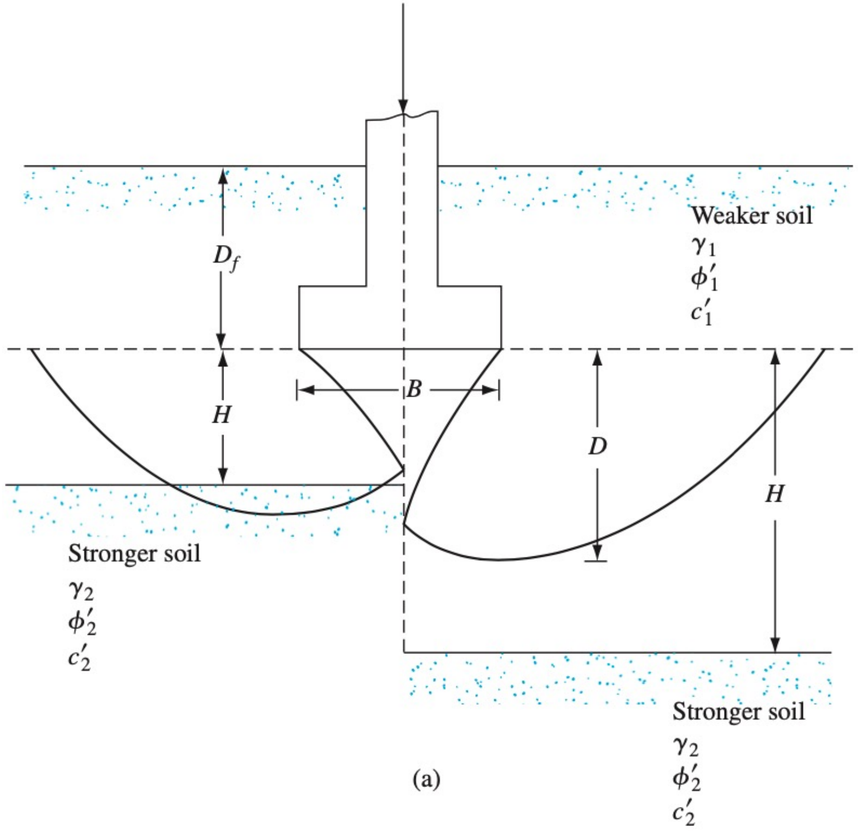
Refer to Figure a. For a layered saturated-clay profile, given: $L = 6$ ft, $B = 4$ ft, $D_f = 3$ ft, $H = 2$ ft, $\gamma_1 = 110$ lb/ft^3, $\phi_1 = 0$, $c_{u(1)} = 1200$ lb/ft^2, $\gamma_2 = 125$ lb/ft^3, $\phi_2 = 0$, and $c_{u(2)} = 2500$ lb/ft^2. Determine the ultimate bearing capacity of the foundation.

Solution (E 9-b-1)

$$\frac{q_2}{q_1} = \frac{c_{u(2)}N_c}{c_{u(1)}N_c} = \frac{c_{u(2)}}{c_{u(1)}} = \frac{2500}{1200} = 2.08 > 1$$

$$q_t = \left(1 + 0.2\frac{B}{L}\right)N_c c_{u(1)} + \gamma_1 D_f$$

$$= \left[1 + (0.2)\left(\frac{4}{6}\right)\right](5.14)(1200) + (3)(110) = 6990.4 + 330 = 7320.4 \text{ lb/ft}^2$$



(a)

$$q_b = \left(1 + 0.2\,\frac{B}{L}\right)N_c c_{u(2)} + \gamma_2 D_f$$

$$= \left[1 + (0.2)\left(\frac{4}{6}\right)\right](5.14)(2500) + (3)(125)$$

$$= 14{,}563.3 + 375 = 14{,}938.3 \text{ lb/ft}^2$$

$$q_u = q_t + (q_b - q_t)\left(\frac{H}{D}\right)^2$$

$$D \approx B$$

$$q_u = 7320.4 + (14{,}938.3 - 7320.4)\left(\frac{2}{4}\right)^2 \approx 9225 \text{ lb/ft}^2 > q_t$$

Hence,

$$q_u = \mathbf{9225\ lb/ft^2}$$

End of part 9-b

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Bearing Capacity (bearing load) of Soil (10)

- **Uplift Capacity of the soil**

Uplift Capacity of Foundations

Foundations may be subjected to uplift forces under special circumstances. During the design process for those foundations, it is desirable to provide a sufficient factor of safety against failure by uplift.

The relationships for the ***uplift capacity*** of foundations in ***granular*** and ***cohesive soils***.

Foundations in Granular Soil ($c = 0$)

Figure U1 shows a shallow continuous foundation that is being subjected to an uplift force. At ultimate load, Q_u the failure surface in soil will be as shown in the figure. The ultimate load can be expressed in the form of a non-dimensional break-out factor, F_q or:

$$F_q = \frac{Q_u}{A\gamma D_f}$$

where A = area of the foundation.

The breakout factor F_q is a function of the soil friction angle φ' and D_f/B.

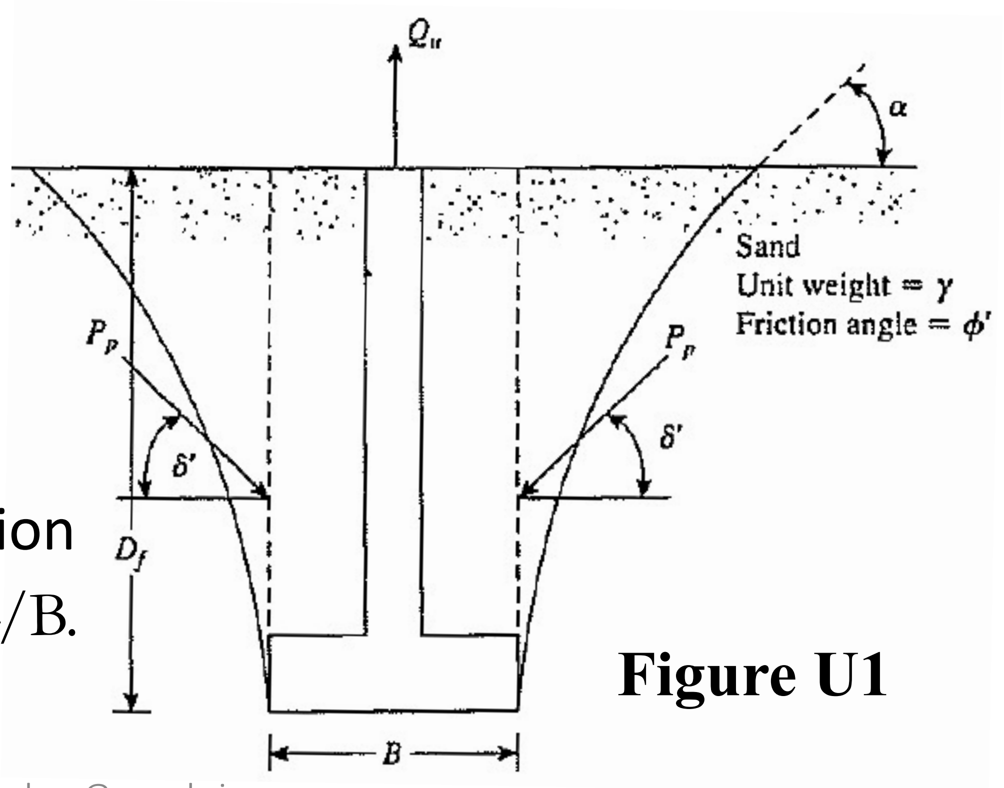

Figure U1

For foundations subjected to uplift:

Shallow foundation : $D_f/B \leq (D_f/B)_{cr}$

Deep foundation: $D_f/B > (D_f/B)_{cr}$

The break-out factor can be found by the following two expressions

$$F_q = 1 + 2\left[1 + m\left(\frac{D_f}{B}\right)\right]\left(\frac{D_f}{B}\right)K_u \tan\phi'$$ Eq: U-1

(for shallow circular foundations)

$$F_q = 1 + \left\{\left[1 + 2m\left(\frac{D_f}{B}\right)\right]\left(\frac{B}{L}\right) + 1\right\}\left(\frac{D_f}{B}\right)K_u \tan\phi'$$ Eq: U-2

(for shallow rectangular foundations)

where

m = a coefficient which is a function of ϕ'

K_u = nominal uplift coefficient

For rectangular foundations, Das and Jones (1982) recommended that

$$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}} = \left(\frac{D_f}{B}\right)_{\text{cr-square}}\left[0.133\left(\frac{L}{B}\right) + 0.867\right] \leq 1.4\left(\frac{D_f}{B}\right)_{\text{cr-square}}$$

Eq: U-3

Table: U-1: Variation of *Ku, m* and (Df/B)

Soil friction angle φ' (deg)	Ku	m	(Df/B)cr for square and circular foundations
20	0.856	0.05	2.5
25	0.888	0.10	3.0
30	0.920	0.15	4.0
35	0.936	0.25	5.0
40	0.960	0.35	7.0
45	0.960	0.50	9.0

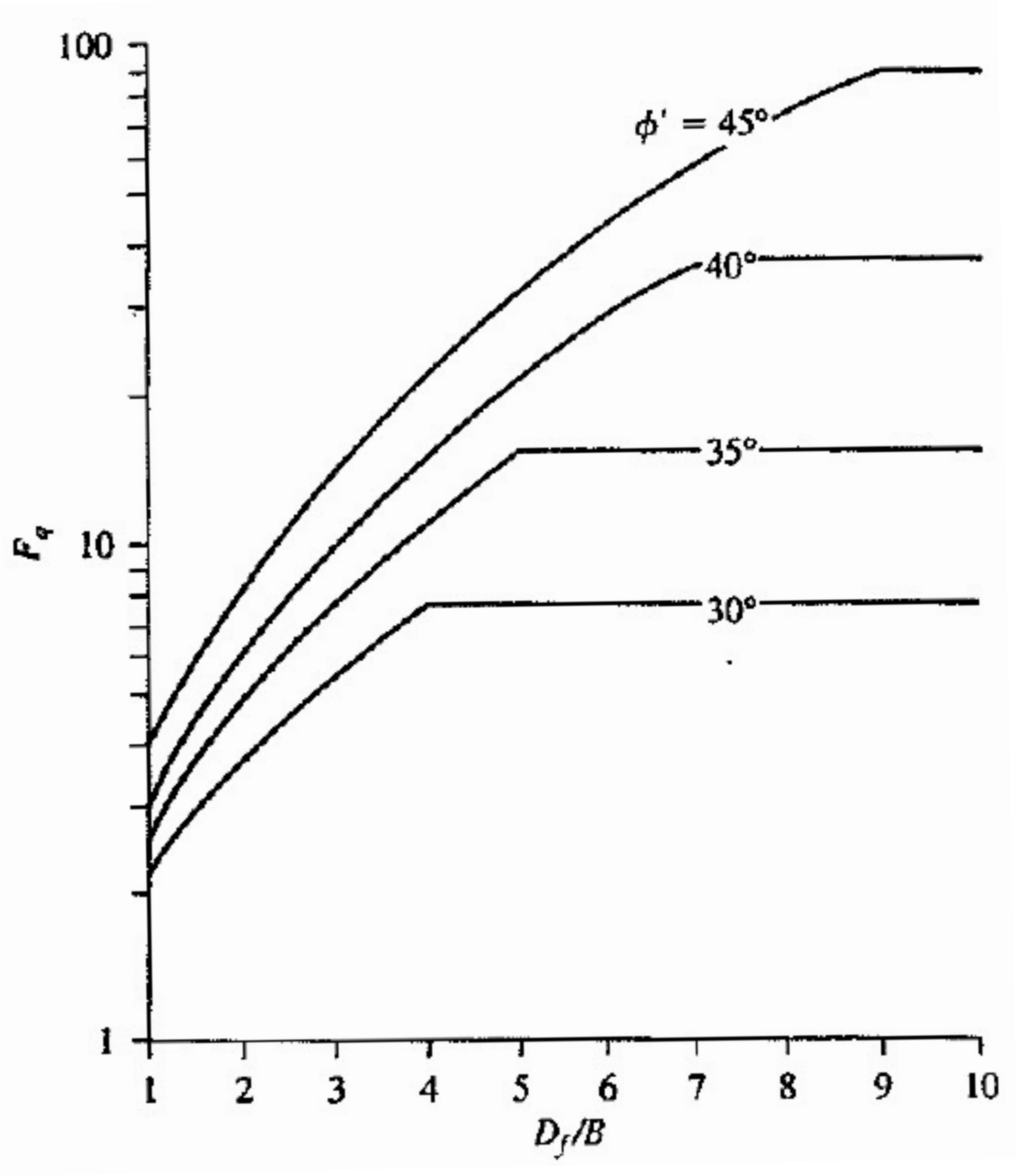

Figure U2: variation of F_q with D_f/B

A step-by-step procedure to estimate the uplift capacity of foundations in granular soil follows.

Step 1: Determine, D_f, B, L, and ϕ'

Step 2: Calculate D_f/B.

Step 3: Using Table U1 and Eq. (U3), *calculate* $(D_f/B)_{cr}$

Step 4: If D_f/B is less than or equal *to* $(D_f/B)cr$ it is a shallow foundation.

Step 5: If $D_f/B > (D_f/B)_{cr}$ it is a deep foundation.

Step 6: For shallow foundations, use D_f/B calculated in Step 2 in Eq. (U1) or (U2) to estimate F_q. Thus, $Q_u = F_q A g D_f$.

Step 7: For deep foundations, substitute $(D_f/B)_{cr}$ *for* D_f/B in Eq. (U1) or (U2) *to* obtain F_q from which the ultimate load Q_u may be obtained.

Foundations in Cohesive Soil ($\phi = 0$)

The ultimate uplift capacity, Q_u of a foundation in a purely cohesive soil can be expressed as:

$$Q_u = A\,(\gamma D_f + c_u F_c) \qquad eq\text{: (U4)}$$

where A = area of the foundation

Cu: undrained shear strength of clay

Fc: Break-out Factor

As in the case of foundations in granular soil, the breakout factor Fc increases with embedment ratio and reaches a maximum value of $F_c = F_c^*$ *at* $D_f/B = (D_f/B)_{cr}$ *and remains constant thereafter.*

Das (1978) also reported some model test results with square and rectangular foundations. Based on these test results, it was proposed that:

$$\left(\frac{D_f}{B}\right)_{\text{cr-square}} = 0.107c_u + 2.5 \leq 7 \qquad \text{Eq: U5}$$

where

$\left(\frac{D_f}{B}\right)_{\text{cr-square}}$ = critical embedment ratio of square (or circular) foundations

c_u = undrained cohesion, in kN/m^2

It was also observed by Das (1980) that

$$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}} = \left(\frac{D_f}{B}\right)_{\text{cr-square}}\left[0.73 + 0.27\left(\frac{L}{B}\right)\right] \leq 1.55\left(\frac{D_f}{B}\right)_{\text{cr-square}} \qquad \text{Eq: U6}$$

where

$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}}$ = critical embedment ratio of rectangular foundations

L = length of foundation

Das' Empirical Method for Qu for Cohesive Soil

$$\alpha' = \frac{\frac{D_f}{B}}{\left(\frac{D_f}{B}\right)_{cr}}$$

Eq: U7

$$\beta' = \frac{F_c}{F_c^*}$$

Eq: U8

α' and β' are two nondimensional factors

$$F^*_{c\text{-rectangular}} = 7.56 + 1.44\left(\frac{B}{L}\right)$$

Eq: U9

where $F^*_{c\text{-rectangular}}$ = breakout factor for deep rectangular foundations

A step-by-step procedure to estimate the uplift capacity of foundations in cohesive soil follows.

Step 1: Determine the representative value of the undrained cohesion, c_u

Step 2: Determine the critical embedment ratio (D_f/B) using Eqs. (U5) and (U6).

Step 3: Determine *the* D_f/B ratio for the foundation.

Step 4: If $D_f/B > (D_f/B)_{cr}$ as determined in Step 2, it is a deep foundation. However, if $D_f/B \leq (D_f/B)_{cr}$ it is a shallow foundation.

Step 5: For $D_f/B > (D_f/B)_{cr}$

$$F_c = F_c^* = 7.56 + 1.44\left(\frac{B}{L}\right)$$

Thus,

$$Q_u = A\left\{\left[7.56 + 1.44\left(\frac{B}{L}\right)\right]c_u + \gamma D_f\right\}$$ Eq: U9

where A = area of the foundation.

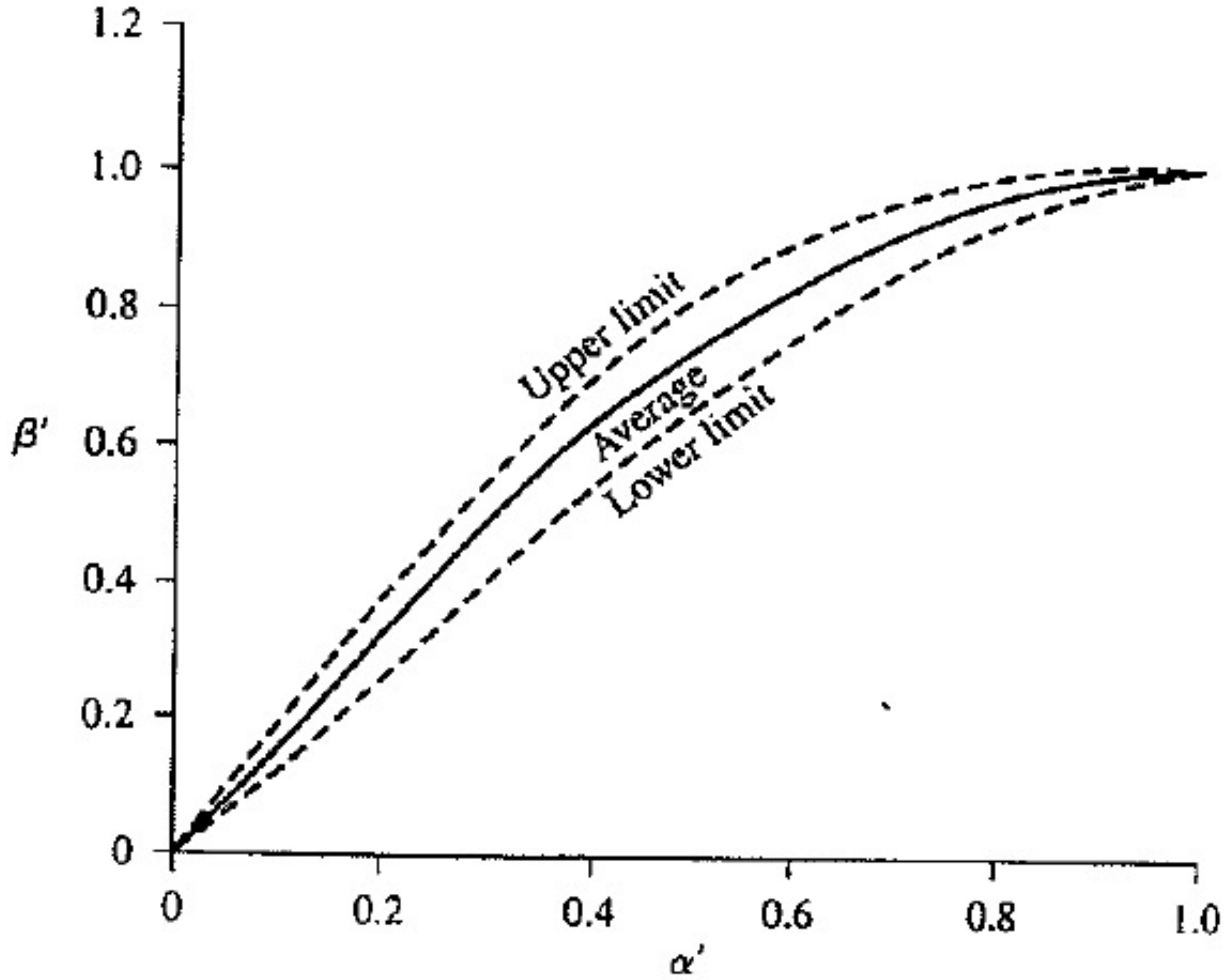

Fig U-3

Plot of β' versus α'

Step 6: for $D_f/B \le (D_f/B)_{cr}$

$$Q_u = A(\beta' F_c^{*} c_u + \gamma D_f) = A\left\{\beta'\left[7.56 + 1.44\left(\frac{B}{L}\right)\right]c_u + \gamma D_f\right\}$$ Eq: U10

The value of β' can be obtained from the average curve of Fig U-3

Example-1-

Consider a circular foundation in sand. Given for the foundation: **diameter**, $B = 1.5$ m and depth of embedment, $D_f = 1.5$ m. Given for the sand: **unit weight**, $\gamma = 17.4\ \text{kN/m}^3$, and friction angle, $\phi' = 35°$. Calculate the ultimate bearing capacity.

Solution

$D_f/B = 1.5/1.5 = 1$ and $\phi' = 35°$. For circular foundation, $(D_f/B)_{cr} = 5$. Hence, it is a shallow foundation. From Eq: U-1

$$F_q = 1 + 2\left[1 + m\left(\frac{D_f}{B}\right)\right]\left(\frac{D_f}{B}\right)K_u \tan\phi'$$

For $\phi' = 35°$, $m = 0.25$, and $K_u = 0.9$ table U-1 So

$$F_q = 1 + 2[1 + (0.25)(1)](1)(0.936)(\tan 35) = 2.638$$

So

$$Q_u = F_q \gamma A D_f = (2.638)(17.4)\left[\left(\frac{\pi}{4}\right)(1.5)^2\right](1.5) = \mathbf{121.7\ kN}$$ ■

Example-2-

A rectangular foundation in a saturated clay measures 1.5 m × 3 m. Given: $D_f = 1.8$ m, $c_u = 52$ kN/m^2, and $\gamma = 18.9$ kN/m^3. Estimate the ultimate uplift capacity.

Solution

From Eq. (4.42)

$$\left(\frac{D_f}{B}\right)_{\text{cr-square}} = 0.107c_u + 2.5 = (0.107)(52) + 2.5 = 8.06$$

So use $(D_f/B)_{\text{cr-square}} = 7$. Again from **Eq-U6-**

$$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}} = \left(\frac{D_f}{B}\right)_{\text{cr-square}}\left[0.73 + 0.27\left(\frac{L}{B}\right)\right]$$

$$= 7\left[0.73 + 0.27\left(\frac{3}{1.5}\right)\right] = 8.89$$

Check: $$1.55\left(\frac{D_f}{B}\right)_{\text{cr-square}} = (1.55)(7) = 10.85$$

So use $(D_f/B)_{\text{cr-rectangular}} = 8.89$. The actual embedment ratio is $D_f/B = 1.8/1.5 = 1.2$.

Hence, this is a shallow foundation.

$$\alpha' = \frac{\dfrac{D_f}{B}}{\left(\dfrac{D_f}{B}\right)_{cr}} = \frac{1.2}{8.89} = 0.135$$

Referring to the average curve of Fig U-3 for $\alpha' = 0.135$, the magnitude of $\beta' = 0.2$. From Eq: U10

$$Q_u = A\left\{\beta'\left[7.56 + 1.44\left(\frac{B}{L}\right)\right]c_u + \gamma D_f\right\}$$

$$= (1.5)(3)\left\{(0.2)\left[7.56 + 1.44\left(\frac{1.5}{3}\right)\right](52) + (18.9)(1.8)\right\} = \mathbf{540.6\ kN} \quad ■$$

Thank You

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

Instructor: Professor Dr. Hussein M. Ashour Al.Khuzaie (Ph.D., Civil Engineering, Foundation Engineering and Structures);

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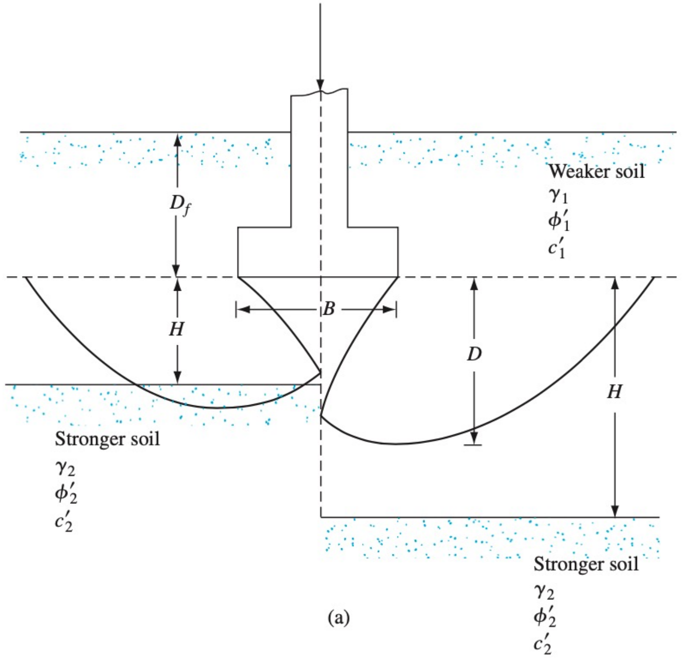
Bearing Capacity (bearing load) of Soil (9-b)

- **Bearing capacity for stratified soil:**
  - b- Bearing Capacity of Layered Soil: Weaker Soil Underlain by Stronger Soil.

Bearing Capacity of Layered Soil: Weaker Soil Underlain by Stronger Soil

When a foundation is supported by a weaker soil layer underlain by a stronger layer (The Figure a), the ratio of q_2/q_1 defined by will be greater than one. Also, if H/B is relatively small, as shown in the left-hand half of the Figure a, the failure surface in soil at ultimate load will pass through both soil layers. However, for larger H/B ratios, the failure surface will be fully located in the top, weaker soil layer, as shown in the right-hand half of Figure 2a. For this condition, the ultimate bearing capacity (Meyerhof, 1974; Meyerhof and Hanna, 1978) can be given by the empirical equation:

$$q_u = q_t + (q_b - q_t)\left(\frac{H}{D}\right)^2 \geq q_t$$



(a)

where

D = depth of failure surface beneath the foundation in the thick bed of the upper weaker soil layer

q_t = ultimate bearing capacity in a thick bed of the upper soil layer

q_b = ultimate bearing capacity in a thick bed of the lower soil layer

$$q_t = c'_1 N_{c(1)} F_{cs(1)} + \gamma_1 D_f N_{q(1)} F_{qs(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} F_{\gamma s(1)}$$

and

$$q_b = c'_2 N_{c(2)} F_{cs(2)} + \gamma_2 D_f N_{q(2)} F_{qs(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)} F_{\gamma s(2)}$$

where

$N_{c(1)}, N_{q(1)}, N_{\gamma(1)}$ = bearing capacity factors corresponding to the soil friction angle ϕ'_1
$N_{c(2)}, N_{q(2)}, N_{\gamma(2)}$ = bearing capacity factors corresponding to the soil friction angle ϕ'_2
$F_{cs(1)}, F_{qs(1)}, F_{\gamma s(1)}$ = shape factors corresponding to the soil friction angle ϕ'_1
$F_{cs(2)}, F_{qs(2)}, F_{\gamma s(2)}$ = shape factors corresponding to the soil friction angle ϕ'_2

Meyerhof and Hanna (1978) suggested that

- $D \approx B$ for loose sand and clay
- $D \approx 2B$ for dense sand

Example (E 9-b-1)

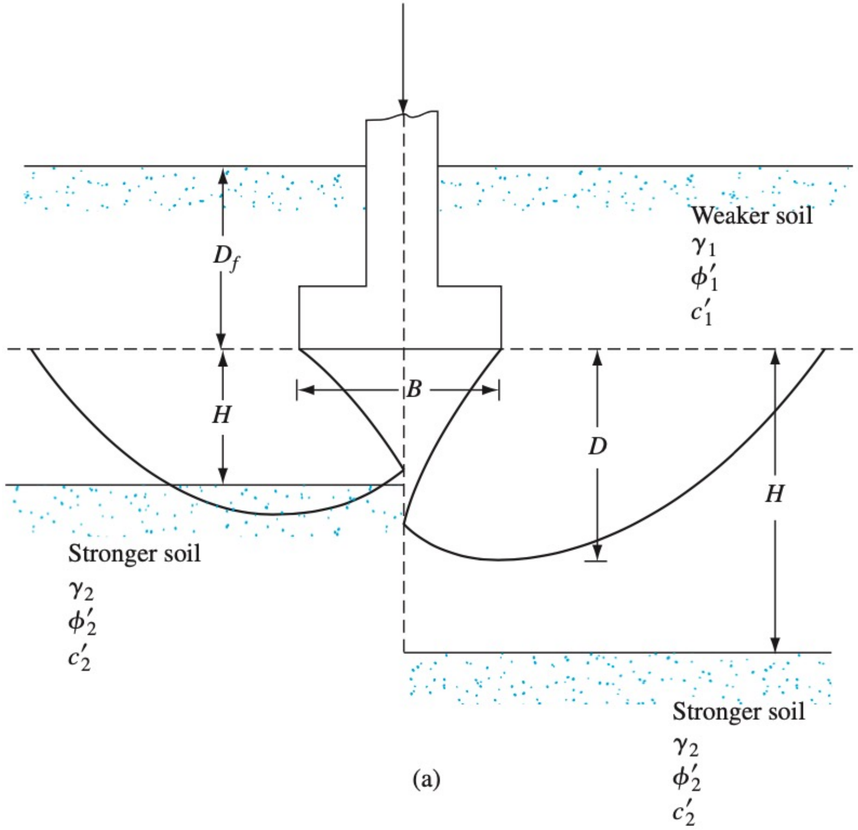
Refer to Figure a. For a layered saturated-clay profile, given: $L = 6$ ft, $B = 4$ ft, $D_f = 3$ ft, $H = 2$ ft, $\gamma_1 = 110$ lb/ft^3, $\phi_1 = 0$, $c_{u(1)} = 1200$ lb/ft^2, $\gamma_2 = 125$ lb/ft^3, $\phi_2 = 0$, and $c_{u(2)} = 2500$ lb/ft^2. Determine the ultimate bearing capacity of the foundation.

Solution (E 9-b-1)

$$\frac{q_2}{q_1} = \frac{c_{u(2)}N_c}{c_{u(1)}N_c} = \frac{c_{u(2)}}{c_{u(1)}} = \frac{2500}{1200} = 2.08 > 1$$

$$q_t = \left(1 + 0.2\frac{B}{L}\right)N_c c_{u(1)} + \gamma_1 D_f$$

$$= \left[1 + (0.2)\left(\frac{4}{6}\right)\right](5.14)(1200) + (3)(110) = 6990.4 + 330 = 7320.4 \text{ lb/ft}^2$$



(a)

$$q_b = \left(1 + 0.2\,\frac{B}{L}\right)N_c c_{u(2)} + \gamma_2 D_f$$

$$= \left[1 + (0.2)\left(\frac{4}{6}\right)\right](5.14)(2500) + (3)(125)$$

$$= 14{,}563.3 + 375 = 14{,}938.3 \text{ lb/ft}^2$$

$$q_u = q_t + (q_b - q_t)\left(\frac{H}{D}\right)^2$$

$$D \approx B$$

$$q_u = 7320.4 + (14{,}938.3 - 7320.4)\left(\frac{2}{4}\right)^2 \approx 9225 \text{ lb/ft}^2 > q_t$$

Hence,

$$q_u = \mathbf{9225\ lb/ft^2}$$

End of part 9-b

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Bearing Capacity (bearing load) of Soil (10)

- **Uplift Capacity of the soil**

Uplift Capacity of Foundations

Foundations may be subjected to uplift forces under special circumstances. During the design process for those foundations, it is desirable to provide a sufficient factor of safety against failure by uplift.

The relationships for the ***uplift capacity*** of foundations in ***granular*** and ***cohesive soils***.

Foundations in Granular Soil (c = 0)

Figure U1 shows a shallow continuous foundation that is being subjected to an uplift force. At ultimate load, Q_u the failure surface in soil will be as shown in the figure. The ultimate load can be expressed in the form of a non-dimensional break-out factor, F_q or:

$$F_q = \frac{Q_u}{A\gamma D_f}$$

where A = area of the foundation.

The breakout factor F_q is a function of the soil friction angle φ' and D_f/B.

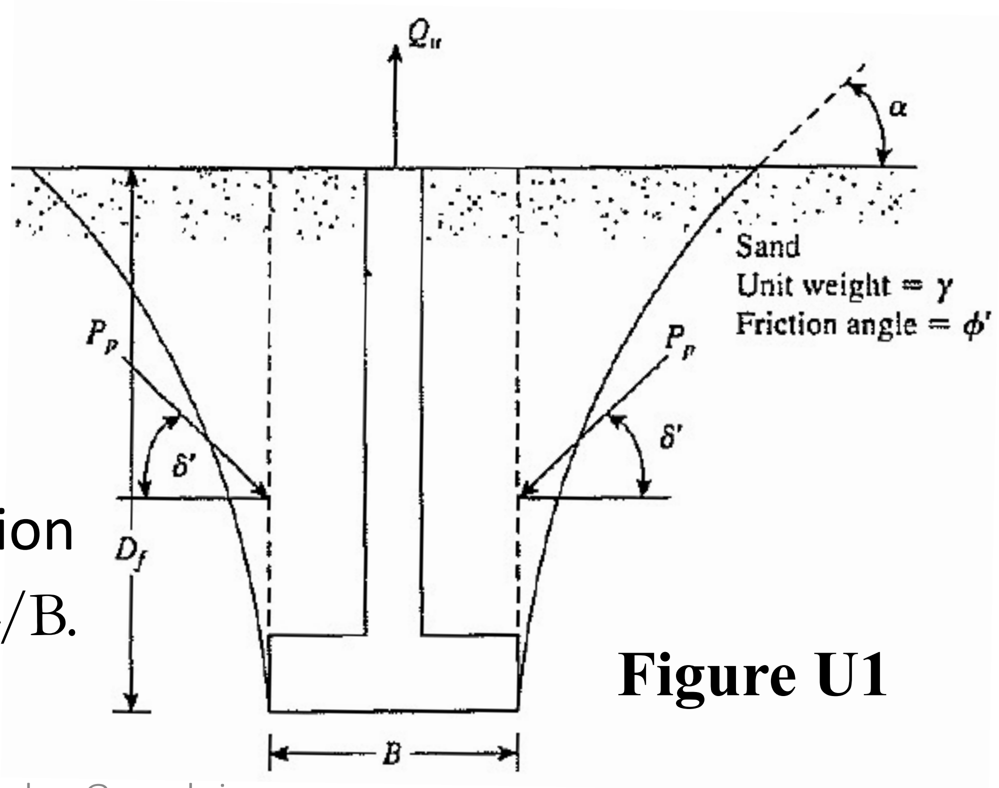

Figure U1

For foundations subjected to uplift:

Shallow foundation : $D_f/B \le (D_f/B)_{cr}$

Deep foundation: $D_f/B > (D_f/B)_{cr}$

The break-out factor can be found by the following two expressions

$$F_q = 1 + 2\left[1 + m\left(\frac{D_f}{B}\right)\right]\left(\frac{D_f}{B}\right)K_u \tan\phi'$$ Eq: U-1

(for shallow circular foundations)

$$F_q = 1 + \left\{\left[1 + 2m\left(\frac{D_f}{B}\right)\right]\left(\frac{B}{L}\right) + 1\right\}\left(\frac{D_f}{B}\right)K_u \tan\phi'$$ Eq: U-2

(for shallow rectangular foundations)

where

m = a coefficient which is a function of ϕ'

K_u = nominal uplift coefficient

For rectangular foundations, Das and Jones (1982) recommended that

$$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}} = \left(\frac{D_f}{B}\right)_{\text{cr-square}}\left[0.133\left(\frac{L}{B}\right) + 0.867\right] \leq 1.4\left(\frac{D_f}{B}\right)_{\text{cr-square}}$$

Eq: U-3

Table: U-1: Variation of *Ku, m* and (Df/B)

Soil friction angle φ' (deg)	Ku	m	(Df/B)cr for square and circular foundations
20	0.856	0.05	2.5
25	0.888	0.10	3.0
30	0.920	0.15	4.0
35	0.936	0.25	5.0
40	0.960	0.35	7.0
45	0.960	0.50	9.0

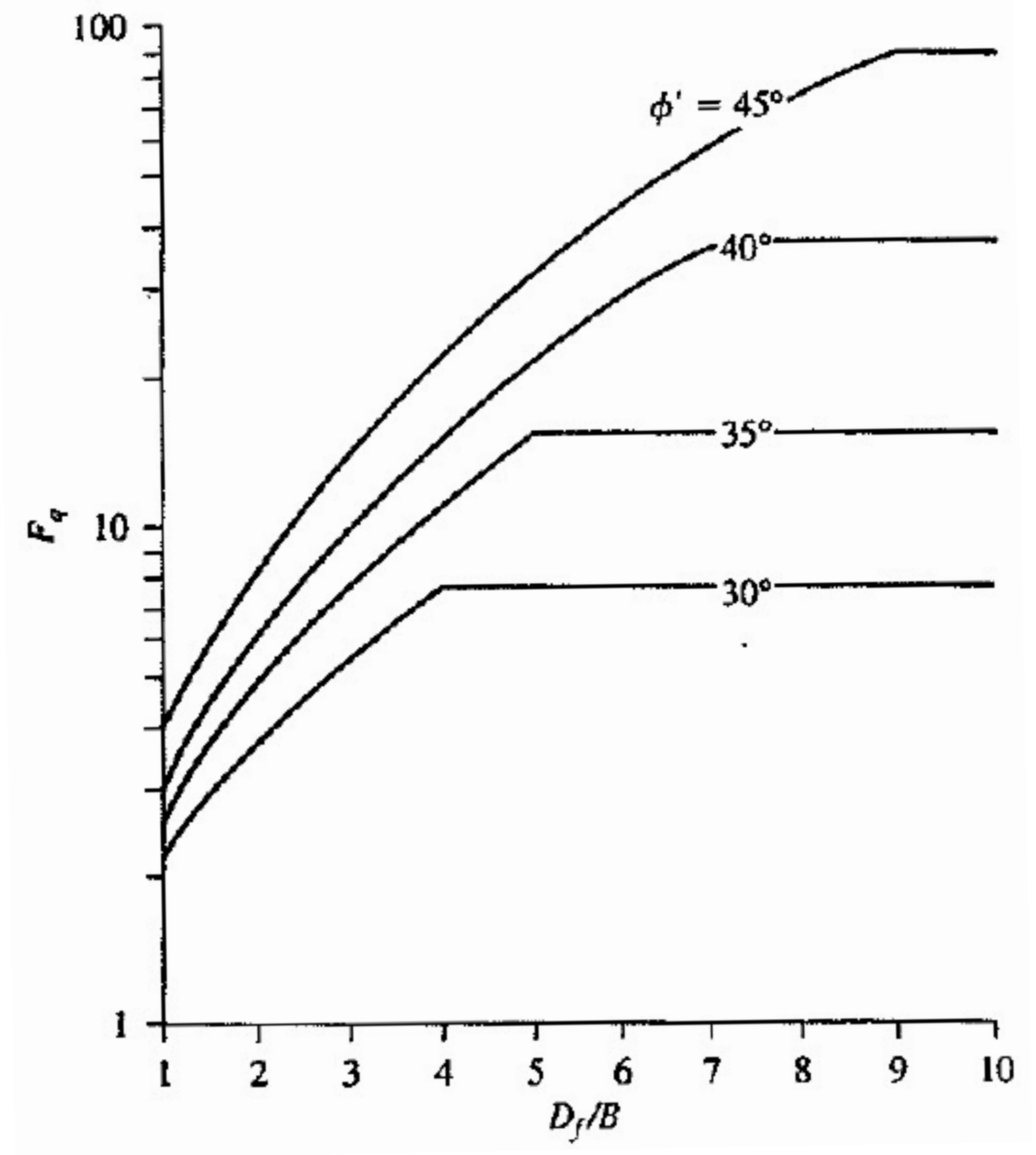

Figure U2: variation of F_q with D_f/B

A ***step-by-step*** *procedure to estimate the uplift capacity of foundations in granular soil follows.*

Step 1: Determine, D_f *,B, L,* and ϕ'

Step 2: Calculate D_f/B.

Step 3: Using Table U1 and Eq. (U3), *calculate* $(D_f/B)_{cr}$

Step 4: If D_f/B is less than or equal *to* $(D_f/B)cr$ it is a shallow foundation.

Step 5: If $D_f/B > (D_f/B)_{cr}$ it is a deep foundation.

Step 6: For shallow foundations, use D_f/B calculated in Step 2 in Eq. (U1) or (U2) to estimate F_q. Thus, $Q_u = F_q A \gamma D_f$.

Step 7: For deep foundations, substitute $(D_f/B)_{cr}$ *for* D_f/B in Eq. (U1) or (U2) *to* obtain F_q from which the ultimate load Q_u may be obtained.

Foundations in Cohesive Soil ($\phi = 0$)

The ultimate uplift capacity, Qu of a foundation in a purely cohesive soil can be expressed as:

$$Qu = A\,(\gamma D_f + Cu\,Fc) \qquad eq{:}\ (U4)$$

where A = area of the foundation

Cu: undrained shear strength of clay

Fc: Break-out Factor

As in the case of foundations in granular soil, the breakout factor Fc increases with embedment ratio and reaches a maximum value of $Fc = Fc^*$ *at* $D_f/B = (D_f/B)cr$ *and remains constant thereafter.*

Das (1978) also reported some model test results with square and rectangular foundations. Based on these test results, it was proposed that:

$$\left(\frac{D_f}{B}\right)_{\text{cr-square}} = 0.107c_u + 2.5 \leq 7 \qquad \text{Eq: U5}$$

where

$\left(\frac{D_f}{B}\right)_{\text{cr-square}}$ = critical embedment ratio of square (or circular) foundations

c_u = undrained cohesion, in kN/m^2

It was also observed by Das (1980) that

$$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}} = \left(\frac{D_f}{B}\right)_{\text{cr-square}}\left[0.73 + 0.27\left(\frac{L}{B}\right)\right] \leq 1.55\left(\frac{D_f}{B}\right)_{\text{cr-square}} \qquad \text{Eq: U6}$$

where

$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}}$ = critical embedment ratio of rectangular foundations

L = length of foundation

Das' Empirical Method for Qu for Cohesive Soil

$$\alpha' = \frac{\frac{D_f}{B}}{\left(\frac{D_f}{B}\right)_{cr}}$$ Eq: U7

$$\beta' = \frac{F_c}{F_c^*}$$ Eq: U8

α' and β' are two nondimensional factors

$$F^*_{c\text{-rectangular}} = 7.56 + 1.44\left(\frac{B}{L}\right)$$ Eq: U9

where $F^*_{c\text{-rectangular}}$ = breakout factor for deep rectangular foundations

A <u>step-by-step</u> procedure to estimate the uplift capacity of foundations in cohesive soil follows.

Step 1: Determine the representative value of the undrained cohesion, c_u

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Step 4: If $D_f/B > (D_f/B)_{cr}$ as determined in Step 2, it is a deep foundation. However, if $D_f/B \leq (D_f/B)_{cr}$ it is a shallow foundation.

Step 5: For $D_f/B > (D_f/B)_{cr}$

$$F_c = F_c^* = 7.56 + 1.44\left(\frac{B}{L}\right)$$

Thus,

$$Q_u = A\left\{\left[7.56 + 1.44\left(\frac{B}{L}\right)\right]c_u + \gamma D_f\right\}$$

Eq: U9

where A = area of the foundation.

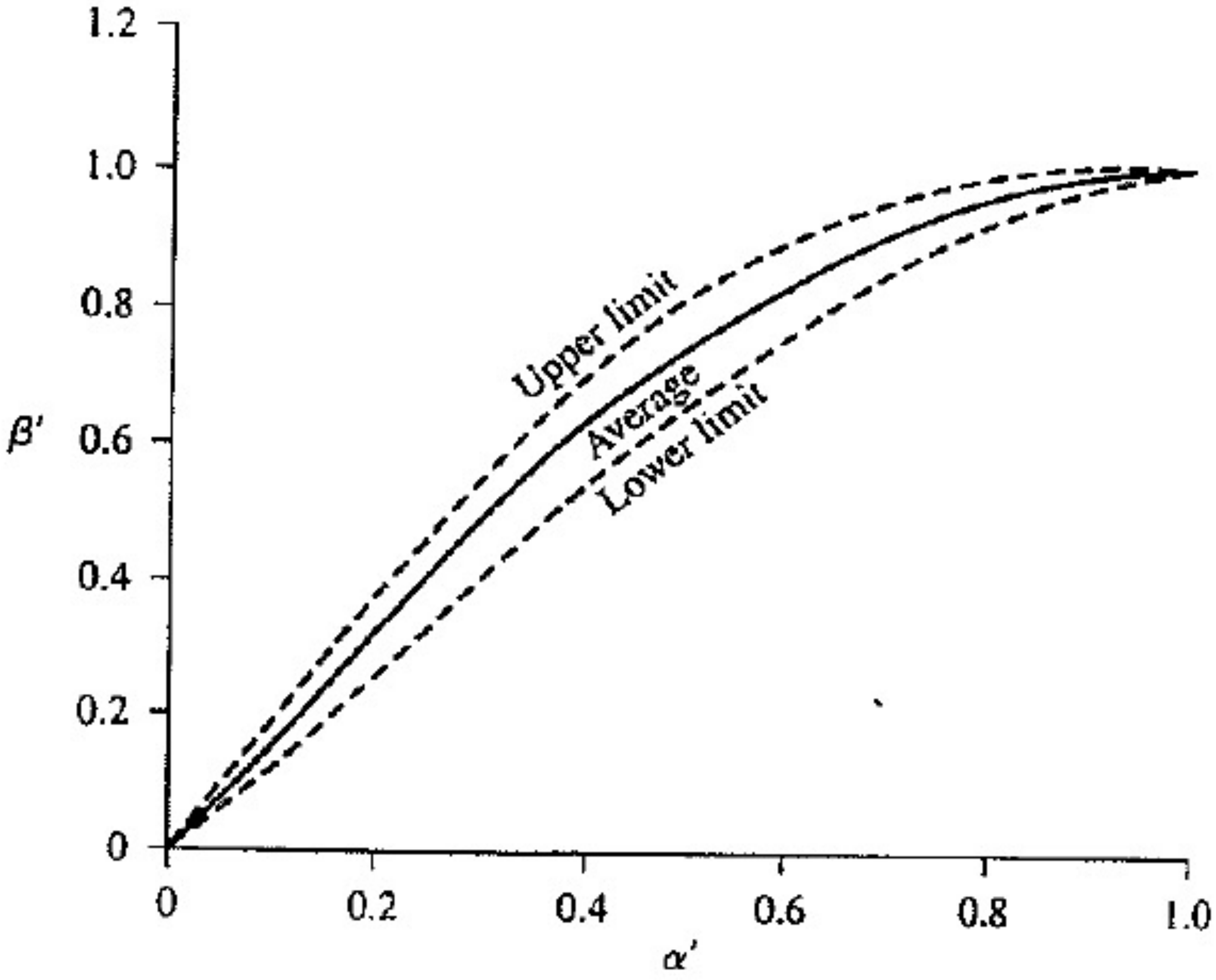

Fig U-3

Plot of β' versus α'

Step 6: for $D_f/B \leq (D_f/B)_{cr}$

$$Q_u = A(\beta' F_c^{*} c_u + \gamma D_f) = A\left\{\beta'\left[7.56 + 1.44\left(\frac{B}{L}\right)\right]c_u + \gamma D_f\right\}$$ Eq: U10

The value of β' can be obtained from the average curve of Fig U-3

Example-1-

Consider a circular foundation in sand. Given for the foundation: **diameter,** $B = 1.5$ m and depth of embedment, $D_f = 1.5$ m. Given for the sand: **unit weight,** $\gamma = 17.4\ \mathrm{kN/m^3}$, and friction angle, $\phi' = 35°$. Calculate the ultimate bearing capacity. against uplift

Solution

$D_f/B = 1.5/1.5 = 1$ and $\phi' = 35°$. For circular foundation, $(D_f/B)_{cr} = 5$. Hence, it is a shallow foundation. From Eq: U-1

$$F_q = 1 + 2\left[1 + m\left(\frac{D_f}{B}\right)\right]\left(\frac{D_f}{B}\right)K_u \tan\phi'$$

For $\phi' = 35°$, $m = 0.25$, and $K_u = 0.9$ table U-1 So

$$F_q = 1 + 2[1 + (0.25)(1)](1)(0.936)(\tan 35) = 2.638$$

So

$$Q_u = F_q \gamma A D_f = (2.638)(17.4)\left[\left(\frac{\pi}{4}\right)(1.5)^2\right](1.5) = \mathbf{121.7\ kN}$$ ■

Example 4.8

A rectangular foundation in a saturated clay measures 1.5 m × 3 m. Given: $D_f = 1.8$ m, $c_u = 52$ kN/m^2, and $\gamma = 18.9$ kN/m^3. Estimate the ultimate uplift capacity.

Solution

From Eq: U5

$$\left(\frac{D_f}{B}\right)_{\text{cr-square}} = 0.107c_u + 2.5 = (0.107)(52) + 2.5 = 8.06$$

So use $(D_f/B)_{\text{cr-square}} = 7$. Again from **Eq-U6-**

$$\left(\frac{D_f}{B}\right)_{\text{cr-rectangular}} = \left(\frac{D_f}{B}\right)_{\text{cr-square}}\left[0.73 + 0.27\left(\frac{L}{B}\right)\right]$$

$$= 7\left[0.73 + 0.27\left(\frac{3}{1.5}\right)\right] = 8.89$$

Check: $$1.55\left(\frac{D_f}{B}\right)_{\text{cr-square}} = (1.55)(7) = 10.85$$

So use $(D_f/B)_{\text{cr-rectangular}} = 8.89$. The actual embedment ratio is $D_f/B = 1.8/1.5 = 1.2$.

Hence, this is a shallow foundation.

$$\alpha' = \frac{\frac{D_f}{B}}{\left(\frac{D_f}{B}\right)_{\text{cr}}} = \frac{1.2}{8.89} = 0.135$$

Referring to the average curve of Fig U-3 for $\alpha' = 0.135$, the magnitude of $\beta' = 0.2$. From Eq: U10

$$Q_u = A\left\{\beta'\left[7.56 + 1.44\left(\frac{B}{L}\right)\right]c_u + \gamma D_f\right\}$$

$$= (1.5)(3)\left\{(0.2)\left[7.56 + 1.44\left(\frac{1.5}{3}\right)\right](52) + (18.9)(1.8)\right\} = \mathbf{540.6\ kN}$$ ■

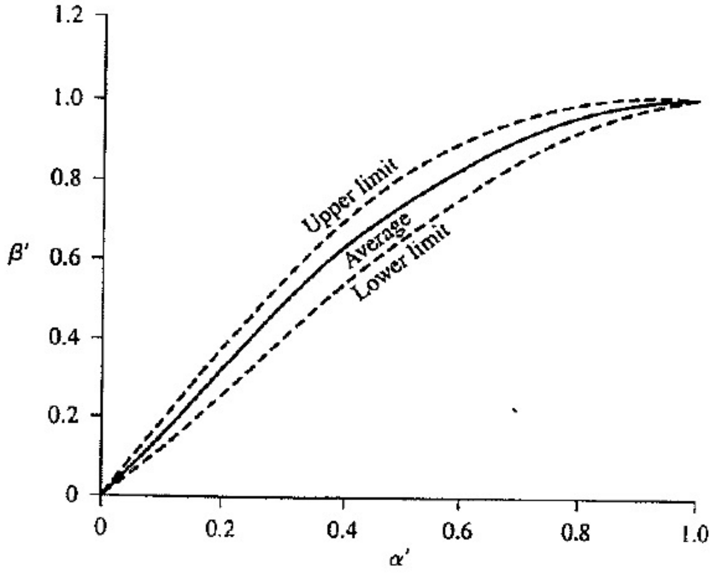

Fig U-3

Thank You

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

Instructor: Professor Dr. Hussein M. Ashour Al.Khuzaie (Ph.D., Civil Engineering, Foundation Engineering and Structures);

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A History Case of Structure Failure due to Shear

- Transcona Grain Elevator Silo in Canada, year was 1913

The tilted Transcona grain elevator (Courtesy: UMA Engineering Ltd., Manitoba, Canada)

https://youtu.be/Isk4kcFR1I4

One of the best known foundation failures occurred in October 1913 at North Transcona, Manitoba, Canada. It was ascertained later on that the failure occurred when the foundation pressure at the base was about equal to the calculated ultimate bearing capacity of an underlaying layer of plastic clay (Peck and Byrant,1953), and was essentially a shearing failure.

As, this event was printed in Press.

Manitoba Fre[e Press]

WINNIPEG, MONDAY, OCTOBER 20, 1913.

BIG ELEVATOR ANNEX SWAYS UNDER STRAIN

Foundation Gives Way Under C. P. R. Grain Warehouse At North Transcona

BUILDING IS IN RUINS

Contained Almost Million Bushels of Wheat---Machinery Is Wrecked

The annex of the huge C.P.R. internal terminal elevator at Transcona sank steadily for 12 hours on Saturday until it had subsided into the ground for 20 feet, at the same time tilting over to an angle of between 30 and 35 degrees. At midnight, when it was feared that the whole structure must collapse, owing to the tremendous strain placed on the concrete of which it was constructed, by the angle at which the building had canted to, the heavy cupola on top collapsed and, by relieving the strain on the foundation, brought the slipping to a stop. The huge storage bins in the annex were filled almost to their capacity of a million bushels of wheat, and, when the cupola fell, it destroyed the whole of the complicated apparatus for removing the grain from the bins to the work-house, so that the C.P.R. is now confronted with the problem of getting almost a million bushels of wheat out of a building tilted to an angle at which it appears as though it must collapse every moment, in time to have is shipped to the great lakes before the

AIR LINE TO WINNIPEG

Aviator Intends Flying from Minneapolis in New Machine.

HAD SUCCESSFUL FLIGHT.

Young Man Wishes to Prove to His Parents that His Biplane Is All Right.

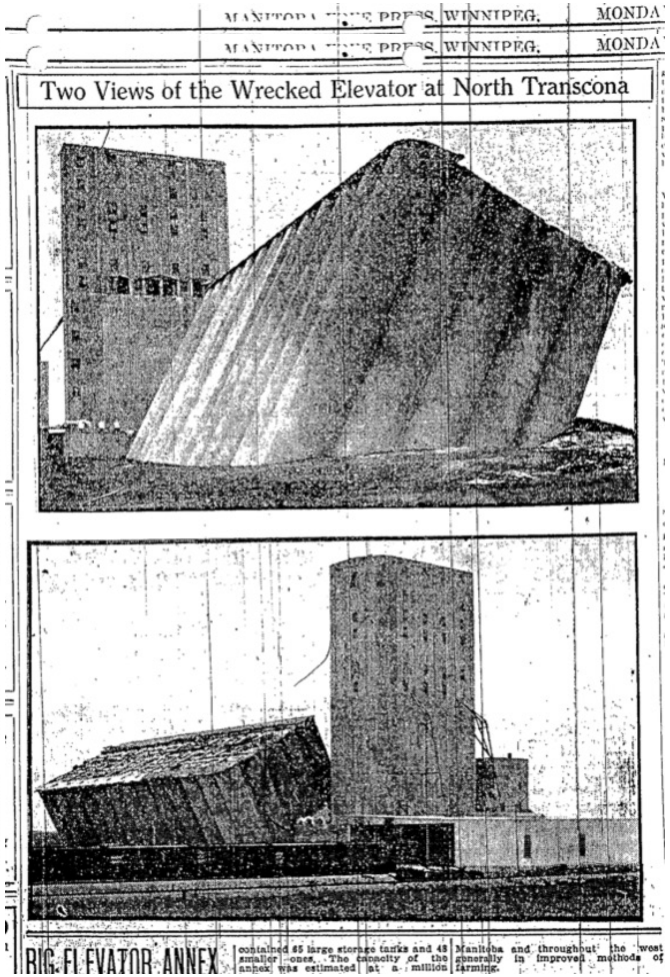
TWENTY SOLDIERS KILLED IN WRECK

Troops Train Plunges Through Trestle Near State Line, Miss. —One Hundred Injured.

SIDENT HUERTA EMAINS AT HELM

Not Resign Nor Run Away, Declares—Has Plenty of ...ey For His Projects.

CANADIAN ... TROUBLE



Two Views of the Wrecked Elevator at North Transcona

BIG ELEVATOR ANNEX

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

Paper No. 1363

THE FAILURE AND RIGHTING OF A MILLION-BUSHEL GRAIN ELEVATOR*

By Alexander Allaire, M. Am. Soc. C. E.

With Discussion by Messrs. David Gutman, W. R. Phillips, and E. P. Goodrich.

Synopsis.

The object of this paper is to present to the attention of the members of the Society the history of a rather unusual engineering feat. The size of the building to be straightened, the angle to which it had tipped, and the weight to be handled, all combined to make the work unique.

The restoration of the elevator to a working condition may be divided into four parts:

1.—Making safe the foundations under the workhouse—a structure resembling a tall office building, resting on a very small base;

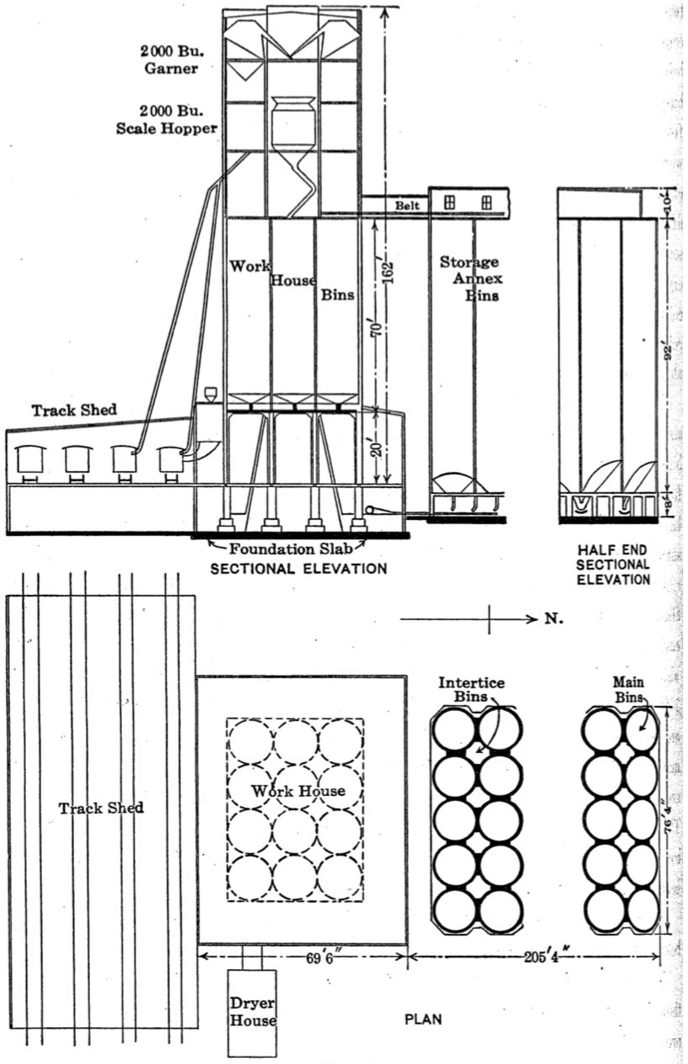
2.—Straightening the binhouse, a structure having an area of 15 000 sq. ft.;

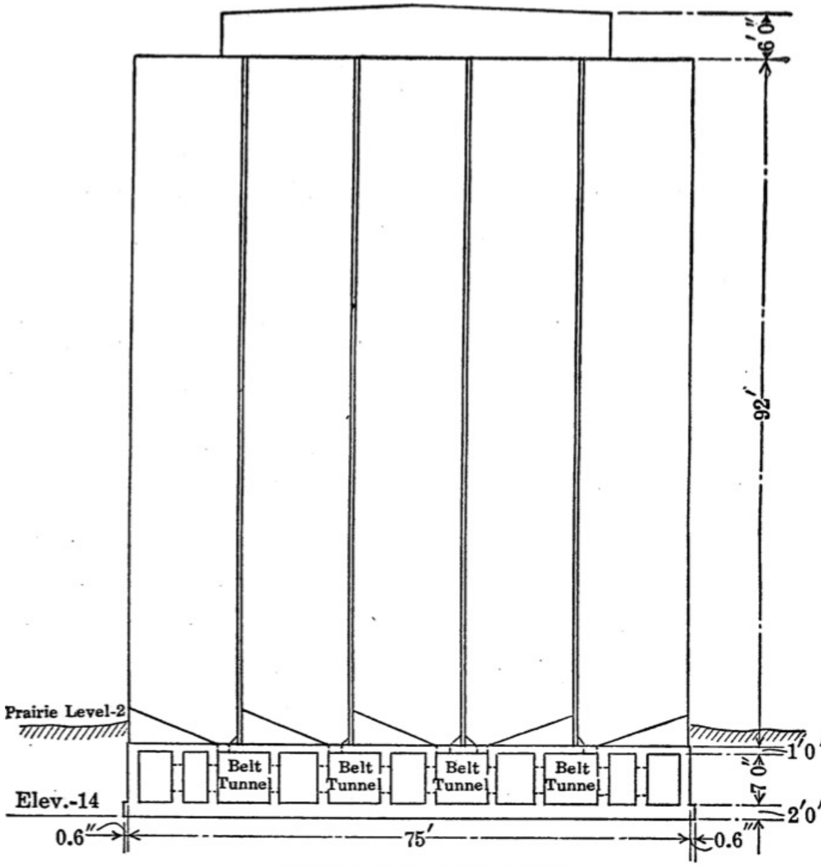
3.—Providing this binhouse with a new and adequate foundation; and

4.—The renewal and repair of those portions of the original buildings which had been broken or deranged at the time of the failure.

This paper treats of the first three.

* Presented at the meeting of February 2d, 1916.





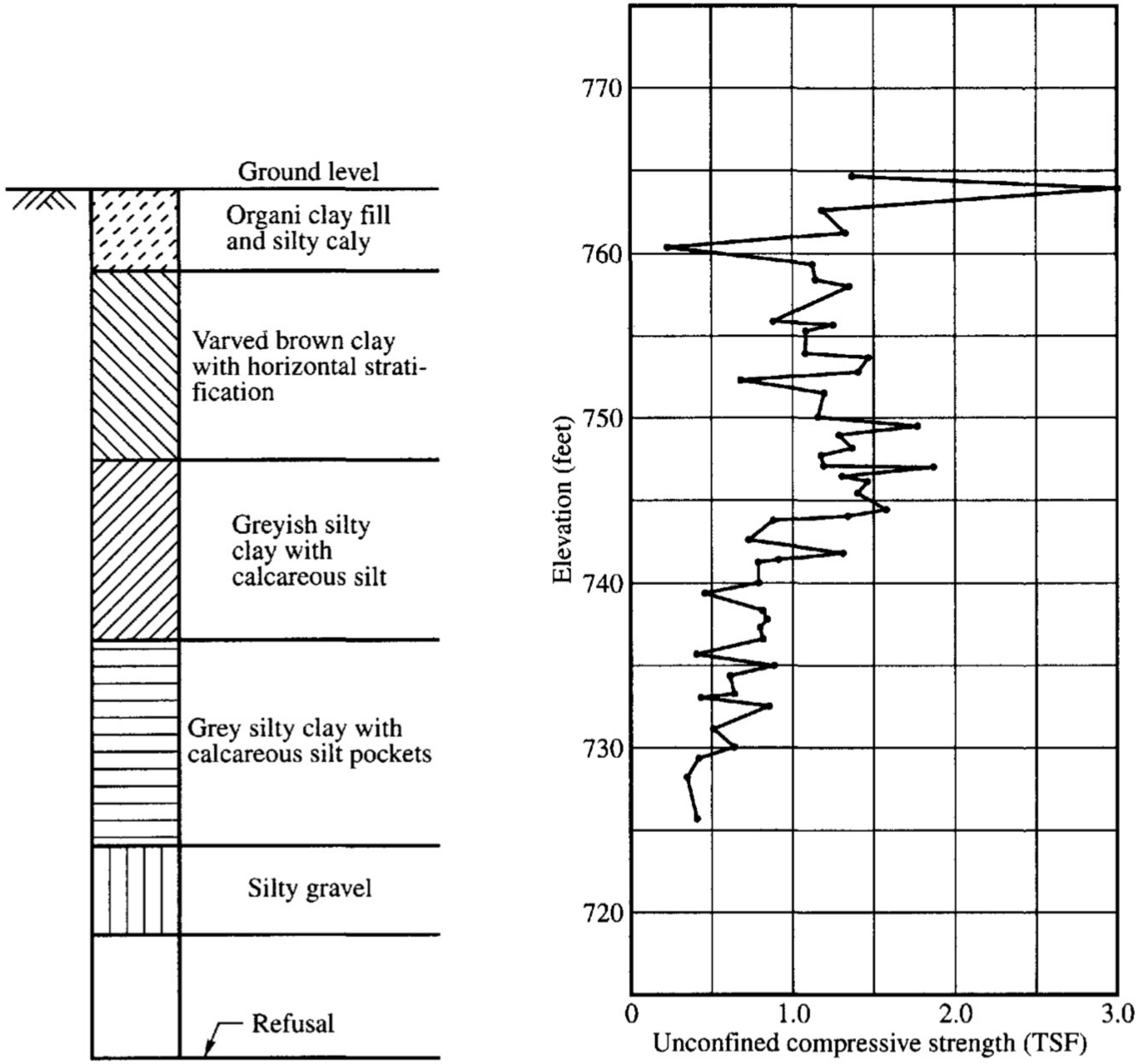
ORIGINAL POSITION

SECTION THROUGH BINS—LOOKING SOUTH

History background and general review of the structure:

The construction of the silo started in 1911 and was completed in the autumn of 1913. The silo is 77 ft by 195 ft in plan and has a capacity of 1,000,000 ***bushels***. It comprises 65 circular bins and 48 inter-bins. The foundation was a reinforced concrete raft 2 ft thick and founded at a depth of 12 ft below the ground surface. The weight of the silo was 20,000 tons, which was 42.5 percent of the total weight, when it was filled. Filling the silo with grain started in September 1913, and in October when the silo contained 875,000 bushels, and the pressure on the ground was 94 percent of the design pressure, a vertical settlement of 1 ft was noticed. The structure began to tilt to the west and within twenty four hours was at an angle of 26.9° from the vertical, the west side being 24 ft below and the east side 5 ft above the original level (Szechy, 1961). The structure tilted as a monolith and there was no damage to the structure except for a few superficial cracks. Figure 12.22 shows a view of the tilted structure. The excellent quality of the reinforced concrete structure is shown by the fact that later it was underpinned and jacked up on new piers founded on rock. The level of the new foundation is 34 ft below the ground surface.

Bushel: **BRITISH**: measure of capacity equal to 8 gallons (equivalent to **36.4 liters**), used for corn, fruit, liquids, etc.
US: measure of capacity equal to 64 US pints (equivalent to **35.2 liters**), used for dry goods.



(a) Classification from test boring

(b) Variation of unconfined compressive strength with depth

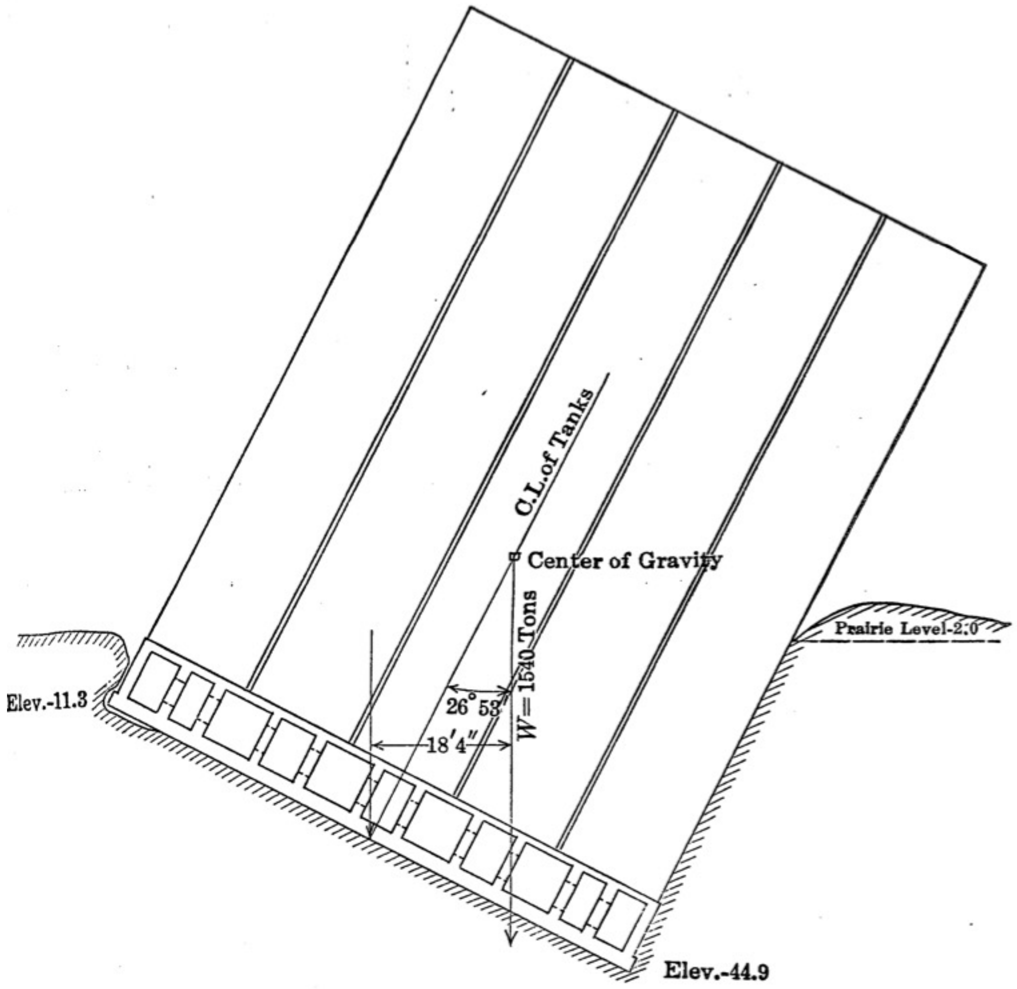
Results of test boring at site of Transcona grain elevator (Peck and Byrant, 1953)

The **contact pressure** due to the load from the silo at the time of failure was estimated as equal to **3.06** tsf. The theoretical values of the ultimate bearing capacity by various methods are as follows:

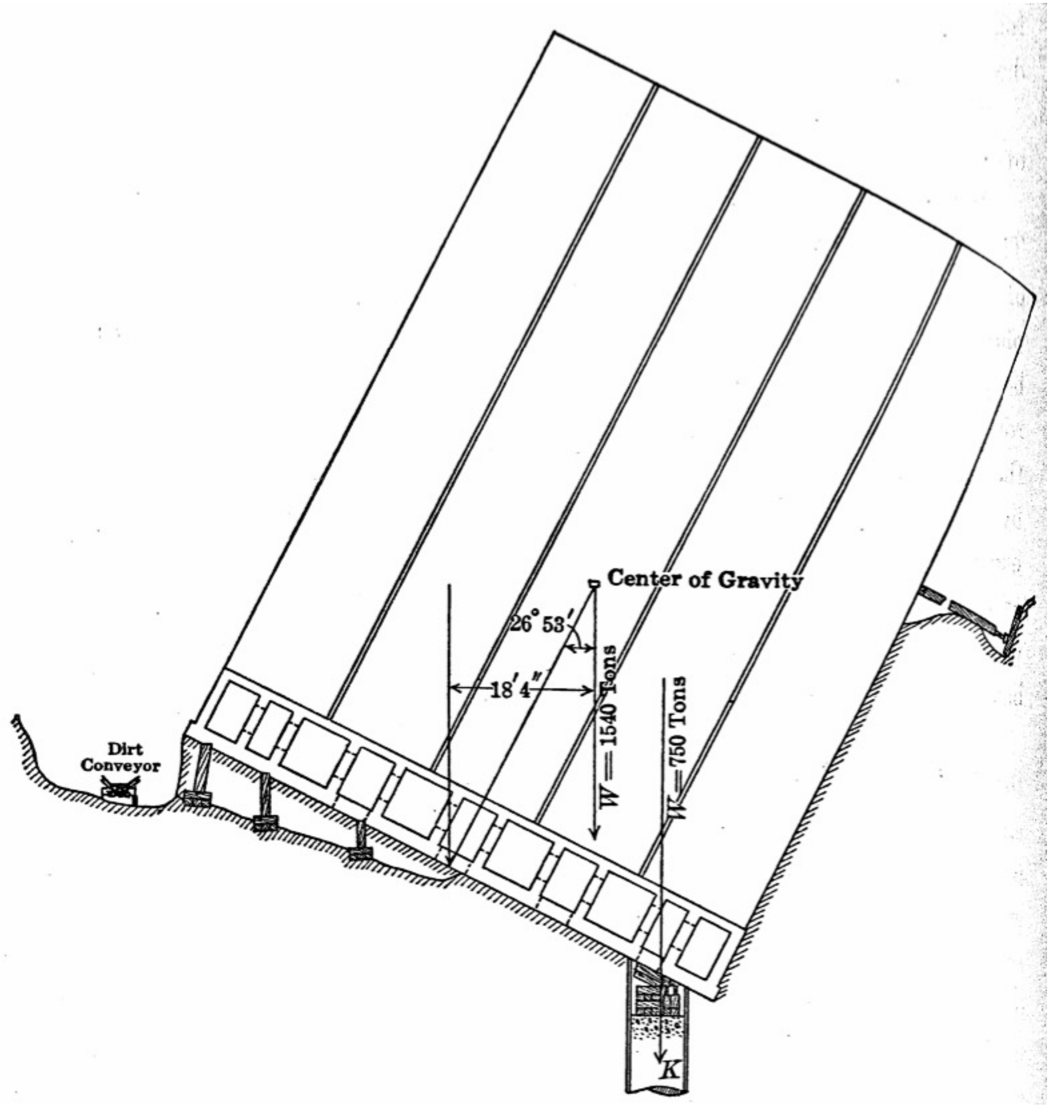
Methods	q_u **tsf**
Terzaghi	3.68
Meyerhof	3.30
Skempton	3.32

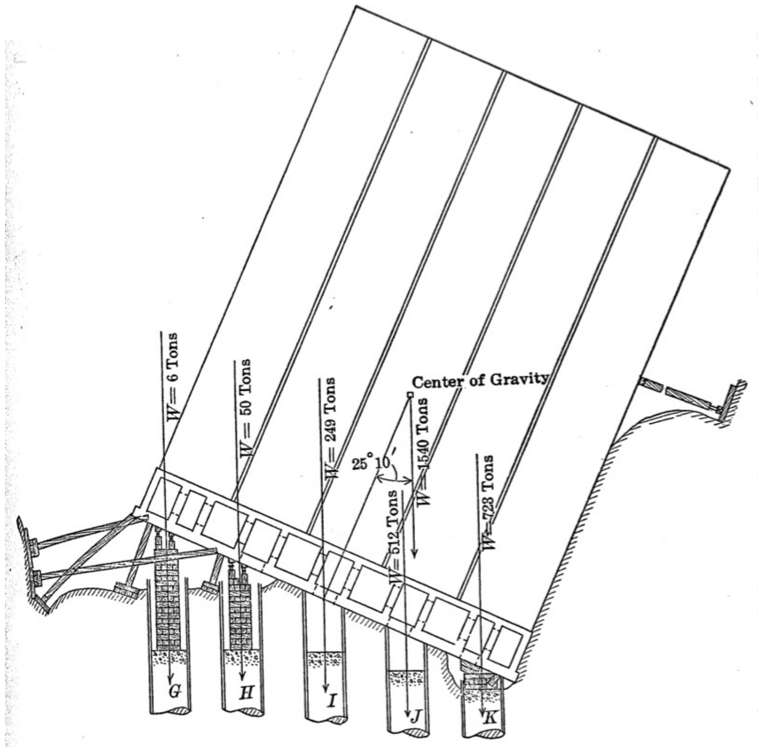
The above values compare reasonably well with the actual failure load 3.06 tsf.

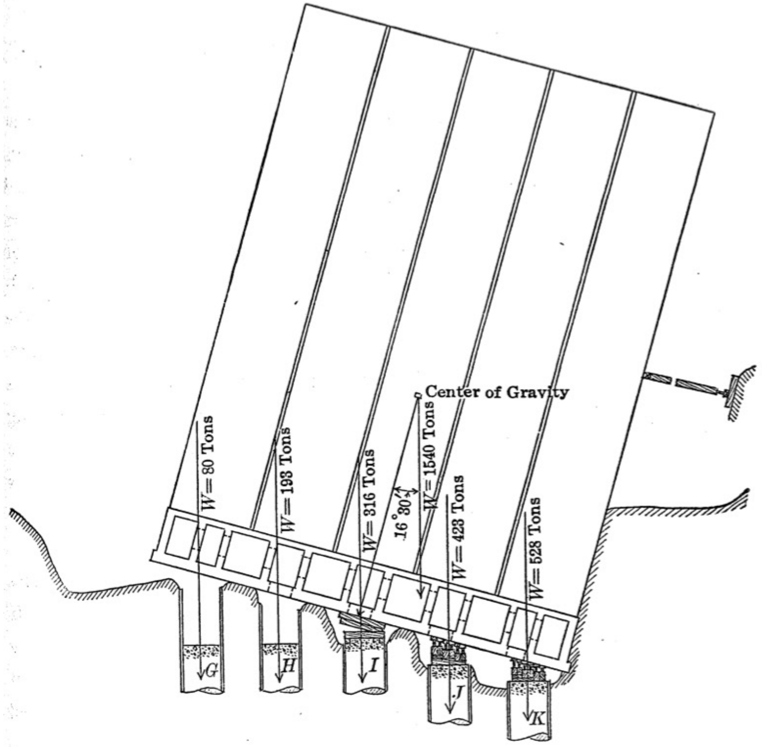
Process of Restoration



SECTION THROUGH BINS AFTER SETTLEMENT




W = 6 Tons
W = 50 Tons
W = 249 Tons
Center of Gravity
25° 10′
W = 1540 Tons
W = 512 Tons
W = 723 Tons
G
H
I
J
K


W = 80 Tons
W = 193 Tons
W = 316 Tons
Center of Gravity
16° 30′
W = 1540 Tons
W = 423 Tons
W = 528 Tons
G
H
I
J
K

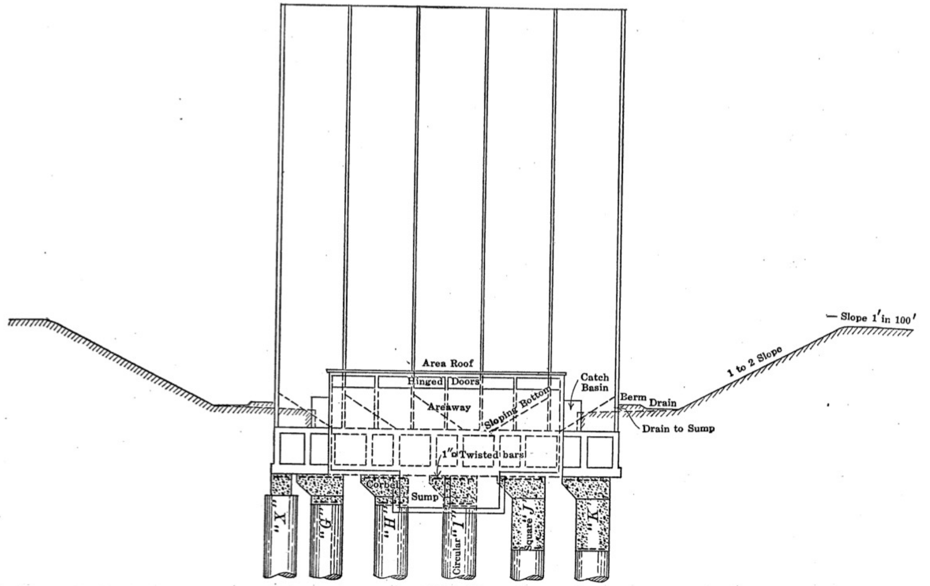

Slope 1′ in 100′
1 to 2 Slope
Area Roof
Hinged Doors
Areaway
Sloping Bottom
Catch Basin
Berm
Drain
Drain to Sump
1″ ⌀ Twisted bars
Corbel
Sump
"X"
"G"
"H"
Circular "I"
Square "J"
"Y"

FIG. 20.


NORTH TRANSCONA ELEVATOR
N.W. CORNER OF BINS
AFTER STRAIGHTENING

Thank You

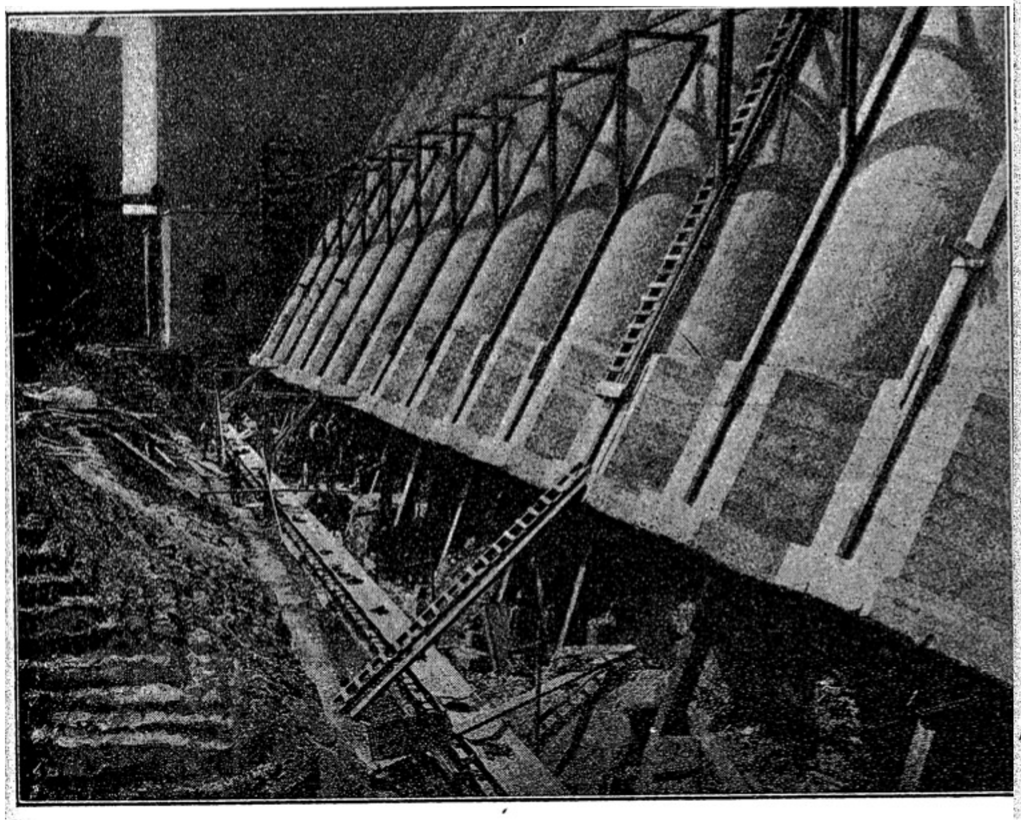


Fig. 10.—Excavation Toward West from Under East Edge of Mat.

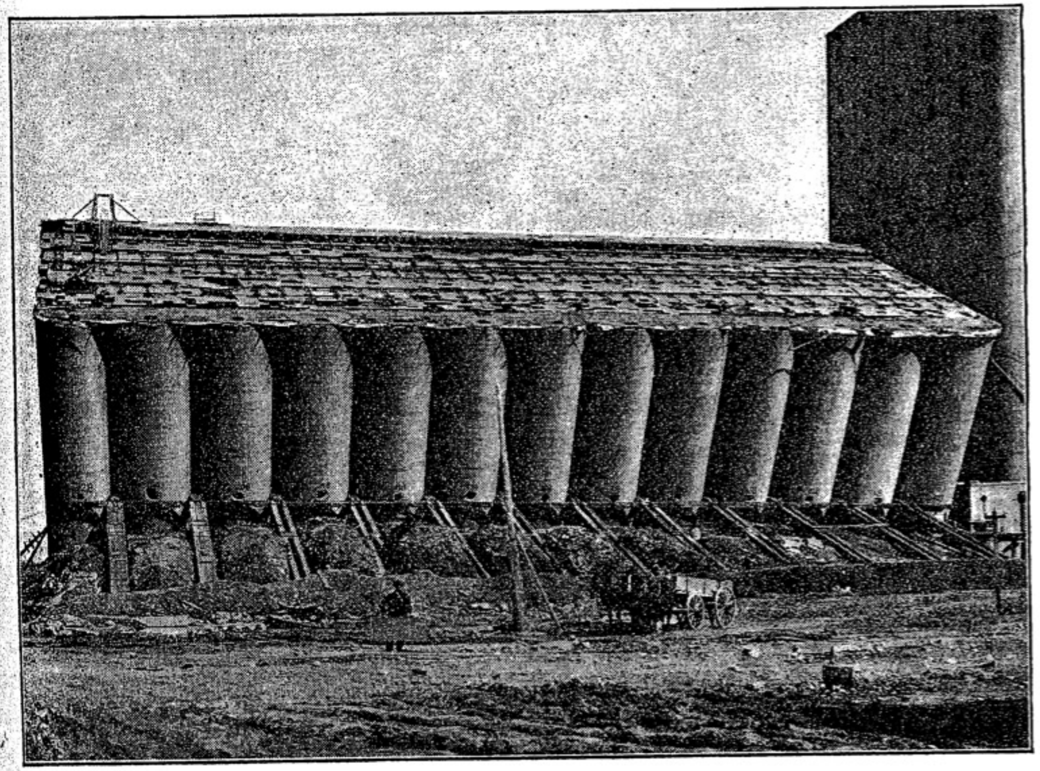


Fig. 11.—Line of Pushers Against West Side of Bin Structure, Showing Holes through which Grain was Taken Out.

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

Instructor: Professor Dr. Hussein M. Al.Khuzaie (Ph.D, Civil Engineering, Foundation Engineering and Structures);

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- ✓ Introduction to the first course
- ✓ Syllabus
- ✓ Presentations
- ✓ Assessment
- ✓ Reference
- ✓ Time table

Syllabus and details

1	➢ **Site Investigation (SI): 3 weeks (9 hrs.)** • Definition and aims • Steps • Number and depth of boring • Sampling • Laboratory tests • Field tests • Report
2	➢ **Bearing capacity for shallow foundation: 7 weeks (30 hrs.)** • Introduction • Terzaghi's bearing capacity equation and BC factors • Meyerhof 's equation and shape factors • SPT used for BC • Eccentricity loading (one axes and bi-axes) • BC of non-homogeneous soil • Uplift Capacity
3	➢ **Settlement for saturated soil: 2 weeks (6 hrs.)** • Elastic calculation • Consolidation settlement • Secondary settlement

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❑ Methods of presentation of the course:

1)Power point presentations (Hand-out).
2) Video for explanation each lecture.
3) Photos and videos for more explanations

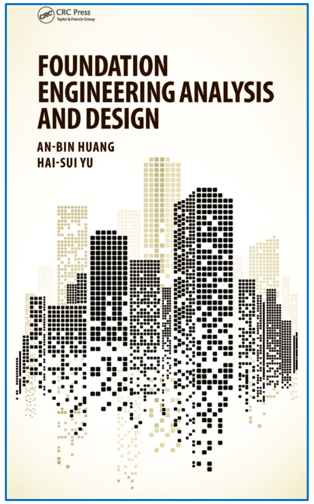
❑ How to get feedback?:

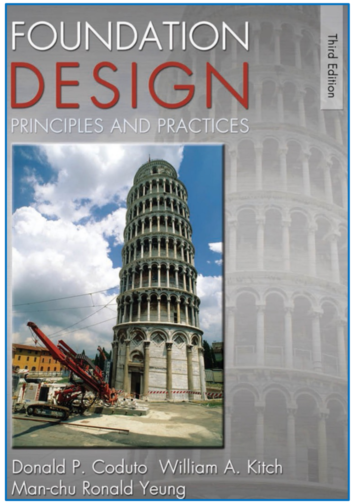
1) Assignments
2) Quizzes
3) Workshop for selective topics done by groups
4) Small projects and reports by small groups (2-3) students.
5) Monthly Exam
6) Final Exam of the course.

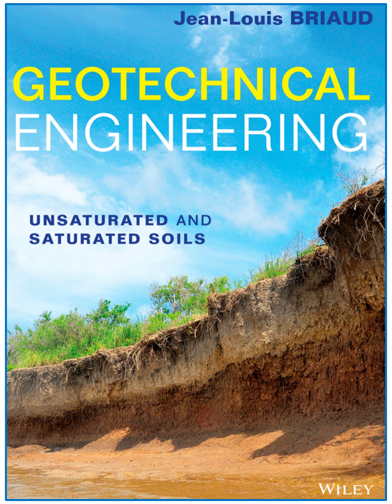
References

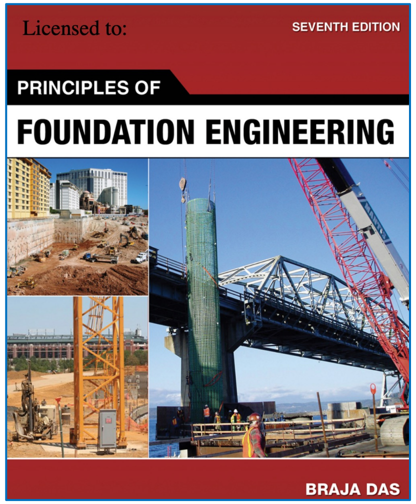
1. *Huang A. B. and Yu H. S.,* *“***Foundation Engineering Analysis and Design***” First Edition, 2018.*
2. *Couto D. P., Kitch W. A., Yeung M. R., “***Foundation design : principles and practices***” Third Edition, 2016***.**
3. *Briaud J. L., “***Geotechnical Engineering: Unsaturated and Saturated Soils***” First Edition, 2013.*
4. *DAS B. M., “***Principles of Foundation Engineering***”, Seventh Edition, 2011.*
5. *DAS B. M., “***Principles of Geotechnical engineering***” Seventh Edition, 2010.*
6. *Bowles J. E. “* **Foundation Analysis and Design***”, Fifth Edition, 2006.*
7. **Videos by YouTube or any other sources relating to the course.**
8. **Websites on the WWW for furnishing more explanations of the themes of this course.**

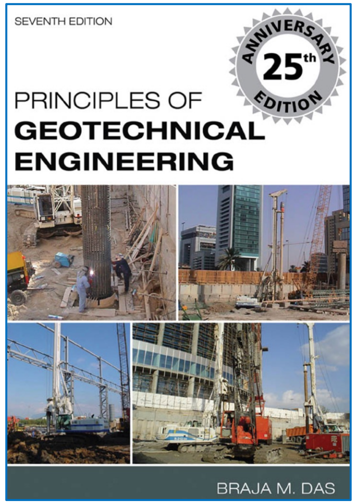
References











-1- -2- -3- -4- -5- -6-

Soil Exploration (Investigation), (SI), Lecture (1)

Outlines

- Introduction and aims of SI
- Design of SI
  - I. Number of boreholes
  - II.Depth of boring (drilling)
- Example on the depth of boring
- Drilling (Boring) methods

SOIL INVESTIGATION (SI), Introduction

- Definition: The process of determining the layers of natural soil deposits that will underlie a proposed structure and their physical, chemical, and mechanical properties.
- The purpose of subsurface exploration is to provide knowledge of the ground conditions for safe and economical foundation design and potential problems that may be encountered during construction.
- A successful subsurface exploration should provide the following information:

1. Stratigraphy of the ground material, soil/rock properties, and groundwater conditions within the area and depth that will be affected by the proposed structure.
2. Geotechnical parameters required for the selection or recommendation of the type and depth of foundations, determination of bearing capacities for the recommended foundation type(s), and estimation of the proposed foundation settlement.
3. Design parameters required for related earth or earth-supporting structures, such as embankments, retaining walls, or braced excavations.
4. Potential problems to be expected for the construction of the recommended foundation system.

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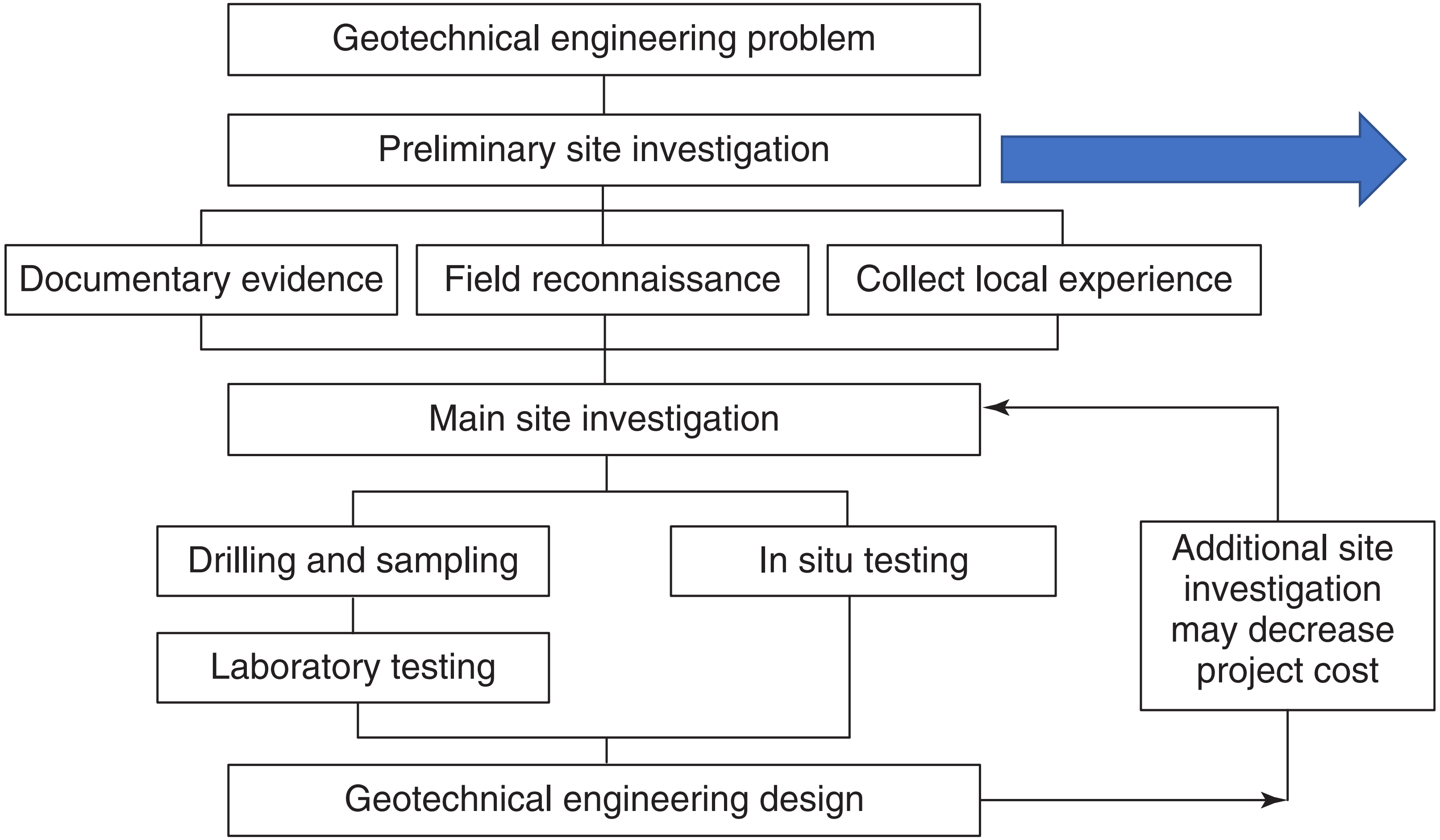
Introduction cont'd

Flowchart for a geotechnical engineering project.



Preliminary site investigation takes place in two steps:

- **Paper study (Desk study):**

The paper study consists of obtaining documents related to the site information and history. In addition to maps, previous records of site uses are very helpful. Maps include geologic maps, aerial photographs, flood maps, and seismicity maps.

- **Site visit (Reconnaissance):**

Going to the site, taking notes and photos of the site conditions, including the behavior of other projects in the vicinity. The site conditions include general topography, rig access, geologic features, stream banks exposing the stratigraphy, land use, water-flow conditions, and possibility of flood. A good site visit requires a keen eye and keeping a detailed record of what is found and observed at the site. In the case of environmentally related problems, special guidelines exist for what is called environmental site assessments (ESAs).

Design of Detailed Site Investigation (SI)

✓ **Number of boreholes:**

- No rule for identifying the number of boreholes that may performed.
- A guide for determination the boreholes' number was proposed by Sowers and Sowers (1970) as in the table depending on the surface horizontal distance between boreholes.

Project	Boring spacings m	Boring spacings ft
One-story buildings	25–30	75–100
Multistory buildings	15–25	50–75
Highways	250–300	750–1000
Earth dams	25–50	75–150
Residential subdivision planning	60–100	200–300

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Design of Detailed Site Investigation (SI), Cont'd

➢ **Depth of boring (drilling) :**

➢ **Approximate determination of depth of drilling**

1) For steel and lightweight concrete building:

$$z_b\,(\mathrm{m}) = 3S^{0.7}$$

$$z_b\,(\mathrm{ft}) = 10S^{0.7}$$

2) For heavy steel and wide concrete building:

$$z_b\,(\mathrm{m}) = 6S^{0.7}$$

$$z_b\,(\mathrm{ft}) = 20S^{0.7}$$

Where: Z_b is the depth of boring and S is the number of stories

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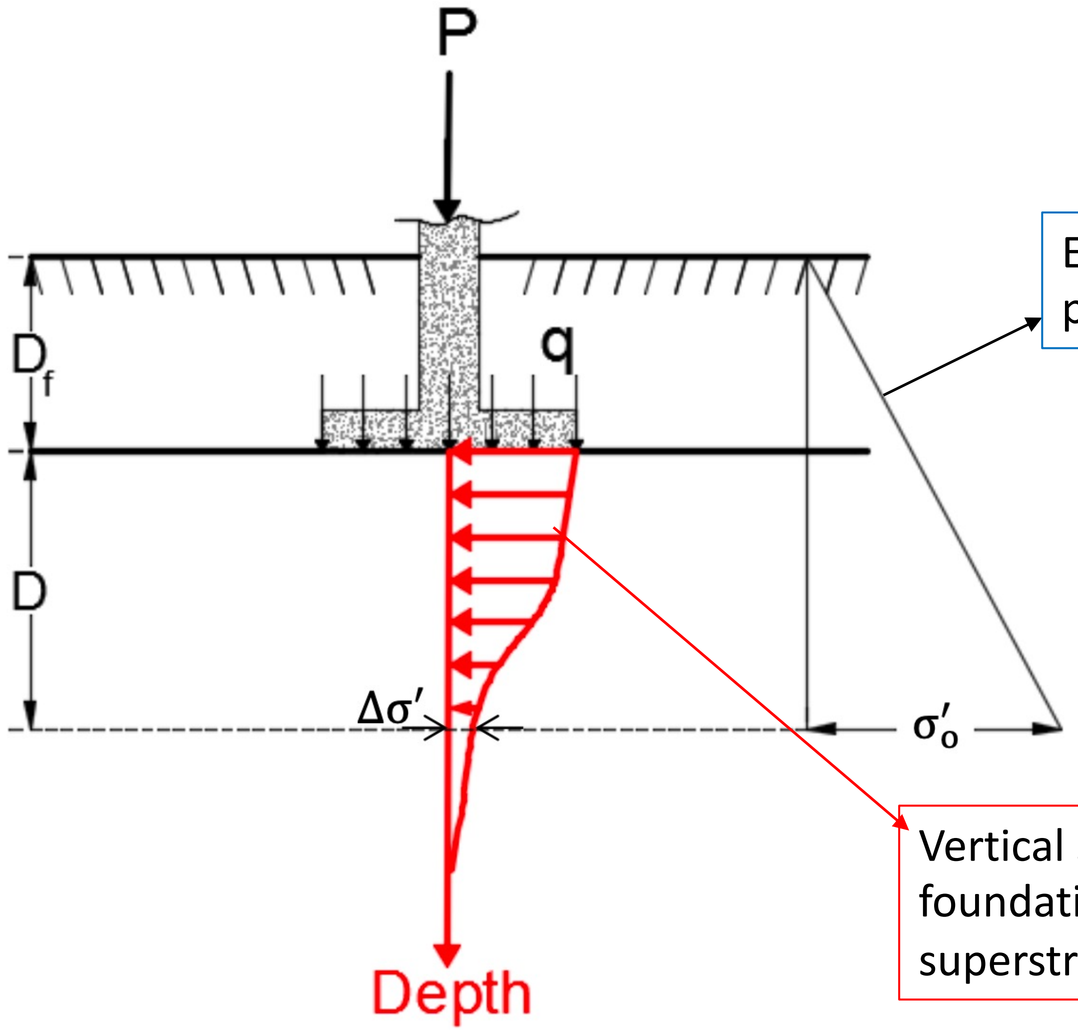
Depth of boring (drilling), Cont'd :

➢ASCE Method

- Determine the net increase in effective stress ($\Delta\sigma'$) under a foundation with depth as shown in the Figure below.

- Estimate the variation of the vertical effective stress (σ'_o) with depth.
- Determine the depth ($D = D_1$) at which the effective stress increase ($\Delta\boldsymbol{\sigma}'$) is equal to ($\mathbf{1/10}$) q (q = estimated net stress on the foundation and at the base).
- Determine the depth ($D = D_2$) at which ($\Delta\boldsymbol{\sigma}'/\boldsymbol{\sigma}'_{\mathbf{o}}$) = $\mathbf{0.05}$.
- Determine the depth ($D = D_3$) which is the distance from the lower face of the foundation to bedrock (if encountered).
- Choose the smaller of the three depths, (D1, D2, and D3), just determined is the approximate required minimum depth of boring.

ASCE Method(1972)



Effective vertical overburden pressure

Vertical stress subjected to the foundation from the superstructures

<u>Depth of boring (drilling), Cont’d :</u>

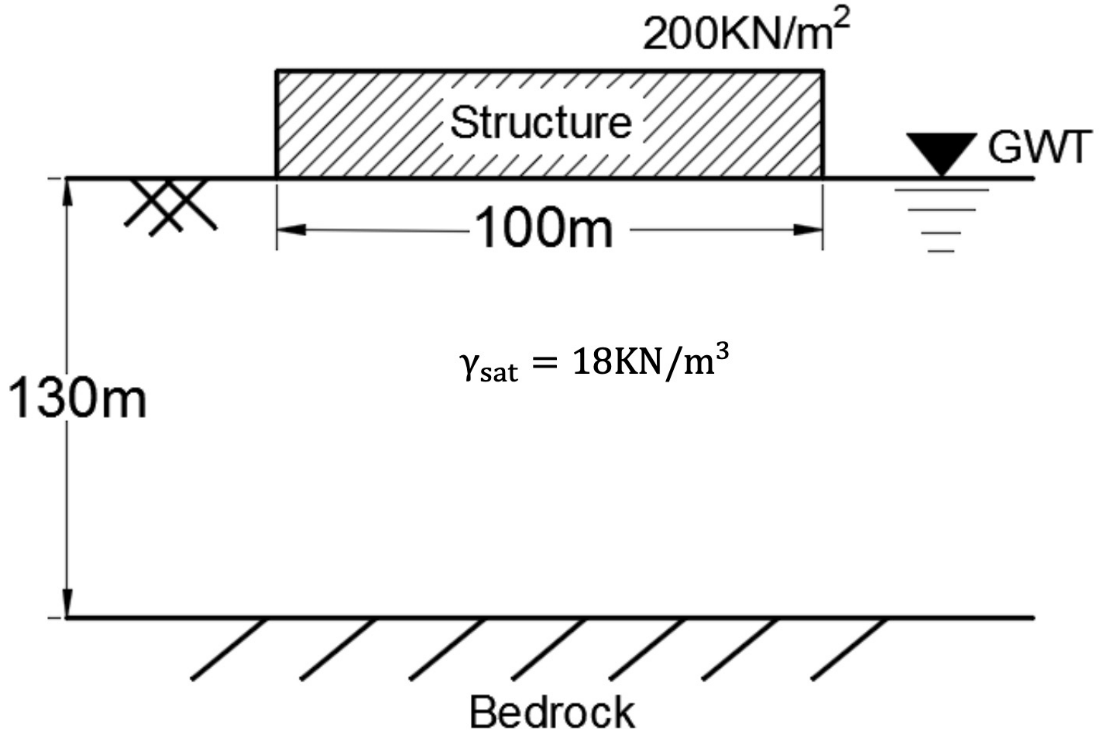
<u>Notes:</u>

- **When the soil exploration is for the construction of dams and embankments, the depth of boring may range from <u>one-half to two times the embankment height.</u>**
- **When deep excavations are anticipated, the depth of boring should be at, least *<u>1.5 times the depth of excavation</u>*.**
- **Sometimes subsoil conditions are such that the foundation load may have to be transmitted to the bedrock. The minimum depth of core boring into the bedrock is about 3m. If the bedrock is irregular or weathered, the core borings may have to be extended to greater depths.**

Depth of boring (drilling), Cont'd :

Example (1): (Reference: Al-Agha A. S. Basics of Foundation Engineering with Solved Problems)

Site investigation is to be made for a structure of 100 m length and 70 m width. The soil profile is shown below, if the structure is subjected to 200 KN/m^2 what is the approximate depth of borehole? (Assume γ_w =10 KN/m^3).



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Depth of boring (drilling), Cont'd :

Solution

Givens:

$q = 200KN/m^2$, structure dimensions $= (70 \times 100)m$

$\rightarrow P = 200 \times (100 \times 70) = 1.4 \times 10^6$ KN.

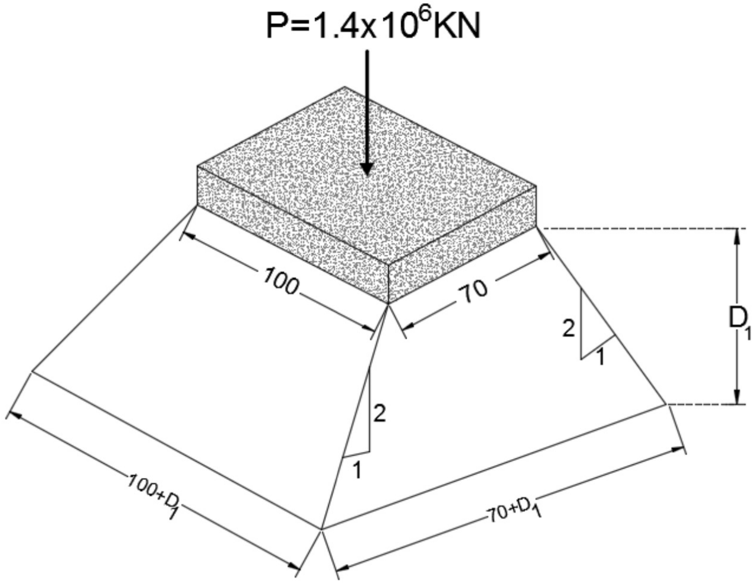
$D_f = 0.0$ (Structure exist on the ground surface) , $\gamma_{sat} = 18KN/m^3$.

$D_3 = 130m$ (distance from the lower face of structure to the bedrock).

1. Calculating the depth (D_1) at which $\Delta\sigma'_{D_1} = \left(\frac{1}{10}\right) \times q$:

$\left(\frac{1}{10}\right) \times q = \left(\frac{1}{10}\right) \times 200 = 20KN/m^2$.

The following figure showing the distribution of stress under the structure at depth (D_1):



The increase in vertical stress ($\Delta\sigma'$) at depth (D_1) is calculated as follows:

$$\Delta\sigma'_{D_1} = \frac{P}{A} = \frac{1.4 \times 10^6}{(100 + D_1) \times (70 + D_1)}$$

$@\ D_1 \rightarrow \Delta\sigma' = \left(\frac{1}{10}\right) \times q \rightarrow \frac{1.4\times10^6}{(100+D_1)\times(70+D_1)} = 20 \rightarrow D_1 = 180$ m.

2. Calculating the depth (D_2) at which $\left(\frac{\Delta\sigma'}{\sigma'_o}\right) = 0.05$:

The effective stress(σ'_o) at depth D_2 is calculated as following:

$\sigma'_{o,D_2} = (\gamma_{sat} - \gamma_w) \times D_2$

$\rightarrow \sigma'_{o,D_2} = (18 - 10) \times D_2 \rightarrow \sigma'_{o,D_2} = 8D_2$.

The increase in vertical stress ($\Delta\sigma'$) at depth (D_2) is calculated as follows:

$$\Delta\sigma'_{D_2} = \frac{P}{A} = \frac{1.4 \times 10^6}{(100 + D_2) \times (70 + D_2)}$$

$@\ D_2 \rightarrow \left(\frac{\Delta\sigma'}{\sigma'_o}\right) = 0.05 \rightarrow \frac{1.4\times10^6}{(100+D_2)\times(70+D_2)} = 0.05 \times (8D_2) \rightarrow D_2 = 101.4$ m

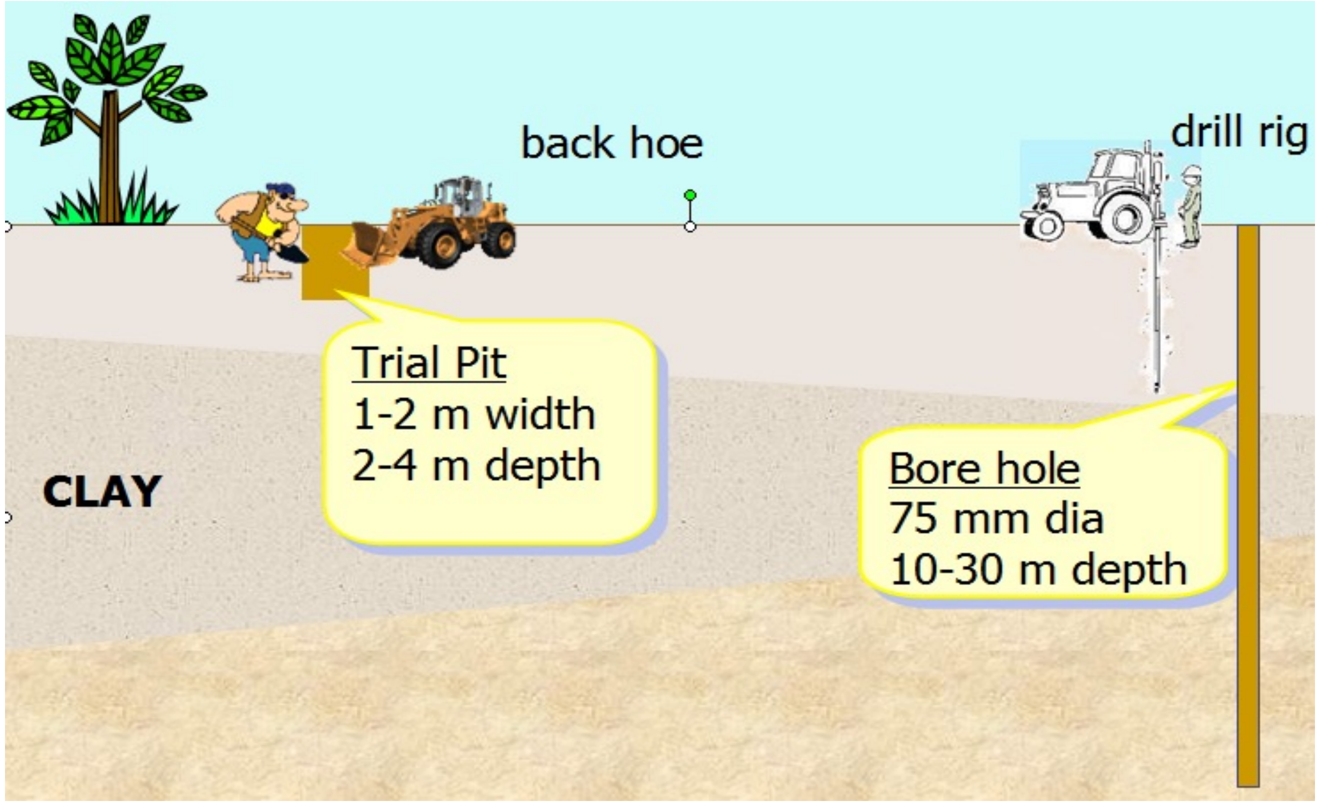
So, the value of (D) is the smallest value of D_1, D_2, and $D_3 \rightarrow D = D_2 = 101.4$ m.

$\rightarrow D_{boring} = D_f + D \rightarrow D_{boring} = 0.0 + 101.4 = 101.4$ m ✓.

Boring Methods

1- Trial Pit, manually or by machine such as shovel or backhoe. (boring depth may reach (2-3) m from the natural ground surface.

2- Drilling borehol, it means to drill a hole in the soil and by this method, the depth of boring may reach 30 m.



Boring Methods, cont'd

The test boring can be advanced in the field by several methods:

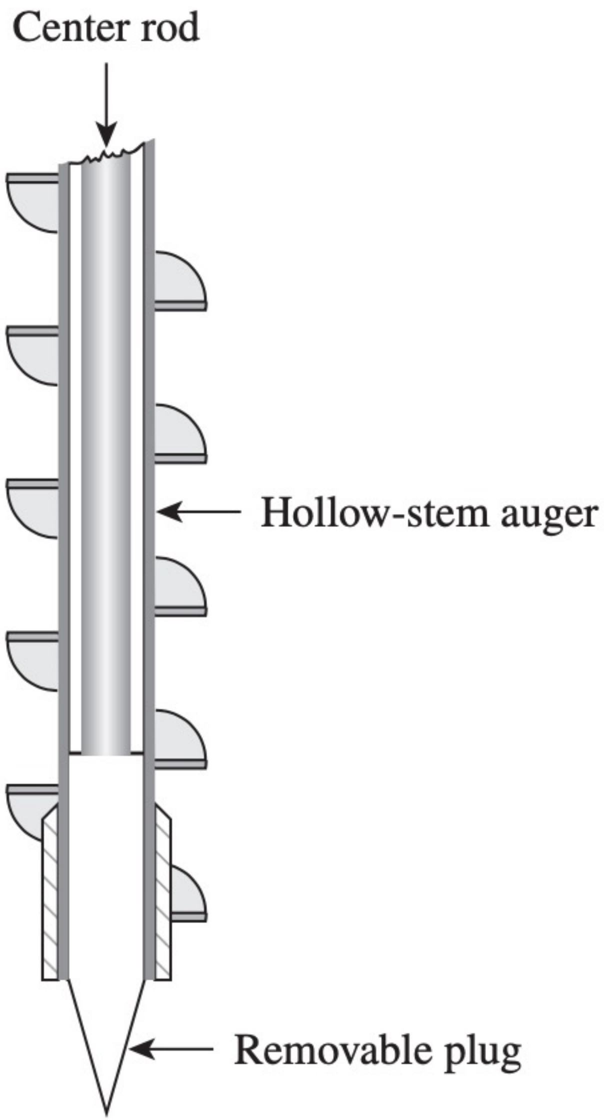
- *Augers* for making boreholes up to a depth of about 3 to 5 m (10 to 15 ft). For highways and small structures. The soil samples are disturbed, but they can be used to conduct laboratory tests such as grain- size determination and Atterberg limits.
- *Continuous-flight augers*, which are power operated. The power for drilling is delivered by truck- or tractor-mounted drilling rigs. Continuous-flight augers are available commercially in 1 to 1.5 m (3 to 5 ft) sections. Two types of Auger:
  - Solid stem auger: The common outside diameter of solid stem auger are: 67 mm, 83 mm 102 mm ,and 114 mm
  - Hollow stem auger: The common dimensions of this type as in the table:

Inside diameter		Outside diameter	
mm	in.	mm	in.
63.5	2.5	158.75	6.25
69.85	2.75	187.8	7.0
76.2	3.0	203.2	8.0
88.9	3.5	228.6	9.0
101.6	4.0	254.0	10.0

Boring Methods, cont'd

Hand Helical Auger

Hollow stem augers



✓Hollow Stem Auger Drilling Method

- The hollow stem auger method, sometimes also called the continuous flight auger method consists of rotating hollow stem augers into the soil. The hollow center part of the augers gives access for sampling or any other testing device that can be lowered to the bottom of the hole. The hollow stem auger has the advantage of providing a casing against collapse of the side walls of the borehole, but is limited in penetration depth because it requires significant torque to advance the augers.

✓Wet Rotary Drilling Method

- The wet rotary method consists of drilling a borehole with a drill bit while circulating drilling mud through the center of the rods. The drill bit is typically 75 to 150 mm in diameter and the rods 40 to 70mm in diameter.

Drill bits

- The drilling mud flows down the center of the rods while they rotate and back to the surface on the outside of the rods between the wall of the borehole and the exterior wall of the rods.
- This return flow carries the soil cuttings back to the surface by entrainment. The drilling mud arrives in the mud pit (Figure 6.3), where it is sucked back up to the top of the drilling rods by a pump.

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Boring Methods, cont'd

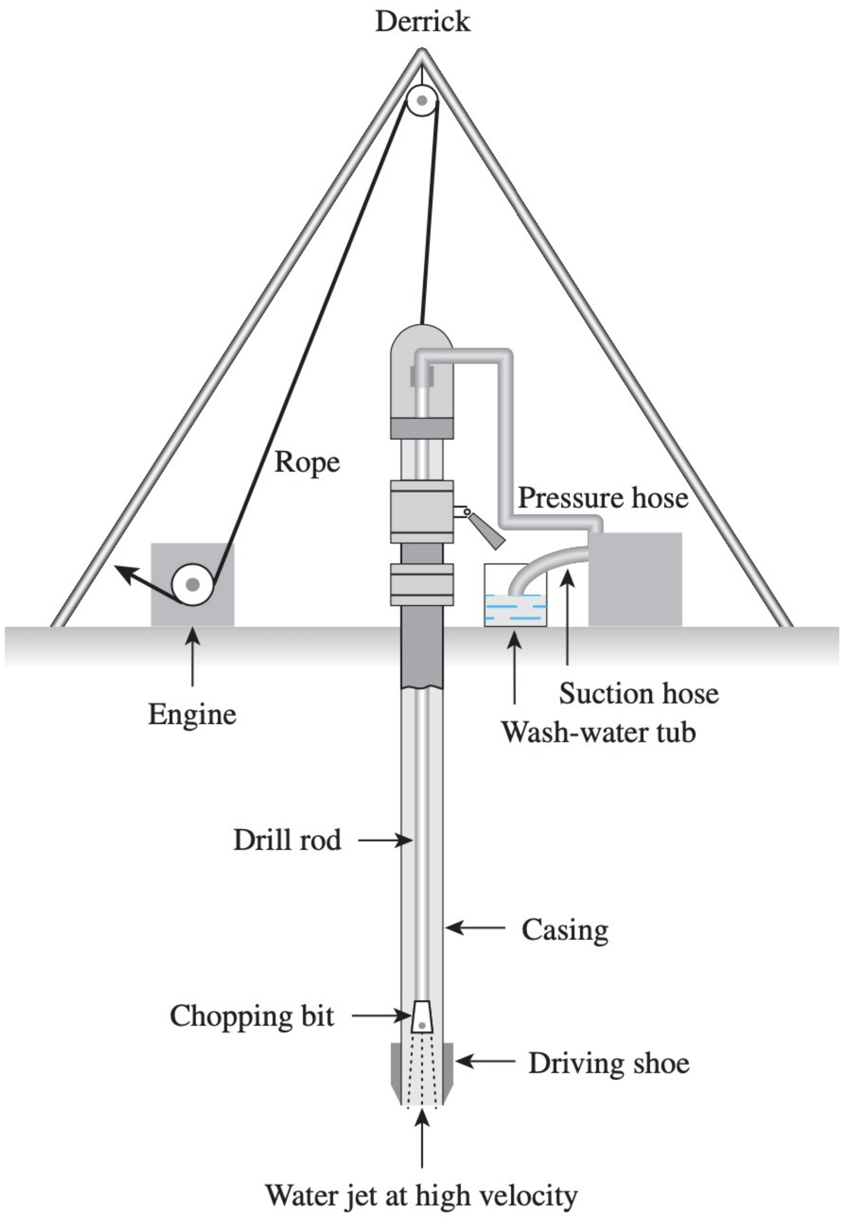
- ***Rotary drilling*** is a procedure by which rapidly rotating drilling bits attached to the bottom of drilling rods cut and grind the soil and advance the borehole down. Several types of drilling bits are available for such work. Rotary drilling can be used in sand, clay, and rock (unless badly fissured). Water or drilling mud is forced down the drilling rods to the bits, and the return flow forces the cuttings to the surface. Drilling mud is a slurry prepared by mixing bentonite and water (bentonite is a montmorillonite clay formed by the weathering of volcanic ash). Boreholes with diameters ranging from 50 to 200 mm (2 to 8 in.) can be made easily by using this technique.

Boring Methods, cont'd

- ***Wash boring*** is another method of advancing boreholes. In this method, a casing about 2 to 3 m (6 to 10 ft) long is driven into the ground. The soil inside the casing then is removed by means of a chopping bit that is attached to a drilling rod. Water is forced through the drilling rod, and it goes out at a very high velocity through the holes at the bottom of the chop- ping bit (The Figure). The water and the chopped soil particles rise upward in the drill hole and overflow at the top of the casing through a T-connection. The wash water then is collected in a container. The casing can be extended with additional pieces as the borehole progresses; however, such extension is not necessary if the borehole will stay open without caving in.

Boring Methods, cont'd

Wash boring



Boring Methods, cont'd

- ***Percussion drilling*** is an alternative method of advancing a borehole, particularly through hard soil and rock. In this technique, a heavy drilling bit is raised and lowered to chop the hard soil. Casing for this type of drilling may be required. The chopped soil particles are brought up by the circulation of water.

Please, refer to the following link to download animation video which is explaining the methods of drilling:

https://www.youtube.com/watch?v=i9eQcc7ilVw

Drilling methods in animation tube


Thank
You

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

Instructor: Professor Dr. Hussein M. Ashour Al.Khuzaie (Ph.D., Civil Engineering, Foundation Engineering and Structures);

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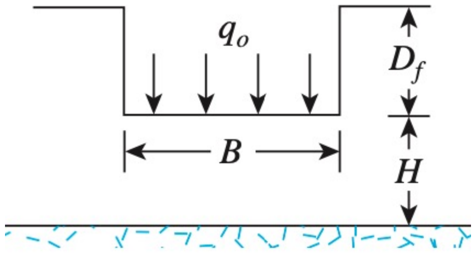
Settlement of Foundation and Compressibility of Soil (2)

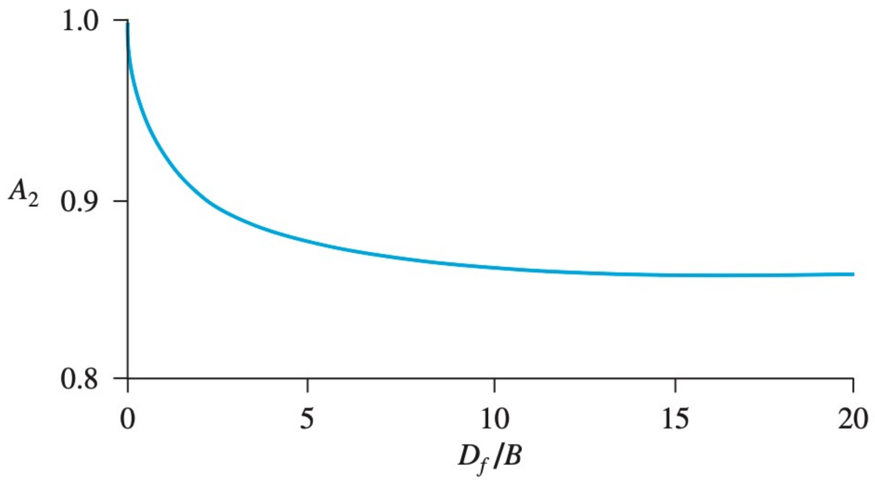
- **Elastic (Immediate) Settlement for saturated clay (μ_s = 0.5)**
- **Consolidation Settlement:**
  - **Primary Settlement**
  - **Secondary Settlement**
- **Limits of tolerable settlement and distortion of structures and foundations**

Elastic (Immediate) Settlement for saturated clay (μ_s = 0.5)

The average settlement of flexible foundations on saturated clay soils (Poisson's ratio, $\mu_s = 0.5$).

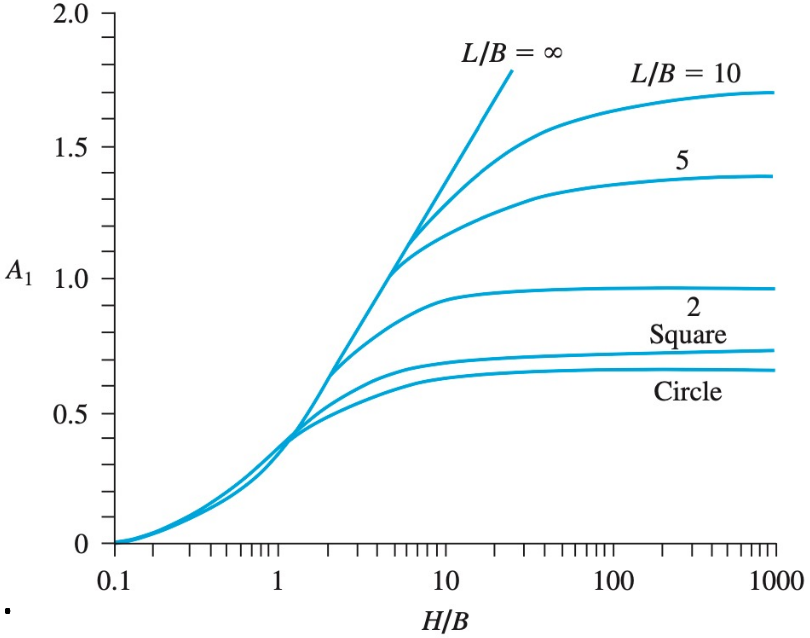
$$S_e = A_1 A_2 \frac{q_o B}{E_s}$$





where

$A_1 = f(H/B, L/B)$
$A_2 = f(D_f/B)$
L = length of the foundation
B = width of the foundation
D_f = depth of the foundation
H = depth of the bottom of the foundation to a rigid layer
q_o = net load per unit area of the foundation



The modulus of elasticity (E_s) for saturated clays can, in general, be given as: $E_s = \beta C_u$

where C_u = undrained shear strength.
The parameter β is primarily a function of the plasticity index and overconsolidation ratio (OCR). Table 7.1 provides a general range for β.

Table 7.1 Range of β for Saturated Clay [Eq. (7.2)][a]

Plasticity Index	β				
	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
<30	1500–600	1380–500	1200–580	950–380	730–300
30 to 50	600–300	550–270	580–220	380–180	300–150
>50	300–150	270–120	220–100	180–90	150–75

[a]Based on Duncan and Buchignani (1976)

The **OCR** is defined as the ratio of the **maximum past effective consolidation stress (Pre-consolidation pressure)** and the **present effective overburden stress.**

Example

Consider a shallow foundation 2 m × 1 m in plan in a saturated clay layer. A rigid rock layer is located 8 m below the bottom of the foundation. Given:

Foundation: $D_f = 1$ m, $q_o = 120$ kN/m^2
Clay: $c_u = 150$ kN/m^2, OCR = 2, and Plasticity index, PI = 35

Estimate the elastic settlement of the foundation.

Solution:

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

For OCR = 2 and PI = 35, the value of $\beta \approx 480$ (Table 7.1). Hence,

$$E_s = (480)(150) = 72{,}000 \text{ kN/m}^2$$

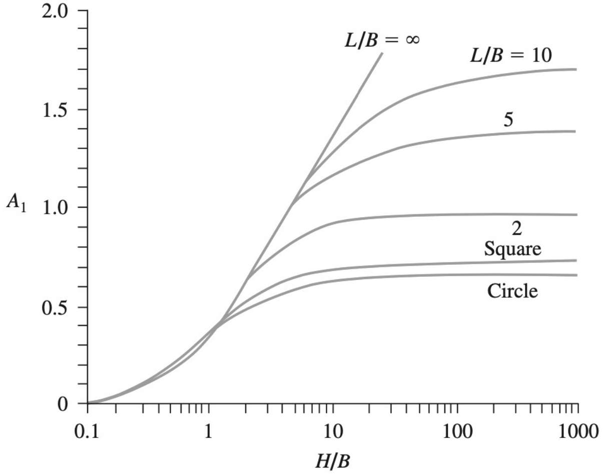
$$\frac{L}{B} = \frac{2}{1} = 2$$

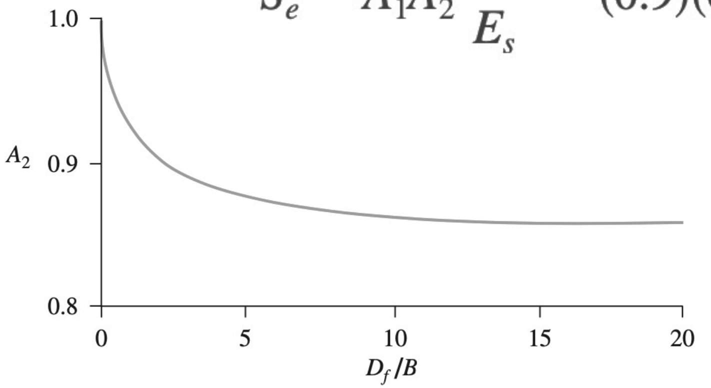
$$\frac{D_f}{B} = \frac{1}{1} = 1$$

$$\frac{H}{B} = \frac{8}{1} = 8$$

$$E_s = \beta c_u$$

Also, from Figures:





$A_1 = 0.9$ and $A_2 = 0.92$. Hence,

$$S_e = A_1 A_2 \frac{q_o B}{E_s} = (0.9)(0.92)\frac{(120)(1)}{72{,}000} = 0.00138 \text{ m} = \mathbf{1.38\ mm}$$

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Primary Consolidation Settlement

Consolidation settlement occurs over time, i.e., it is time dependent, in saturated clayey soils subjected to an increased load caused by construction of the foundation.

$$S_{c(p)} = \int \varepsilon_z dz$$

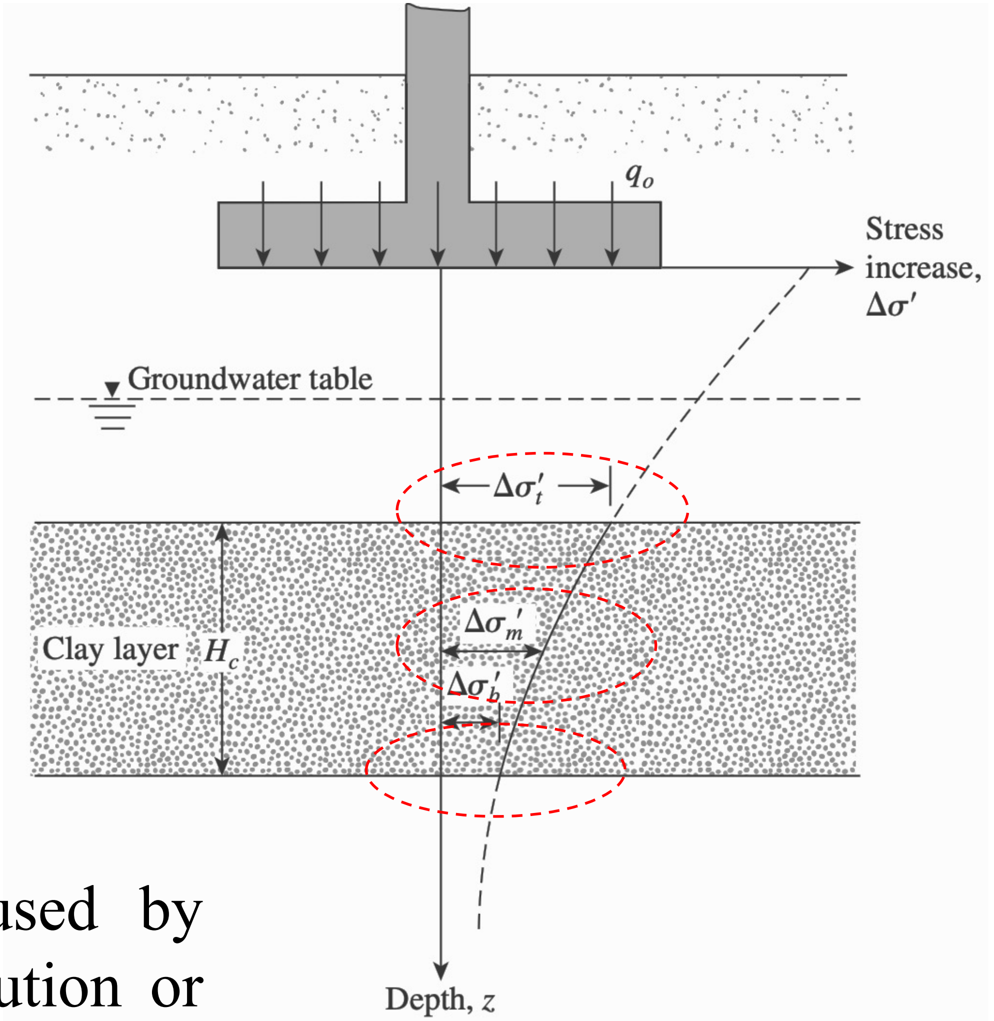
where

ε_z = vertical strain

$= \dfrac{\Delta e}{1 + e_o}$

Δe = change of void ratio

$= f(\sigma'_o, \sigma'_c, \text{and } \Delta\sigma')$



The method of determining the pressure increase caused by various types of foundation load using Boussinesq's solution or approximately by 2:1 method

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

So, **Primary Consolidation Settlement** can be calculated using the following equations

$$S_{c(p)} = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$ (for normally consolidated clays)

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

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

where

σ'_o = average effective pressure on the clay layer before the construction of the foundation

$\Delta\sigma'_{av}$ = 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

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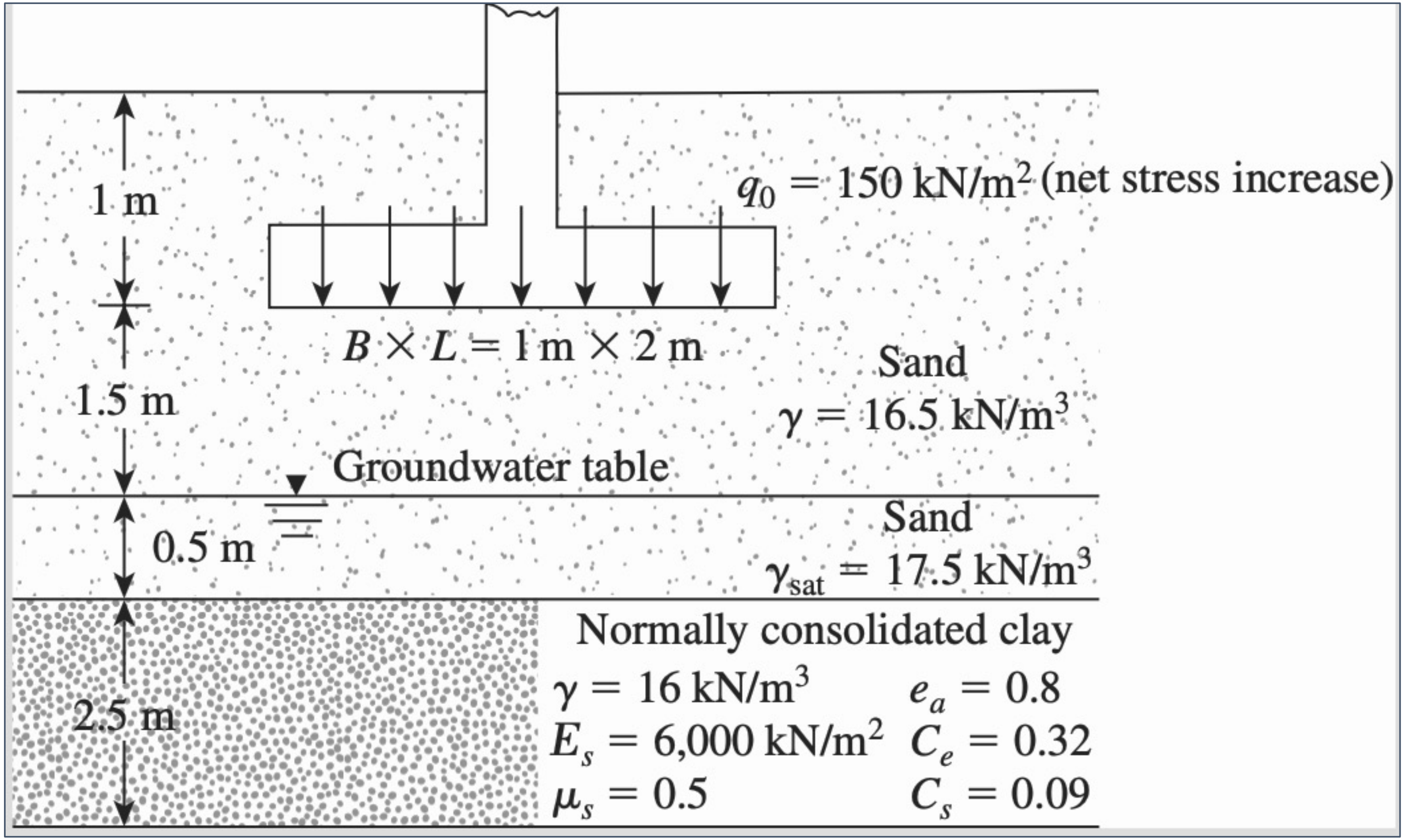
Example: A plan of a foundation 1 m X 2 m is shown in the Figure. Estimate the primary consolidation settlement of the foundation.



Solution

The clay is normally consolidated. Thus,

$$S_{c(p)-\text{oed}} = \frac{C_c H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{\text{av}}}{\sigma'_o} = \frac{(0.32)(2.5)}{1+0.8} \log\left(\frac{52.84 + 14.38}{52.84}\right) = 0.0465 \text{ m}$$

$$= \mathbf{46.5\ mm}$$

so

$$\sigma'_o = (2.5)(16.5) + (0.5)(17.5 - 9.81) + (1.25)(16 - 9.81)$$

$$= 41.25 + 3.85 + 7.74 = 52.84 \text{ kN/m}^2$$

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

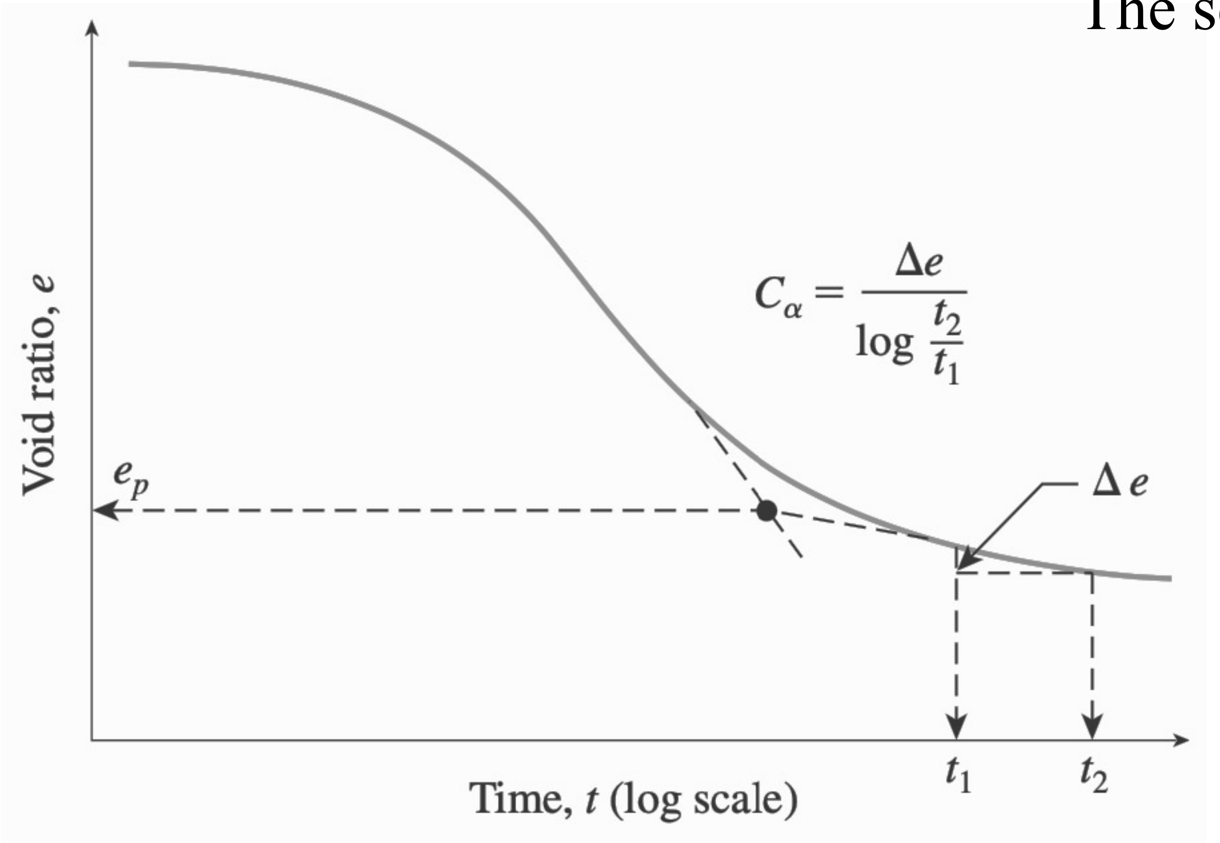
Now the following table can be prepared (*Note:* $L = 2$ m; $B = 1$ m):

$m_1 = L/B$	z(m)	$z/(B/2) = n_1$	I_c[a]	$\Delta\sigma' = q_o I_c$[b]
2	2	4	0.190	$28.5 = \Delta\sigma'_t$
2	$2 + 2.5/2 = 3.25$	6.5	≈ 0.085	$12.75 = \Delta\sigma'_m$
2	$2 + 2.5 = 4.5$	9	0.045	$6.75 = \Delta\sigma'_b$

$$\Delta\sigma'_{\text{av}} = \tfrac{1}{6}(28.5 + 4 \times 12.75 + 6.75) = 14.38 \text{ kN/m}^2$$

Secondary Consolidation Settlement

The plastic adjustment of soil fabrics due to the dislocation of soil particles resulting in settlement that called secondary settlement. This type of settlement is observed at the end of primary consolidation (i.e., after the complete dissipation of excess pore water pressure). A plot of deformation against the logarithm of time during secondary consolidation is practically linear as shown in the Figure.



The secondary compression index can be defined as:

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

where

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

The magnitude of the secondary consolidation can be calculated as:

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

where

$C'_\alpha = C_\alpha/(1 + e_p)$
e_p = void ratio at the end of primary consolidation
H_c = thickness of clay layer

C'_α correlated with the natural moisture content (w) of soils as: $C'_\alpha \approx 0.0001w$

The magnitude of C_α/C_c (C_c = compression index) for a number of soils:

- 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:

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

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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.

Example:

Refer to the example in slide(7),Given for the clay layer: $C_\alpha = 0.02$. Estimate the total consolidation settlement five (5) years after the completion of the primary consolidation settlement. (*Note:* Time for completion of primary consolidation settlement is 1.3 years).

$$C_c = \frac{e_1 - e_2}{\log\left(\frac{\sigma_2'}{\sigma_1'}\right)}$$

$$\sigma_2' = \sigma_o' + \Delta\sigma' = 52.84 + 14.38 = 67.22 \text{ kN/m}^2$$

$$\sigma_1' = \sigma_o' = 52.84 \text{ kN/m}^2$$

$$C_c = 0.32$$

For this problem, $e_1 - e_2 = \Delta e$.

Hence,

$$\Delta e = C_c \log\left(\frac{\sigma_o' + \Delta\sigma}{\sigma_o'}\right) = 0.32 \log\left(\frac{67.22}{52.84}\right) = 0.0335$$

Given: $e_o = 0.8$. Hence,

$$e_p = e_o - e = 0.8 - 0.0335 = 0.7665$$

$$C'_\alpha = \frac{C_\alpha}{1 + e_p} = \frac{0.02}{1 + 0.7665} = 0.0113$$

$$S_{c(s)} = C'_\alpha H_c \log\left(\frac{t_2}{t_1}\right)$$

Note: $t_1 = 1.3$ years; $t_2 = 1.3 + 5 = 6.3$ years.

Thus,

$$S_{c(s)} = (0.0113)(2.5\text{ m})\log\left(\frac{6.3}{1.3}\right) = 0.0194\text{ m} = 19.4\text{ mm}$$

Total consolidation settlement = Primary settlement (S_p) + Secondary settlement (S_s)

= *46.5 mm + 19.4 mm = 65.9 mm*

Tolerable Settlement of Buildings

In most instances of construction, the subsoil is not homogeneous and the load carried by various shallow foundations of a given structure can vary widely. As a result, it is reasonable to expect varying degrees of settlement in different parts of a given building.

The *differential settlement* of the parts of a building can lead to damage of the superstructure.

Hence, it is important to define certain parameters that quantify differential settlement and to develop limiting values for those parameters in order that the resulting structures be safe.

Burland and Wroth (1970) summarized the important parameters relating to differential settlement.

The figure shows a structure in which various foundations, at *A, B, C, D,* and *E,* have gone through some settlement. The settlement at *A* is *AA′*, at *B* is *BB′*, etc. Based on this figure, the definitions of the various parameters are as follows:

S_T = total settlement of a given point

ΔS_T = difference in total settlement between any two points

α = gradient between two successive points

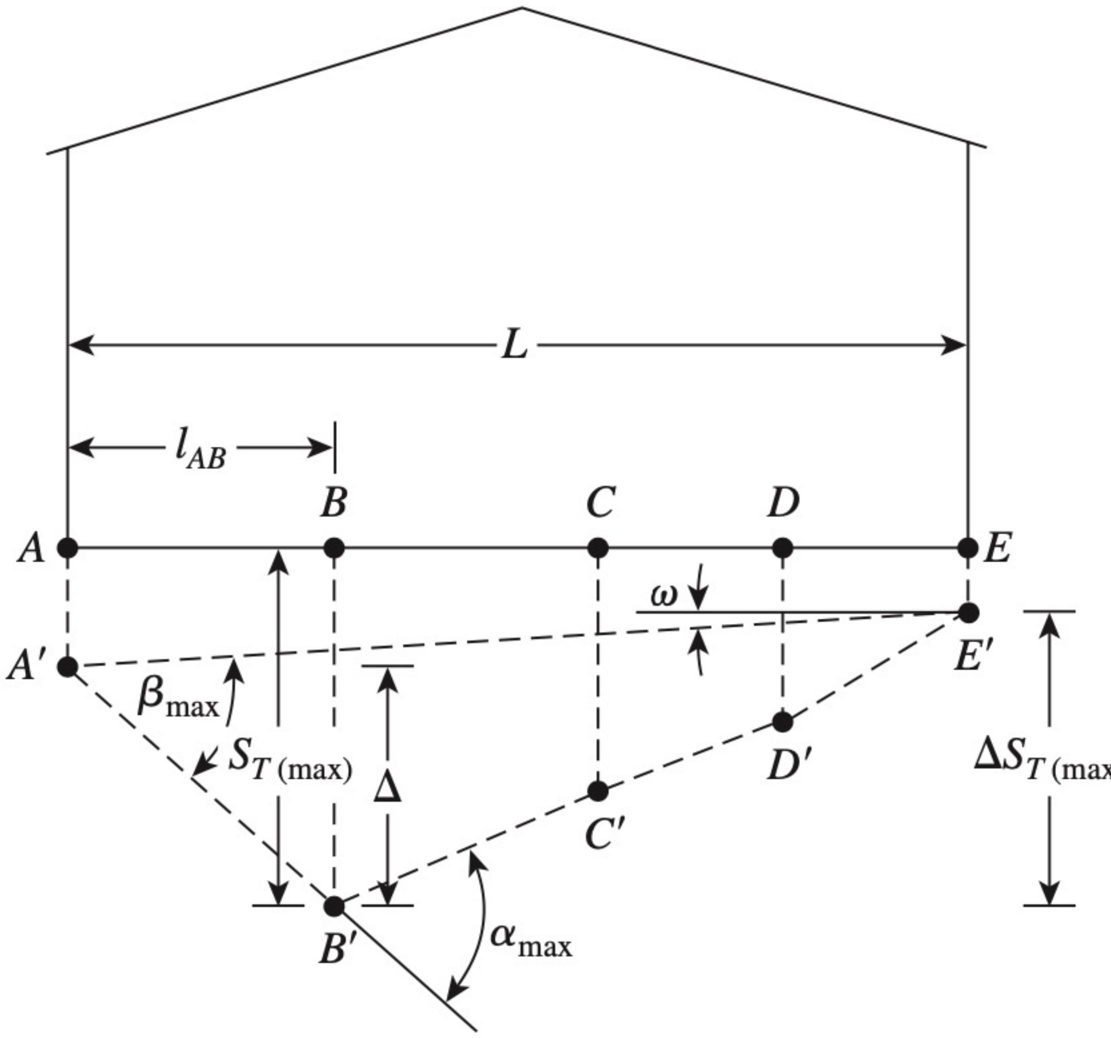
β = angular distortion = $\dfrac{\Delta S_{T(ij)}}{l_{ij}}$

(*Note*: l_{ij} = distance between points i and j)

ω = tilt

Δ = relative deflection (i.e., movement from a straight line joining two reference points)

$\dfrac{\Delta}{L}$ = deflection ratio



Allowable linear distortion for building, SNIP (Russian Code), 1955

Type of building	L/H	Δ/L
Multistory buildings and civil dwellings	$\leqslant 3$	0.0003 (for sand) 0.0004 (for clay)
	$\geqslant 5$	0.0005 (for sand) 0.0007 (for clay)
One-story mills		0.001 (for sand and clay)

Angular distortion for some structures as proposed by Norwegian scientist, Bjerrum (1963)

Category of potential damage	β_{max}
Safe limit for flexible brick wall ($L/H > 4$)	1/150
Danger of structural damage to most buildings	1/150
Cracking of panel and brick walls	1/150
Visible tilting of high rigid buildings	1/250
First cracking of panel walls	1/300
Safe limit for no cracking of building	1/500
Danger to frames with diagonals	1/600

In 1956, Skempton and McDonald proposed the following limiting values for maximum settlement and maximum angular distortion, to be used for building purposes:

Maximum settlement, $S_{T(\max)}$	
In sand	32 mm
In clay	45 mm
Maximum differential settlement, $\Delta S_{T(\max)}$	
Isolated foundations in sand	51 mm
Isolated foundations in clay	76 mm
Raft in sand	51–76 mm
Raft in clay	76–127 mm
Maximum angular distortion, $\beta_{\max}$	1/300

If the maximum allowable values of $\beta_{\max}$ are known, the magnitude of the allowable $S_{T(\max)}$ can be calculated with the use of the foregoing correlations.

The ***European Committee for Standardization*** has also provided limiting values for serviceability and the maximum accepted foundation movements, as in the table:

Item	Parameter	Magnitude	Comments
Limiting values for serviceability (European Committee for Standardization, 1994a)	S_T	25 mm	Isolated shallow foundation
		50 mm	Raft foundation
	ΔS_T	5 mm	Frames with rigid cladding
		10 mm	Frames with flexible cladding
		20 mm	Open frames
	β	1/500	—
Maximum acceptable foundation movement (European Committee for Standardization, 1994b)	S_T	50	Isolated shallow foundation
	ΔS_T	20	Isolated shallow foundation
	β	≈1/500	—

End of Settlement theme,
Any Question //???

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

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Settlement of Foundation and Compressibility of Soil (1)

- **General**
- **Elastic (Immediate) Settlement**

General

When the load induced by structures or other sources increases, it results in compression the soil under the foundation.

The compression is due to:

(a)deformation of soil particles,

(b)relocations of soil particles, and

(c)expulsion of water or air from the void spaces.

So, the soil settlement caused by loads may be divided into three broad categories:

1. *Elastic settlement* (or *immediate settlement*), which is caused by the elastic deformation of dry soil and of moist and saturated soils without any change in the moisture content. Elastic settlement calculations generally are based on equations derived from the theory of elasticity.

2. *Primary consolidation settlement,* which is the result of a volume change in saturated cohesive soils because of expulsion of the water that occupies the void spaces.

3. *Secondary consolidation settlement,* which is observed in saturated cohesive soils and is the result of the plastic adjustment of soil fabrics. It is an additional form of compression that occurs at constant effective stress. It is similar to the creep of solid material, where the deformation increases while the stress is constant.

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General, cont'd

The total settlement of a foundation can then be given as:

$$S_T = S_c + S_s + S_e$$

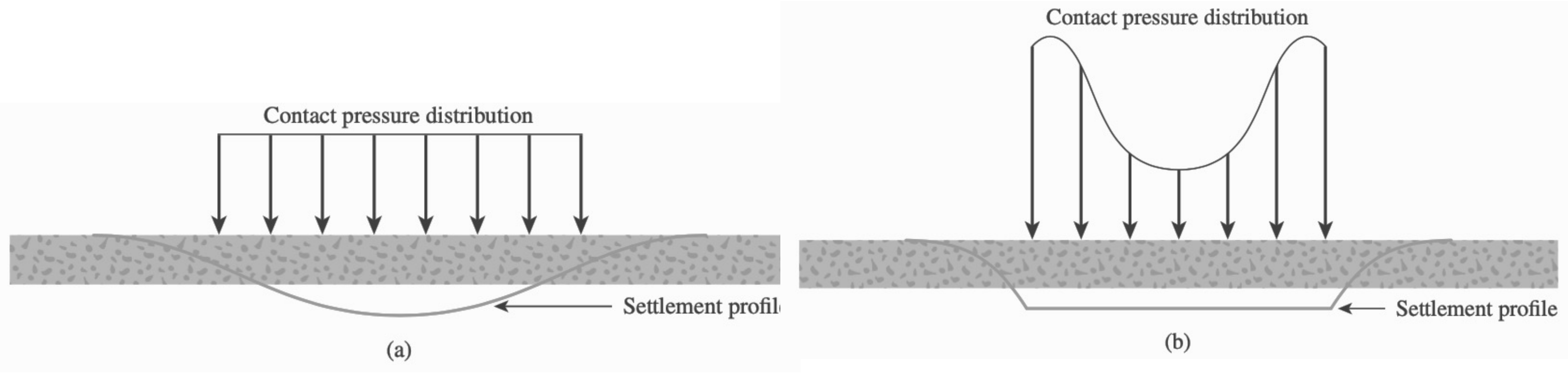
where S_T = total settlement
S_c = primary consolidation settlement
S_s = secondary consolidation settlement
S_e = elastic settlement

When foundations are constructed on very compressible clays, the consolidation settlement can be several times greater than the elastic settlement.

Elastic Settlement

The distribution of the contact pressure of the structure within the media of the soil is depending on some assumptions in spite of type of this loading (line load, strip load, embankment load, circular load, and rectangular load). These assumptions are:

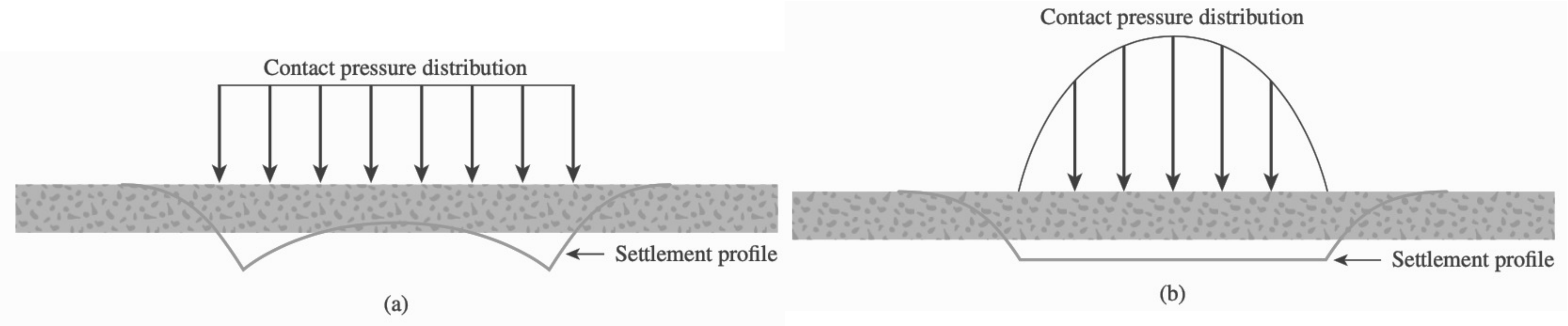
- **The load is applied at the ground surface.**
- **The loaded area is flexible.**
- **The soil medium is homogeneous, elastic, isotropic, and extends to a great depth.**



Elastic settlement profile and contact pressure in clay:
(a) flexible foundation; (b) rigid foundation

Elastic Settlement, cont’d

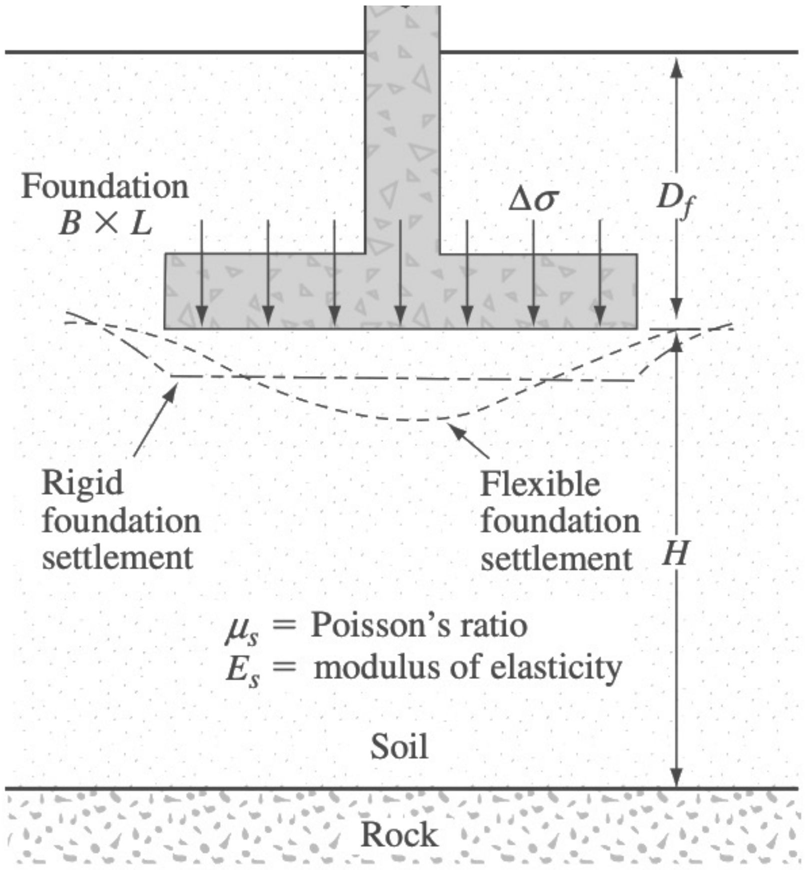
In the case of cohesionless soil (sand), the modulus of elasticity increases with depth. Additionally, there is a lack of lateral confinement on the edge of the foundation at the ground surface. The sand at the edge of a flexible foundation is pushed outward, and the deflection curve of the foundation takes a concave downward shape. The distributions of contact pressure and the settlement profiles of a flexible and a rigid foundation resting on sand and subjected to uniform loading are shown in below, Figures a and b, respectively.



Elastic settlement profile and contact pressure in sand:
(a) flexible foundation; (b) rigid foundation

As shown in the Figure, a shallow foundation subjected to a net force per unit area equal to $\Delta\sigma$. Let the Poisson's ratio and the modulus of elasticity of the soil supporting it be μ_s and E_s, respectively. Theoretically, if the foundation is perfectly flexible, the settlement may be expressed as:

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



Elastic settlement of flexible and rigid foundations

where $\Delta\sigma$ = net applied pressure on the foundation

μ_s = Poisson's ratio of soil

E_s = average modulus of elasticity of the soil under the foundation measured from $z = 0$ to about $z = 4B$

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

$= B$ for corner of foundation

I_s = shape factor (Steinbrenner, 1934)

I_f = depth factor (Fox, 1948) $= f\left(\frac{D_f}{B}, \mu_s, \text{and } \frac{L}{B}\right)$

α = factor that depends on the location on the foundation where settlement is being calculated

$$I_s = \text{shape factor}$$

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

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

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

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

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

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

- For calculation of settlement at the *center* of the foundation:

$$\alpha = 4$$

$$m' = \frac{L}{B}$$

$$n' = \frac{H}{\left(\frac{B}{2}\right)}$$

- For calculation of settlement at a *corner* of the foundation:

$$\alpha = 1$$

$$m' = \frac{L}{B}$$

$$n' = \frac{H}{B}$$

Table 11.1 Variation of F_1 with m' and n'

	m'									
n'	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.014	0.013	0.012	0.011	0.011	0.011	0.010	0.010	0.010	0.010
0.50	0.049	0.046	0.044	0.042	0.041	0.040	0.038	0.038	0.037	0.037
0.75	0.095	0.090	0.087	0.084	0.082	0.080	0.077	0.076	0.074	0.074
1.00	0.142	0.138	0.134	0.130	0.127	0.125	0.121	0.118	0.116	0.115
1.25	0.186	0.183	0.179	0.176	0.173	0.170	0.165	0.161	0.158	0.157
1.50	0.224	0.224	0.222	0.219	0.216	0.213	0.207	0.203	0.199	0.197
1.75	0.257	0.259	0.259	0.258	0.255	0.253	0.247	0.242	0.238	0.235
2.00	0.285	0.290	0.292	0.292	0.291	0.289	0.284	0.279	0.275	0.271
2.25	0.309	0.317	0.321	0.323	0.323	0.322	0.317	0.313	0.308	0.305
2.50	0.330	0.341	0.347	0.350	0.351	0.351	0.348	0.344	0.340	0.336
2.75	0.348	0.361	0.369	0.374	0.377	0.378	0.377	0.373	0.369	0.365
3.00	0.363	0.379	0.389	0.396	0.400	0.402	0.402	0.400	0.396	0.392
3.25	0.376	0.394	0.406	0.415	0.420	0.423	0.426	0.424	0.421	0.418
3.50	0.388	0.408	0.422	0.431	0.438	0.442	0.447	0.447	0.444	0.441
3.75	0.399	0.420	0.436	0.447	0.454	0.460	0.467	0.458	0.466	0.464
4.00	0.408	0.431	0.448	0.460	0.469	0.476	0.484	0.487	0.486	0.484
4.25	0.417	0.440	0.458	0.472	0.481	0.484	0.495	0.514	0.515	0.515
4.50	0.424	0.450	0.469	0.484	0.495	0.503	0.516	0.521	0.522	0.522
4.75	0.431	0.458	0.478	0.494	0.506	0.515	0.530	0.536	0.539	0.539
5.00	0.437	0.465	0.487	0.503	0.516	0.526	0.543	0.551	0.554	0.554
5.25	0.443	0.472	0.494	0.512	0.526	0.537	0.555	0.564	0.568	0.569
5.50	0.448	0.478	0.501	0.520	0.534	0.546	0.566	0.576	0.581	0.584
5.75	0.453	0.483	0.508	0.527	0.542	0.555	0.576	0.588	0.594	0.597
6.00	0.457	0.489	0.514	0.534	0.550	0.563	0.585	0.598	0.606	0.609
6.25	0.461	0.493	0.519	0.540	0.557	0.570	0.594	0.609	0.617	0.621
6.50	0.465	0.498	0.524	0.546	0.563	0.577	0.603	0.618	0.627	0.632
6.75	0.468	0.502	0.529	0.551	0.569	0.584	0.610	0.627	0.637	0.643
7.00	0.471	0.506	0.533	0.556	0.575	0.590	0.618	0.635	0.646	0.653
7.25	0.474	0.509	0.538	0.561	0.580	0.596	0.625	0.643	0.655	0.662
7.50	0.477	0.513	0.541	0.565	0.585	0.601	0.631	0.650	0.663	0.671
7.75	0.480	0.516	0.545	0.569	0.589	0.606	0.637	0.658	0.671	0.680
8.00	0.482	0.519	0.549	0.573	0.594	0.611	0.643	0.664	0.678	0.688
8.25	0.485	0.522	0.552	0.577	0.598	0.615	0.648	0.670	0.685	0.695
8.50	0.487	0.524	0.555	0.580	0.601	0.619	0.653	0.676	0.692	0.703
8.75	0.489	0.527	0.558	0.583	0.605	0.623	0.658	0.682	0.698	0.710
9.00	0.491	0.529	0.560	0.587	0.609	0.627	0.663	0.687	0.705	0.716
9.25	0.493	0.531	0.563	0.589	0.612	0.631	0.667	0.693	0.710	0.723
9.50	0.495	0.533	0.565	0.592	0.615	0.634	0.671	0.697	0.716	0.719
9.75	0.496	0.536	0.568	0.595	0.618	0.638	0.675	0.702	0.721	0.735
10.00	0.498	0.537	0.570	0.597	0.621	0.641	0.679	0.707	0.726	0.740
20.00	0.529	0.575	0.614	0.647	0.677	0.702	0.756	0.797	0.830	0.858
50.00	0.548	0.598	0.640	0.678	0.711	0.740	0.803	0.853	0.895	0.931
100.00	0.555	0.605	0.649	0.688	0.722	0.753	0.819	0.872	0.918	0.956

Table 11.1 *(continued)*

	m'									
n'	4.5	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.25	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
0.50	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036
0.75	0.073	0.073	0.072	0.072	0.072	0.072	0.071	0.071	0.071	0.071
1.00	0.114	0.113	0.112	0.112	0.112	0.111	0.111	0.110	0.110	0.110
1.25	0.155	0.154	0.153	0.152	0.152	0.151	0.151	0.150	0.150	0.150
1.50	0.195	0.194	0.192	0.191	0.190	0.190	0.189	0.188	0.188	0.188
1.75	0.233	0.232	0.229	0.228	0.227	0.226	0.225	0.223	0.223	0.223
2.00	0.269	0.267	0.264	0.262	0.261	0.260	0.259	0.257	0.256	0.256
2.25	0.302	0.300	0.296	0.294	0.293	0.291	0.291	0.287	0.287	0.287
2.50	0.333	0.331	0.327	0.324	0.322	0.321	0.320	0.316	0.315	0.315
2.75	0.362	0.359	0.355	0.352	0.350	0.348	0.347	0.343	0.342	0.342
3.00	0.389	0.386	0.382	0.378	0.376	0.374	0.373	0.368	0.367	0.367
3.25	0.415	0.412	0.407	0.403	0.401	0.399	0.397	0.391	0.390	0.390
3.50	0.438	0.435	0.430	0.427	0.424	0.421	0.420	0.413	0.412	0.411
3.75	0.461	0.458	0.453	0.449	0.446	0.443	0.441	0.433	0.432	0.432
4.00	0.482	0.479	0.474	0.470	0.466	0.464	0.462	0.453	0.451	0.451
4.25	0.516	0.496	0.484	0.473	0.471	0.471	0.470	0.468	0.462	0.460
4.50	0.520	0.517	0.513	0.508	0.505	0.502	0.499	0.489	0.487	0.487
4.75	0.537	0.535	0.530	0.526	0.523	0.519	0.517	0.506	0.504	0.503
5.00	0.554	0.552	0.548	0.543	0.540	0.536	0.534	0.522	0.519	0.519
5.25	0.569	0.568	0.564	0.560	0.556	0.553	0.550	0.537	0.534	0.534
5.50	0.584	0.583	0.579	0.575	0.571	0.568	0.585	0.551	0.549	0.548
5.75	0.597	0.597	0.594	0.590	0.586	0.583	0.580	0.565	0.583	0.562
6.00	0.611	0.610	0.608	0.604	0.601	0.598	0.595	0.579	0.576	0.575
6.25	0.623	0.623	0.621	0.618	0.615	0.611	0.608	0.592	0.589	0.588
6.50	0.635	0.635	0.634	0.631	0.628	0.625	0.622	0.605	0.601	0.600
6.75	0.646	0.647	0.646	0.644	0.641	0.637	0.634	0.617	0.613	0.612
7.00	0.656	0.658	0.658	0.656	0.653	0.650	0.647	0.628	0.624	0.623
7.25	0.666	0.669	0.669	0.668	0.665	0.662	0.659	0.640	0.635	0.634
7.50	0.676	0.679	0.680	0.679	0.676	0.673	0.670	0.651	0.646	0.645
7.75	0.685	0.688	0.690	0.689	0.687	0.684	0.681	0.661	0.656	0.655
8.00	0.694	0.697	0.700	0.700	0.698	0.695	0.692	0.672	0.666	0.665
8.25	0.702	0.706	0.710	0.710	0.708	0.705	0.703	0.682	0.676	0.675
8.50	0.710	0.714	0.719	0.719	0.718	0.715	0.713	0.692	0.686	0.684
8.75	0.717	0.722	0.727	0.728	0.727	0.725	0.723	0.701	0.695	0.693
9.00	0.725	0.730	0.736	0.737	0.736	0.735	0.732	0.710	0.704	0.702
9.25	0.731	0.737	0.744	0.746	0.745	0.744	0.742	0.719	0.713	0.711
9.50	0.738	0.744	0.752	0.754	0.754	0.753	0.751	0.728	0.721	0.719
9.75	0.744	0.751	0.759	0.762	0.762	0.761	0.759	0.737	0.729	0.727
10.00	0.750	0.758	0.766	0.770	0.770	0.770	0.768	0.745	0.738	0.735
20.00	0.878	0.896	0.925	0.945	0.959	0.969	0.977	0.982	0.965	0.957
50.00	0.962	0.989	1.034	1.070	1.100	1.125	1.146	1.265	1.279	1.261
100.00	0.990	1.020	1.072	1.114	1.150	1.182	1.209	1.408	1.489	1.499

Table 11.2 Variation of F_2 with m' and n'

n'	m' = 1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	3.5	4.0
0.25	0.049	0.050	0.051	0.051	0.051	0.052	0.052	0.052	0.052	0.052
0.50	0.074	0.077	0.080	0.081	0.083	0.084	0.086	0.086	0.0878	0.087
0.75	0.083	0.089	0.093	0.097	0.099	0.101	0.104	0.106	0.107	0.108
1.00	0.083	0.091	0.098	0.102	0.106	0.109	0.114	0.117	0.119	0.120
1.25	0.080	0.089	0.096	0.102	0.107	0.111	0.118	0.122	0.125	0.127
1.50	0.075	0.084	0.093	0.099	0.105	0.110	0.118	0.124	0.128	0.130
1.75	0.069	0.079	0.088	0.095	0.101	0.107	0.117	0.123	0.128	0.131
2.00	0.064	0.074	0.083	0.090	0.097	0.102	0.114	0.121	0.127	0.131
2.25	0.059	0.069	0.077	0.085	0.092	0.098	0.110	0.119	0.125	0.130
2.50	0.055	0.064	0.073	0.080	0.087	0.093	0.106	0.115	0.122	0.127
2.75	0.051	0.060	0.068	0.076	0.082	0.089	0.102	0.111	0.119	0.125
3.00	0.048	0.056	0.064	0.071	0.078	0.084	0.097	0.108	0.116	0.122
3.25	0.045	0.053	0.060	0.067	0.074	0.080	0.093	0.104	0.112	0.119
3.50	0.042	0.050	0.057	0.064	0.070	0.076	0.089	0.100	0.109	0.116
3.75	0.040	0.047	0.054	0.060	0.067	0.073	0.086	0.096	0.105	0.113
4.00	0.037	0.044	0.051	0.057	0.063	0.069	0.082	0.093	0.102	0.110
4.25	0.036	0.042	0.049	0.055	0.061	0.066	0.079	0.090	0.099	0.107
4.50	0.034	0.040	0.046	0.052	0.058	0.063	0.076	0.086	0.096	0.104
4.75	0.032	0.038	0.044	0.050	0.055	0.061	0.073	0.083	0.093	0.101
5.00	0.031	0.036	0.042	0.048	0.053	0.058	0.070	0.080	0.090	0.098
5.25	0.029	0.035	0.040	0.046	0.051	0.056	0.067	0.078	0.087	0.095
5.50	0.028	0.033	0.039	0.044	0.049	0.054	0.065	0.075	0.084	0.092
5.75	0.027	0.032	0.037	0.042	0.047	0.052	0.063	0.073	0.082	0.090
6.00	0.026	0.031	0.036	0.040	0.045	0.050	0.060	0.070	0.079	0.087
6.25	0.025	0.030	0.034	0.039	0.044	0.048	0.058	0.068	0.077	0.085
6.50	0.024	0.029	0.033	0.038	0.042	0.046	0.056	0.066	0.075	0.083
6.75	0.023	0.028	0.032	0.036	0.041	0.045	0.055	0.064	0.073	0.080
7.00	0.022	0.027	0.031	0.035	0.039	0.043	0.053	0.062	0.071	0.078
7.25	0.022	0.026	0.030	0.034	0.038	0.042	0.051	0.060	0.069	0.076
7.50	0.021	0.025	0.029	0.033	0.037	0.041	0.050	0.059	0.067	0.074
7.75	0.020	0.024	0.028	0.032	0.036	0.039	0.048	0.057	0.065	0.072
8.00	0.020	0.023	0.027	0.031	0.035	0.038	0.047	0.055	0.063	0.071
8.25	0.019	0.023	0.026	0.030	0.034	0.037	0.046	0.054	0.062	0.069
8.50	0.018	0.022	0.026	0.029	0.033	0.036	0.045	0.053	0.060	0.067
8.75	0.018	0.021	0.025	0.028	0.032	0.035	0.043	0.051	0.059	0.066
9.00	0.017	0.021	0.024	0.028	0.031	0.034	0.042	0.050	0.057	0.064
9.25	0.017	0.020	0.024	0.027	0.030	0.033	0.041	0.049	0.056	0.063
9.50	0.017	0.020	0.023	0.026	0.029	0.033	0.040	0.048	0.055	0.061
9.75	0.016	0.019	0.023	0.026	0.029	0.032	0.039	0.047	0.054	0.060
10.00	0.016	0.019	0.022	0.025	0.028	0.031	0.038	0.046	0.052	0.059
20.00	0.008	0.010	0.011	0.013	0.014	0.016	0.020	0.024	0.027	0.031
50.00	0.003	0.004	0.004	0.005	0.006	0.006	0.008	0.010	0.011	0.013
100.00	0.002	0.002	0.002	0.003	0.003	0.003	0.004	0.005	0.006	0.006

Table 11.2 *(continued)*

n'	m' = 4.5	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.25	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053	0.053
0.50	0.087	0.087	0.088	0.088	0.088	0.088	0.088	0.088	0.088	0.088
0.75	0.109	0.109	0.109	0.110	0.110	0.110	0.110	0.111	0.111	0.111
1.00	0.121	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.25	0.128	0.130	0.131	0.132	0.132	0.133	0.133	0.134	0.134	0.134
1.50	0.132	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
1.75	0.134	0.136	0.138	0.140	0.141	0.142	0.142	0.144	0.144	0.145
2.00	0.134	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148
2.25	0.133	0.136	0.140	0.142	0.144	0.145	0.146	0.149	0.150	0.150
2.50	0.132	0.135	0.139	0.142	0.144	0.146	0.147	0.151	0.151	0.151
2.75	0.130	0.133	0.138	0.142	0.144	0.146	0.147	0.152	0.152	0.153
3.00	0.127	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
3.25	0.125	0.129	0.135	0.140	0.143	0.145	0.147	0.153	0.154	0.154
3.50	0.122	0.126	0.133	0.138	0.142	0.144	0.146	0.153	0.155	0.155
3.75	0.119	0.124	0.131	0.137	0.141	0.143	0.145	0.154	0.155	0.155
4.00	0.116	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
4.25	0.113	0.119	0.127	0.133	0.138	0.141	0.144	0.154	0.156	0.156
4.50	0.110	0.116	0.125	0.131	0.136	0.140	0.143	0.154	0.156	0.156
4.75	0.107	0.113	0.123	0.130	0.135	0.139	0.142	0.154	0.156	0.157
5.00	0.105	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
5.25	0.102	0.108	0.118	0.126	0.131	0.136	0.139	0.154	0.156	0.157
5.50	0.099	0.106	0.116	0.124	0.130	0.134	0.138	0.154	0.156	0.157
5.75	0.097	0.103	0.113	0.122	0.128	0.133	0.136	0.154	0.157	0.157
6.00	0.094	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157
6.25	0.092	0.098	0.109	0.118	0.124	0.129	0.134	0.153	0.157	0.158
6.50	0.090	0.096	0.107	0.116	0.122	0.128	0.132	0.153	0.157	0.158
6.75	0.087	0.094	0.105	0.114	0.121	0.126	0.131	0.153	0.157	0.158
7.00	0.085	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
7.25	0.083	0.090	0.101	0.110	0.117	0.123	0.128	0.152	0.157	0.158
7.50	0.081	0.088	0.099	0.108	0.115	0.121	0.126	0.152	0.156	0.158
7.75	0.079	0.086	0.097	0.106	0.114	0.120	0.125	0.151	0.156	0.158
8.00	0.077	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
8.25	0.076	0.082	0.093	0.102	0.110	0.117	0.122	0.150	0.156	0.158
8.50	0.074	0.080	0.091	0.101	0.108	0.115	0.121	0.150	0.156	0.158
8.75	0.072	0.078	0.089	0.099	0.107	0.114	0.119	0.150	0.156	0.158
9.00	0.071	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
9.25	0.069	0.075	0.086	0.096	0.104	0.110	0.116	0.149	0.156	0.158
9.50	0.068	0.074	0.085	0.094	0.102	0.109	0.115	0.148	0.156	0.158
9.75	0.066	0.072	0.083	0.092	0.100	0.107	0.113	0.148	0.156	0.158
10.00	0.065	0.071	0.082	0.091	0.099	0.106	0.112	0.147	0.156	0.158
20.00	0.035	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
50.00	0.014	0.016	0.019	0.022	0.025	0.028	0.031	0.071	0.113	0.142
100.00	0.007	0.008	0.010	0.011	0.013	0.014	0.016	0.039	0.071	0.113

Table 11.3 Variation of I_f with L/B and D_f/B

		I_f		
L/B	D_f/B	$\mu_s = 0.3$	$\mu_s = 0.4$	$\mu_s = 0.5$
1	0.5	0.77	0.82	0.85
	0.75	0.69	0.74	0.77
	1	0.65	0.69	0.72
2	0.5	0.82	0.86	0.89
	0.75	0.75	0.79	0.83
	1	0.71	0.75	0.79
5	0.5	0.87	0.91	0.93
	0.75	0.81	0.86	0.89
	1	0.78	0.82	0.85

Table 11.5 Representative Values of Poisson's Ratio

Type of soil	Poisson's ratio, μ_s
Loose sand	0.2–0.4
Medium sand	0.25–0.4
Dense sand	0.3–0.45
Silty sand	0.2–0.4
Soft clay	0.15–0.25
Medium clay	0.2–0.5

The elastic settlement of a *rigid foundation* can be estimated as: $S_{e(\text{rigid})} \approx 0.93 S_{e(\text{flexible, center})}$

Note:

Due to the nonhomogeneous nature of soil deposits, the magnitude of E_s may vary with depth. For that reason, Bowles (1987) recommended using a weighted average value of E_s *as:*

$$E_s = \frac{\Sigma E_{s(i)} \Delta z}{\bar{z}}$$

Where $E_{s(i)}$ = soil modulus of elasticity within a depth Δz

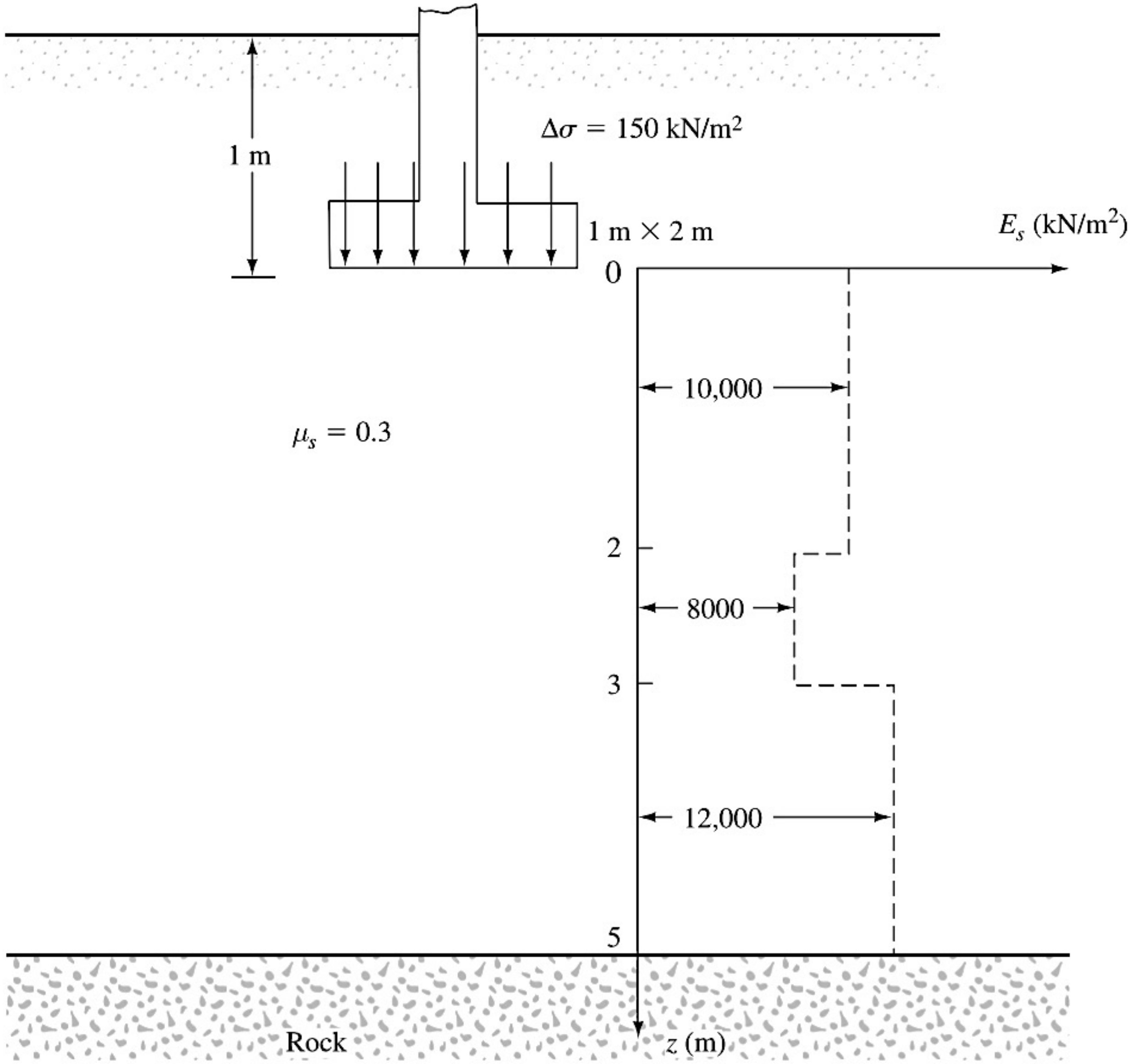
$\bar{z} = H$ or $5B$, whichever is smaller

Table 11.4 Representative Values of the Modulus of Elasticity of Soil

Soil type	E_s kN/m²	E_s lb/in.²
Soft clay	1,800–3,500	250–500
Hard clay	6,000–14,000	850–2,000
Loose sand	10,000–28,000	1,500–4,000
Dense sand	35,000–70,000	5,000–10,000

Example 1

A rigid shallow foundation 1mx 2m is shown in Figure. Calculate the elastic settlement at the center of the foundation.



Solution

Given: $B = 1$ m and $L = 2$ m. Note that $\bar{z} = 5 \text{ m} = 5B$. From Eq.

$$E_s = \frac{\Sigma E_{s(i)} \Delta z}{\bar{z}}$$

$$= \frac{(10{,}000)(2) + (8{,}000)(1) + (12{,}000)(2)}{5} = 10{,}400 \text{ kN/m}^2$$

For the *center of the foundation*,

$$\alpha = 4$$

$$m' = \frac{L}{B} = \frac{2}{1} = 2$$

$$n' = \frac{H}{\left(\frac{B}{2}\right)} = \frac{5}{\left(\frac{1}{2}\right)} = 10$$

From Tables 11.1 and 11.2, $F_1 = 0.641$ and $F_2 = 0.031$.

$$I_s = F_1 + \frac{2 - \mu_s}{1 - \mu_s} F_2$$
$$= 0.641 + \frac{2 - 0.3}{1 - 0.3}(0.031) = 0.716$$

Again, $\dfrac{D_f}{B} = \dfrac{1}{1} = 1, \dfrac{L}{B} = 2, \mu_s = 0.3$. From Table 11.3, $I_f = 0.71$. Hence,

$$S_{e(\text{flexible})} = \Delta\sigma(\alpha B') \frac{1 - \mu_s^2}{E_s} I_s I_f$$
$$= (150)\left(4 \times \frac{1}{2}\right)\left(\frac{1 - 0.3^2}{10{,}400}\right)(0.716)(0.71) = 0.0133\,\text{m} = 13.3\,\text{mm}$$

Since the foundation is rigid, from

$$S_e(\text{rigid}) = (0.93)(13.3) = \mathbf{12.4\ mm}$$

Go a head to the next lecture

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

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Soil Investigation (4)

- ❑ Plate Load Test
- ❑ Some of soil investigation reports:
  - ✓ Soil investigation of 400 beds hospital in Samawa
  - ✓ others

Plate load test

load-test small steel plates of diameters from 0.3 to 0.75 m or squares of side 0.3 X 0.3 and perhaps 0.6 X 0.6 m

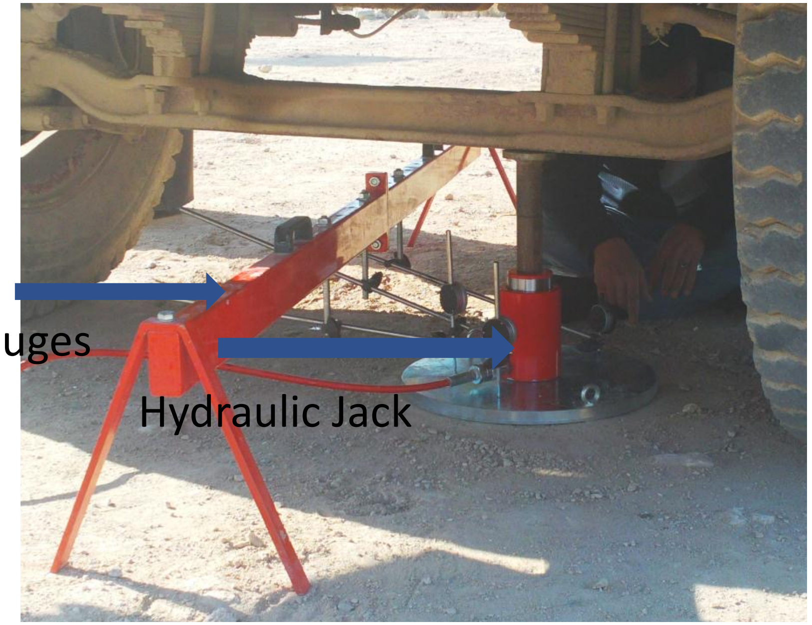
Different sizes of plate, type circular

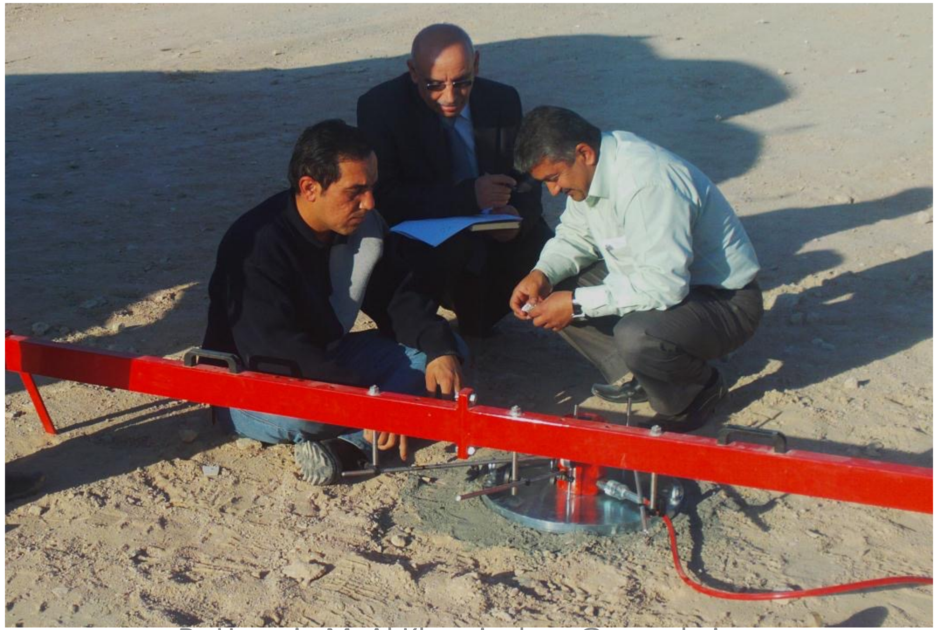
Pressure (Load)gauge

Hydraulic manual pump

Holder of settlement gauges

Hydraulic Jack





The procedure has been standardized as ASTM D 1194, which is essentially as follows:

1. Decide on the type of load application. If it is to be a reaction against piles, they should be driven or installed first to avoid excessive vibration and loosening of the soil in the excavation where the load test will be performed.

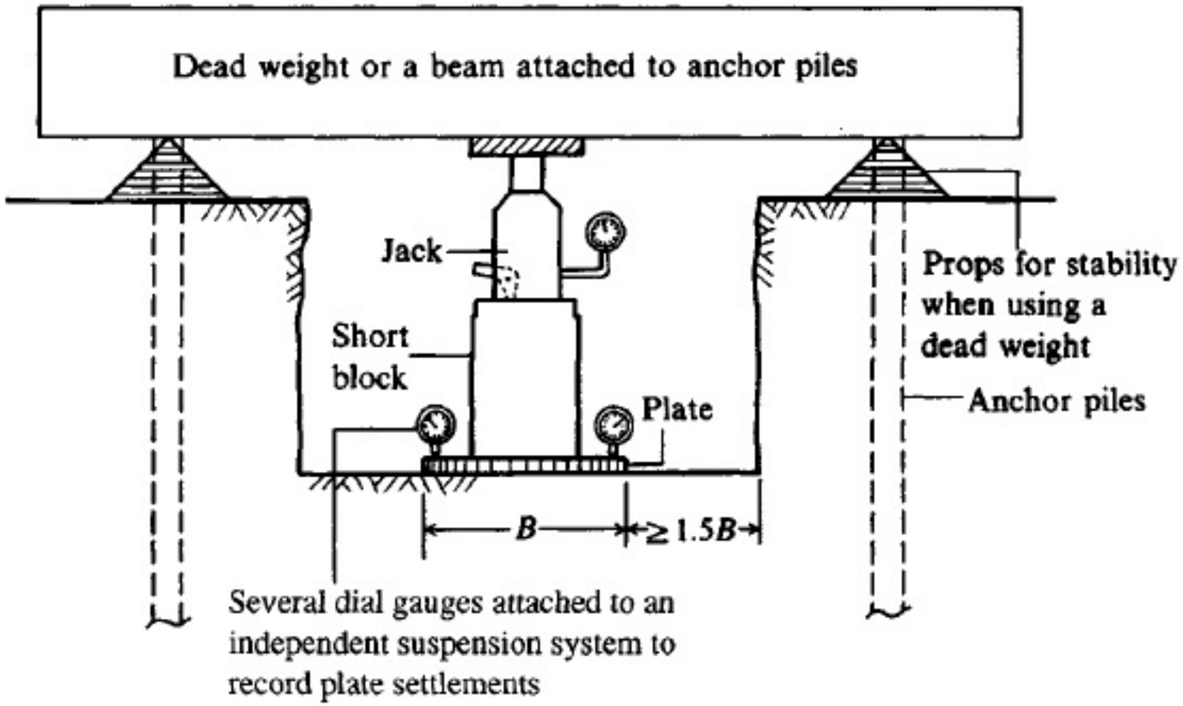
2. Excavate a pit to the depth the test is to be performed. The test pit should be at least four times as wide as the plate and to the depth the foundation is to be placed. If it is specified that three sizes of plates are to be used for the test, the pit should be large enough so that there is an available spacing between tests of 3D of the largest plate.

3. A load is placed on the plate, and settlements are recorded from a dial gauge accurate to 0.25 mm. Observations on a load increment should be taken until the rate of settlement is beyond the capacity of the dial gauge. Load increments should be approximately one-fifth of the estimated bearing capacity of the soil. Time intervals of loading should not be less than 1 h and should be approximately of the same duration for all the load increments.

4. The test should continue until a total settlement of 25 mm is obtained, or until the capacity of the testing apparatus is reached. After the load is released, the elastic rebound of the soil should be recorded for a period of time at least equal to the time duration of a load increment.

In the following Figure presents the essential features of the load test.

Plate load Test in Process



Method of calculation:

a- The yield point, as shown in Figure (2) is obtained at the intersection of the lines extended from the straightest initial and final portions of stress-settlement curves. From this yield point the ultimate bearing capacity, *q*ult is predicted.

b- Calculate q_{all} at 1/2 yield point load.

c- Estimate δ:

δ = corrected settlement = observed settlement - δ_c

δ_c = to be estimated by backward projection of arithmetic load-settlement curve to zero load (if any).

d- Calculate the modulus of subgrade reaction, K_s as : $K_s = q_{all}/\delta$, kN/m^3 , where, q_{all} *is the allowable bearing load of soil.*

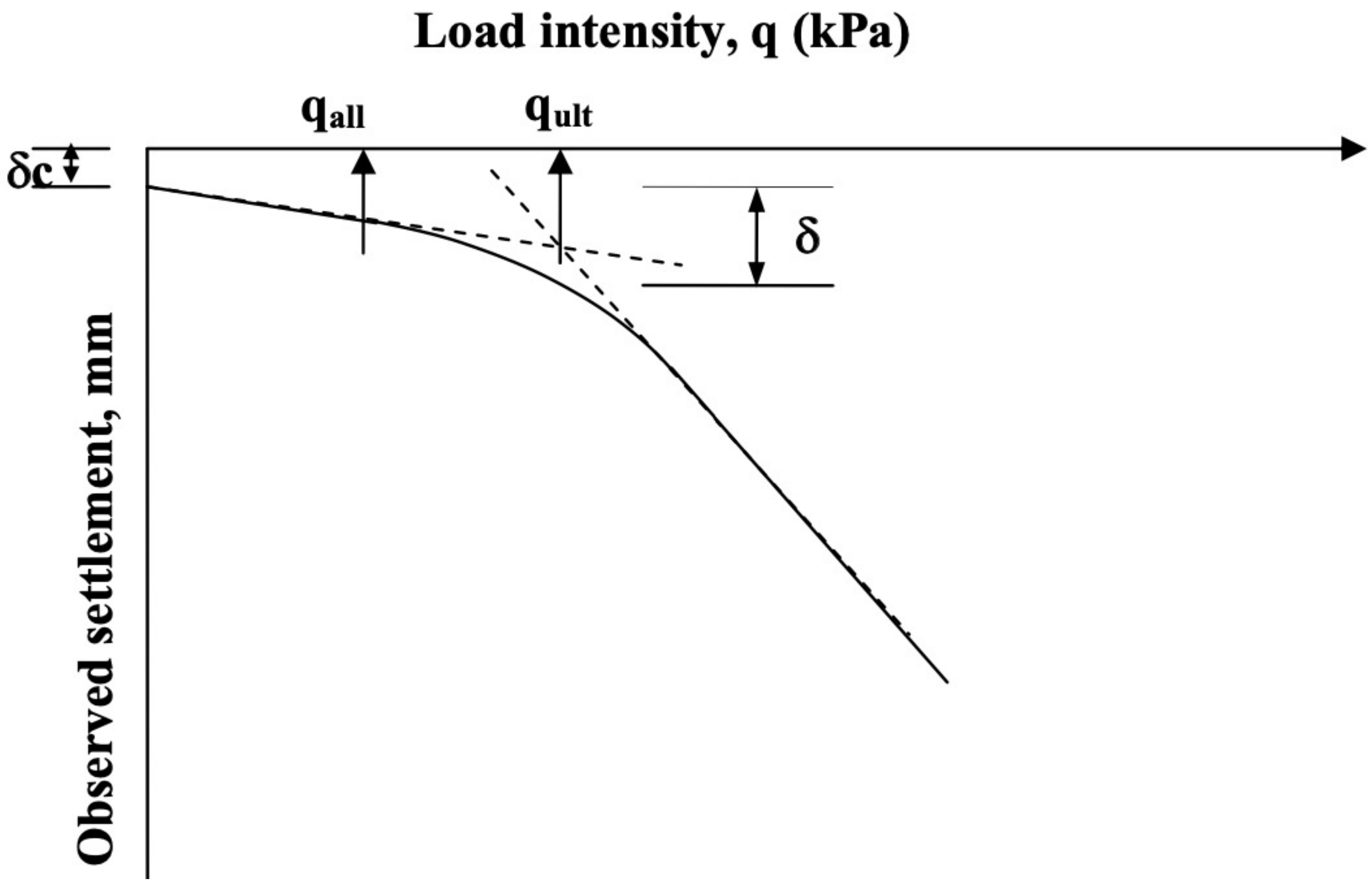
e- Calculate the modulus of deformation, E as: $E = 1.5\ R\ K_s$, where, R is the radius of the plate.

f- The allowable bearing capacity may be calculated by dividing the the minimum value of ultimate bearing capacity of the soil that estimated from the following two approaches by a factor of safety of 3:

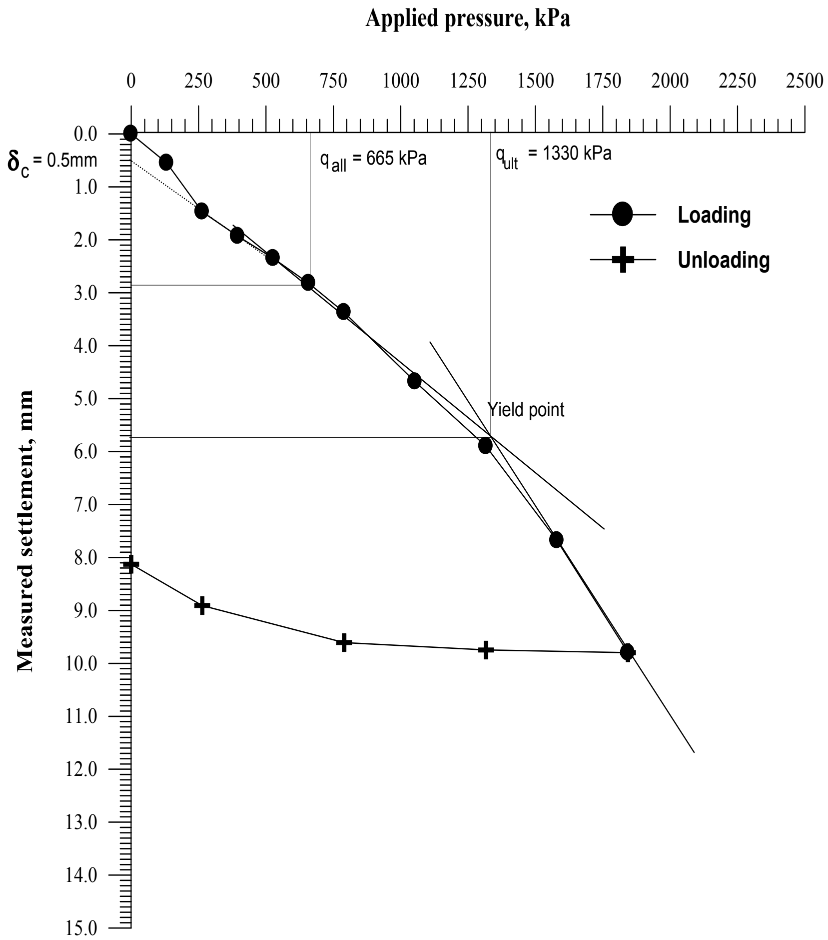
(1) The approach explained in (a) above.

(2) The plate stress which gives a settlement (Sp) corresponding to allowable settlement of the actual footing (Sf) that can be calculated from the following equation: $S_p = S_f\left(\frac{B_p(B_f+30)}{B_f(B_p+30)}\right)^2$

where, S_p = plate settlement corresponding to the actual footing settlement (mm). S_f = allowable settlement of actual footing (mm).B_p = plate diameter (cm)B_f = width of footing (cm) .



Procedure of calculation



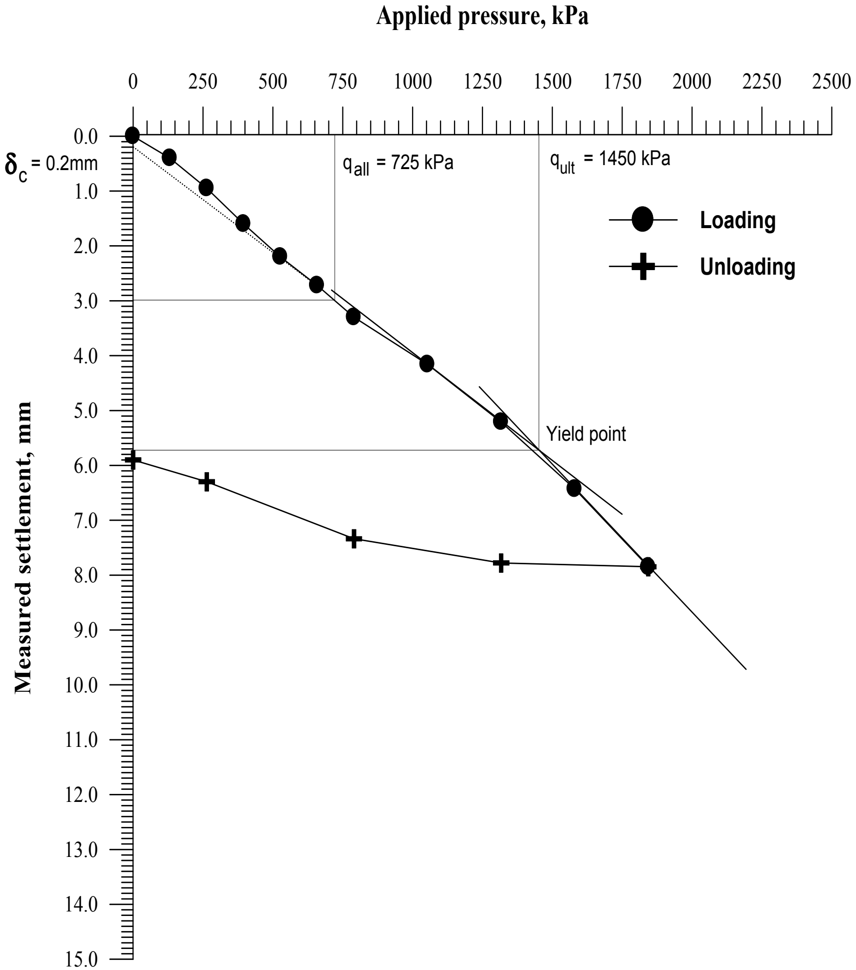
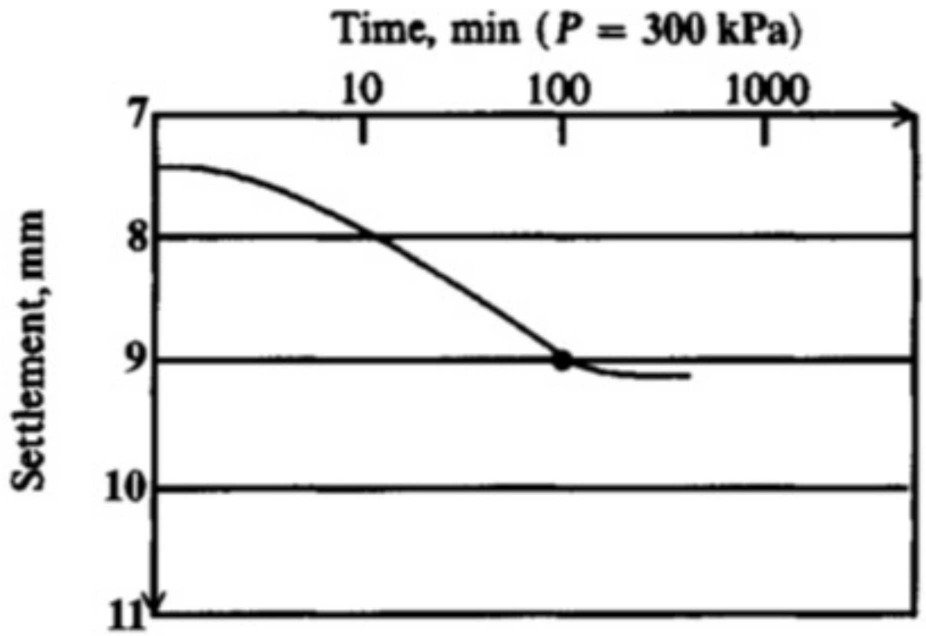
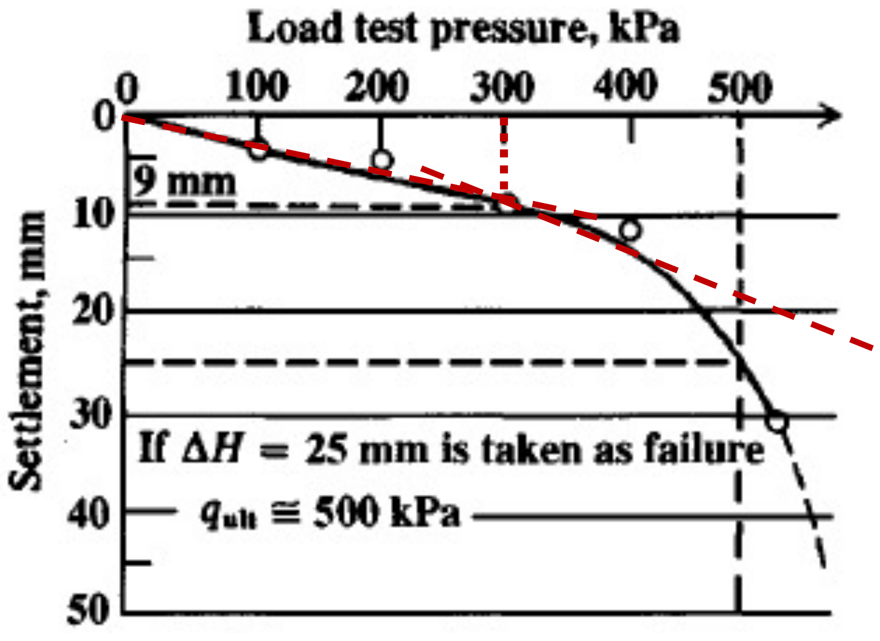


Plate Load Tests Results

point No.	Location	Estimated Allowable Bearing Capacity, q_{all} (kPa)	Modulus of Sub grade reaction, Ks (kN/m^3)	Modulus of Deformation, E (MPa)
1		443	282979	64.7
2		483	258929	59.2

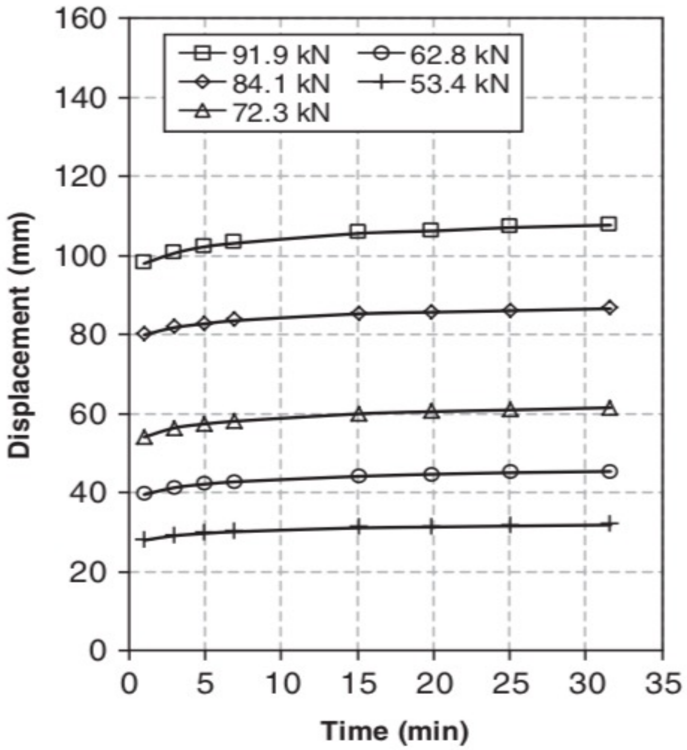


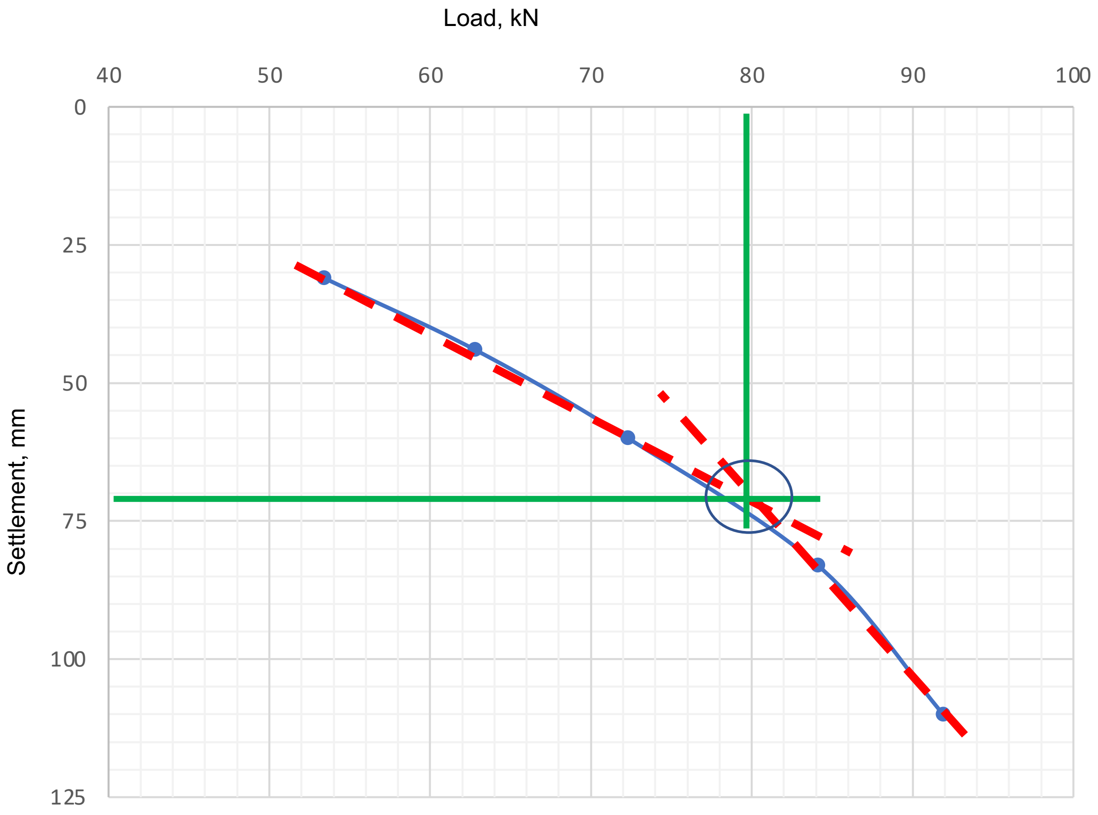
a) Plot of settlement vs. log time to determine the maximum settlement for a load increment (300 kPa in this case)



Load vs. settlement plot to establish the maximum design pressure.

Example (1) : For the data obtained from executing a load test on a soil, drawn in the Fig., shown below, using a plate of diameter 0.5 m, tabulate the settlement against each load and then draw the pressure-settlement curve to determine the expected bearing capacity in kPa, where the permissible settlement is 25 mm.





Load,kN	Settlement, mm
53.4	31
62.8	44
72.3	60
84.1	83
91.9	110

Load = 80 kN, Settlement = 69 mm

So, load for 25 mm =80*25/69 = 29 kN

Bearing capacity of soil = Load/Area of plate = 29/(π*0.5^2/4) = 145.9 kPa = 147.7 kPa.

- Extrapolating load-test results to full-size footings is not standard. For clay soils it is common to note that the $BN\gamma$ term is zero, so that one *might* say that q_u is independent of footing size.

$$q_{ult,\,foundation} = q_{ult,\,load\ test}$$

- In cohesionless (and *(ϕ - c)* soils all three terms of the bearing-capacity equation apply and, noting that the $N\gamma$ term includes the footing width, one *might* say

$$q_{ult,\,foundation} = M + N\frac{B_{foundation}}{B_{load\ test}}$$

- where M includes the Nc and Nq terms and N is the Nγ term. By using several sizes of plates this equation can be solved graphically for q_u. Practically, for extrapolating plate load tests for sands (which are often in a configuration so that the Nq term is negligible), use the following:

$$q_{ult} = q_{plate}\left(\frac{B_{foundation}}{B_{plate}}\right)$$

Review some soil investigation reports

Report for project : 400 beds hospital in Samawa, 2012:

It will be some oral questions to the students for review the main topic of this topic(Soil Investigation).


Thank
You

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

Instructor: Professor Dr. Hussein M. Al.Khuzaie (Ph.D., Civil Engineering, Foundation Engineering and Structures);

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Soil Investigation (2)

- Sampling Methods
  - Types of Samples
  - SPT
    - Description
    - Recovery Ratio
    - Corrections
  - Thin-walled sampler (Shelby Tube)

Sampling

- Two types of soil samples can be obtained during subsurface exploration: *disturbed* and *undisturbed*.

The following lab. tests carried out on disturbed samples:

1. Grain-size analysis.

1. Determination of liquid and plastic limits.

3. Specific gravity of soil solids.

4. Determination of organic content.

5. Classification of soil.

- The undisturbed samples are needed for the following tests:

1. Permeability test.
2. Shear strength tests.
3. Consolidation tests.

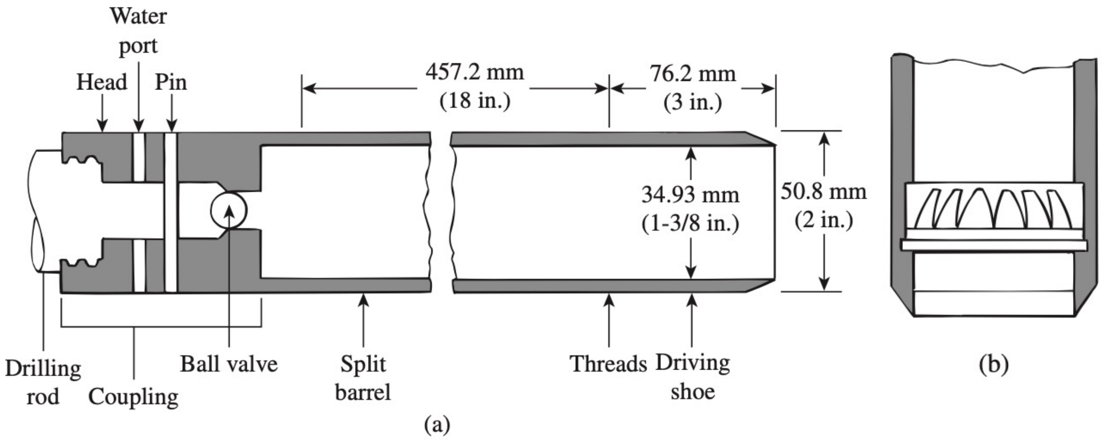
❑ Split-Spoon Sampler (SS)

The tool consists of :

✓ a steel driving shoe

✓ a steel tube that is split longitudinally in half, and a coupling at the top. The coupling connects the sampler to the drill rod.

The standard split tube has an inside diameter of 34.93 mm and an outside diameter of 50.8 mm, however,
samplers having inside and outside diameters up to 63.5 mm and 76.2 mm, respectively, are also available.



(a) Standard split-spoon sampler; (b) spring core catcher

Sampling by SS

❖When a borehole is extended to a predetermined depth:

➢The drill tools are removed and the sampler is lowered to the bottom of the hole.

➢The sampler is driven into the soil by hammer blows to the top of the drill rod. The standard weight of the hammer is 622.72 N (63.48 Kg) (140 lb), and for each blow, the hammer drops a distance of 0.762 m (30 in.).

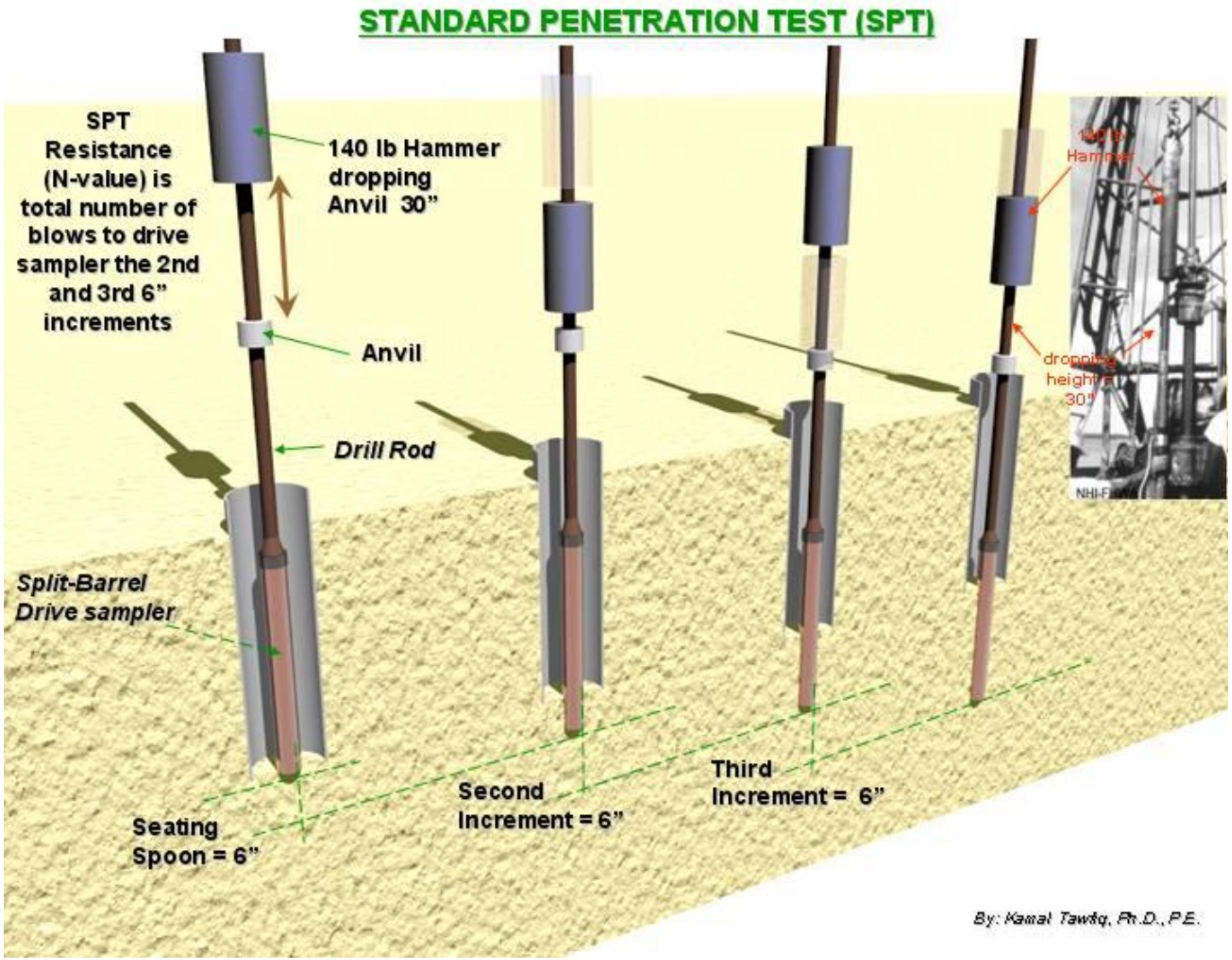
➢The number of blows required for a spoon penetration of three 152.4-mm (6-in.) intervals are recorded.

➢The number of blows required for the last two intervals are added to give the standard penetration number, N, at that depth. This number is generally referred to as the N value (American Society for Testing and Materials, 2014, Designation D-1586-11).

➢The sampler is then withdrawn, and the shoe and coupling are removed. Finally, the soil sample recovered from the tube is placed in a glass bottle and transported to the laboratory.

➢This field test is called the standard penetration test (SPT).

Penetration Intervals of Split Spoon Sampler (SS)

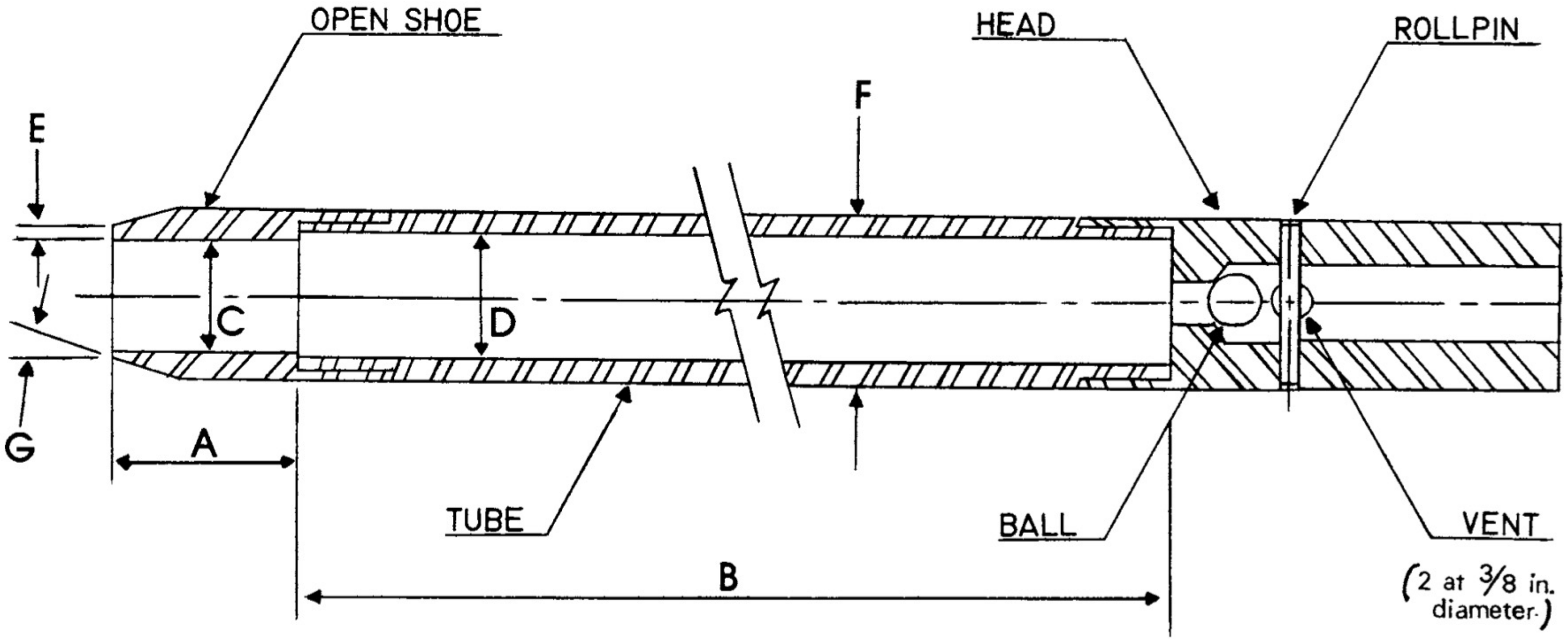


Sampling by SS

Designation: D1586 – 11

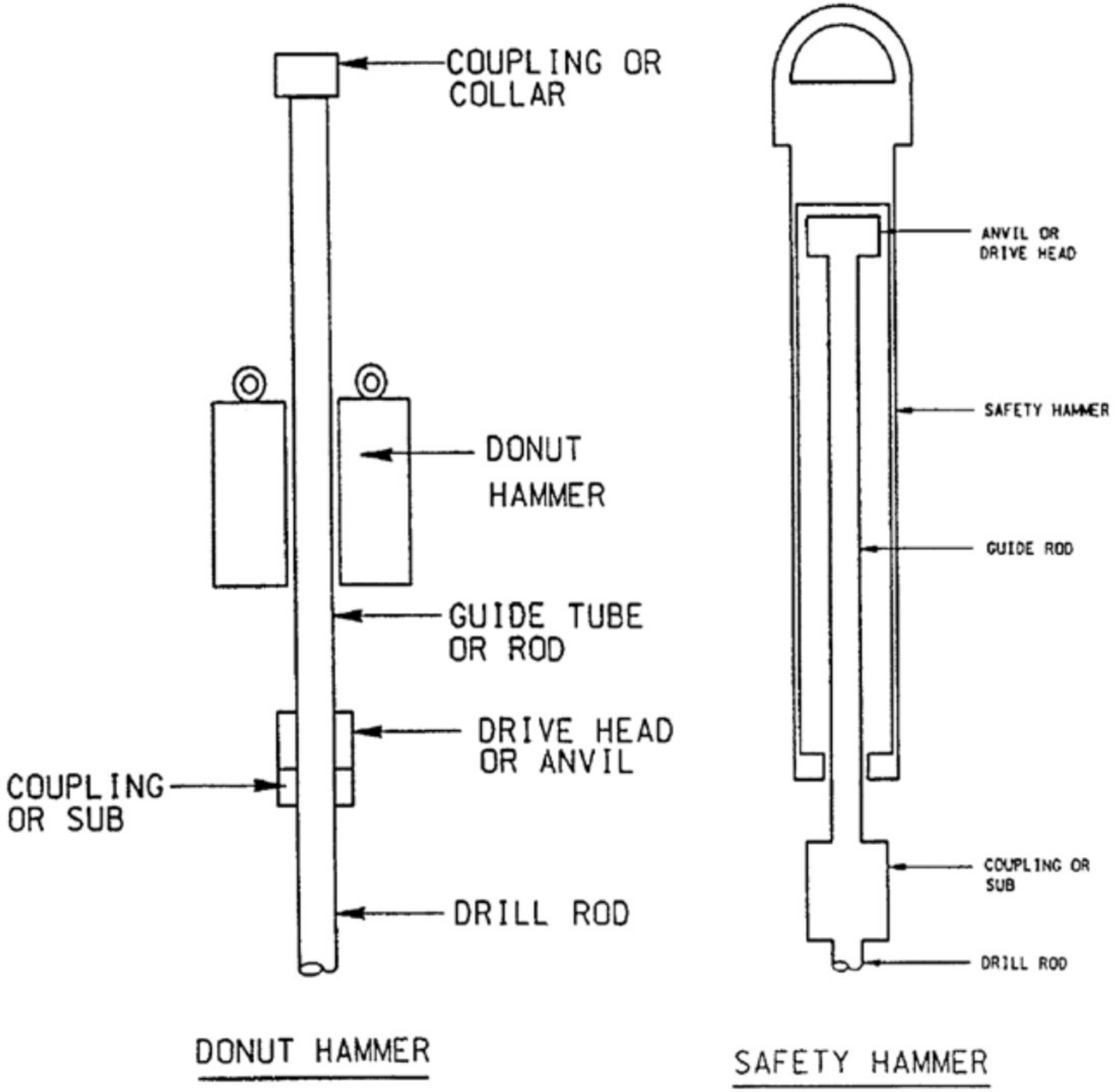
Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils[1]

This standard is issued under the fixed designation D1586; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.



A = 1.0 to 2.0 in. (25 to 50 mm)
B = 18.0 to 30.0 in. (0.457 to 0.762 m)
C = 1.375± 0.005 in. (34.93 ± 0.13 mm)
D = 1.50 ± 0.05 – 0.00 in. (38.1 ± 1.3 – 0.0 mm)
E = 0.10 ± 0.02 in. (2.54 ± 0.25 mm)
F = 2.00 ± 0.05 – 0.00 in. (50.8 ± 1.3 – 0.0 mm)
G = 16.0° to 23.0°

Hammer for driving the split barrel sampler



Sampling by SS

The degree of disturbance for a soil sample is usually expressed as:

$$A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2}(100)$$

where

A_R = area ratio (ratio of disturbed area to total area of soil)
D_o = outside diameter of the sampling tube
D_i = inside diameter of the sampling tube

When the area ratio is 10% or less, the sample generally is considered to be undisturbed.

For a standard split-spoon sampler,

$$A_R(\%) = \frac{(50.8)^2 - (34.93)^2}{(34.93)^2}(100) = 111.5\%$$

Is this sample disturbed or not, if it is, why?

Sampling by SS

Example:

Calculate the area ratio for the sampler (its outside diameter (D_o = 76.2 mm and the inside diameter D_i = 72.9 mm) and for the split spoon sampler (its outside diameter = 50.8 mm and the inside diameter = 34.9 mm).

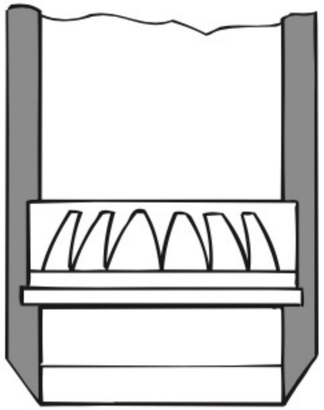
$$A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2}(100)$$

$$A_R\ (\%) = \frac{D_o^2 - D_i^2}{D_i^2}\ (100) = \frac{76.2^{2} - 72.9^2}{72.9^2}\ (100) = 9.3\%$$

Is this sample disturbed or not, if it is, why?

Sampling by SS

- Split-spoon samples generally are taken at intervals of about 1.5 m (5 ft).
- When the material encountered in the field is sand (particularly fine sand below the water table), recovery of the sample by a split-spoon sampler may be difficult. In that case, a device such as a spring core catcher may have to be placed inside the split spoon (Figure).



Spring Core Catcher

Energy Efficiency of Hammer of SPT

The SPT hammer energy efficiency can be expressed as:

$$E_r(\%) = \frac{\text{actual hammer energy to the sampler}}{\text{input energy}} \times 100 \qquad (3.4)$$

$$\text{Theoretical input energy} = Wh \qquad (3.5)$$

where

W = weight of the hammer $\approx$ 0.623 kN (140 lb)
h = height of drop $\approx$ 0.76 mm (30 in.)

So,

$$Wh = (0.623)(0.76) = 0.474 \text{ kN-m } (4200 \text{ in.-lb})$$

Energy Efficiency of Hammer of SPT

In practice, the efficiency of hammer energy taken as average to be 60%, so the N value obtained in the field may standardized to this average by considering correction factors:

1) SPT hammer efficiency (η_H) ,
2) Borehole diameter (η_B),
3) Sampling method (η_S), and
4) Rod length (η_R)

$$N_{60} = \frac{N\eta_H \eta_B \eta_S \eta_R}{60}$$

Energy Efficiency of Hammer of SPT

Table 3.5 Variations of η_H, η_B, η_S, and η_R [Eq. (3.6)]

1. Variation of η_H

Country	Hammer type	Hammer release	η_H (%)
Japan	Donut	Free fall	78
	Donut	Rope and pulley	67
United States	Safety	Rope and pulley	60
	Donut	Rope and pulley	45
Argentina	Donut	Rope and pulley	45
China	Donut	Free fall	60
	Donut	Rope and pulley	50

2. Variation of η_B

Diameter		
mm	in.	η_B
60–120	2.4–4.7	1
150	6	1.05
200	8	1.15

3. Variation of η_S

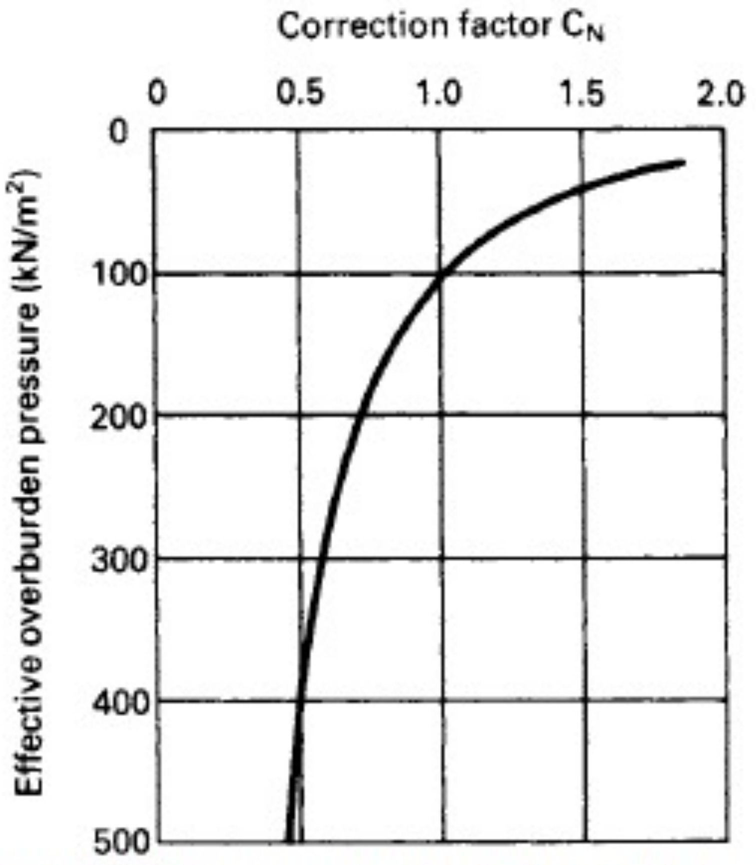
Variable	η_S
Standard sampler	1.0
With liner for dense sand and clay	0.8
With liner for loose sand	0.9

4. Variation of η_R

Rod length		
m	ft	η_R
>10	>30	1.0
6–10	20–30	0.95
4–6	12–20	0.85
0–4	0–12	0.75

Correction of N Value for Field Testing and Overburden Pressure

For geotechnical earthquake engineering, such as liquefaction analyses, the standard penetration test N_{60} value is corrected for the overburden soil pressure, also known as the effective overburden pressure or the vertical effective stress (σ'_{vo}). When a correction is applied to the N_{60} value to account for the vertical effective stress, these values are referred to as $(N_1)_{60}$ values. The procedure consists of multiplying the N_{60} value by a correction C_N in order to calculate the $(N_1)_{60}$ value. The Figure presents a chart that is commonly used to obtain the correction factor C_N. Another option is to use the following equation:



Correction of measured values of standard penetration resistance.

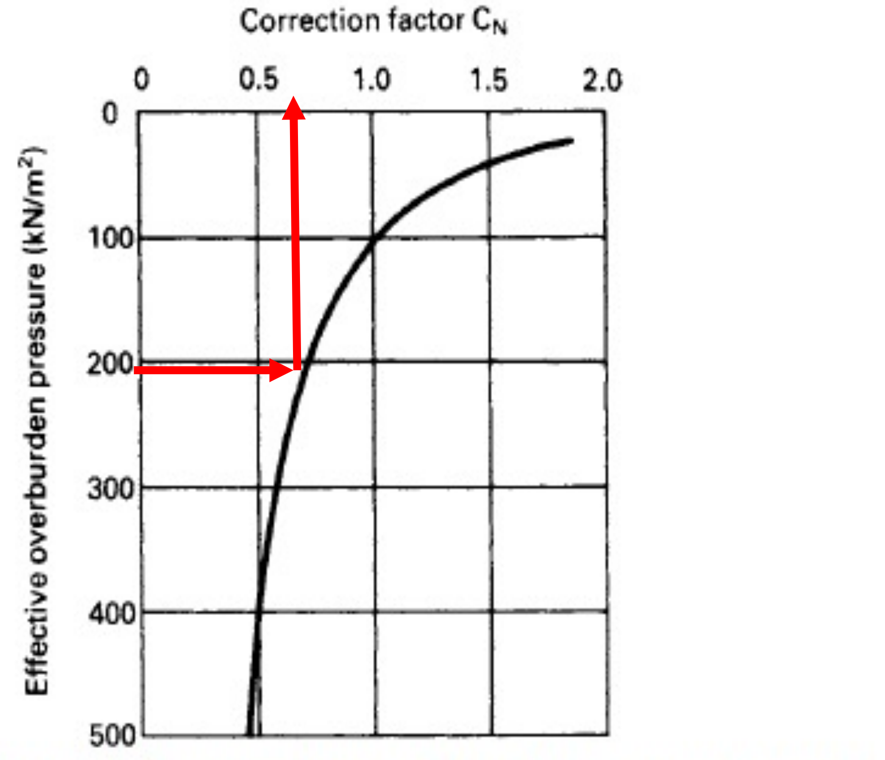
$$(N_1)_{60} = C_N N_{60} = \left(\frac{100}{\sigma'_{vo}}\right)^{0.5} N_{60}$$

Corrections for SPT: Example

Given. N = 20; rod length = 12 m; hole diam. = 150 mm; σ'_{vo} = 205 kPa; use safety hammer with E_r = 80; dense sand; no liner; What are the "standard" N_{60}?

Solution:

$$N_{60} = \frac{N\eta_H \eta_B \eta_S \eta_R}{60}$$



Correction of measured values of standard penetration resistance.

N_{80}= 20, $\eta_H = 0.8$, $\eta_B = 1.05$, $\eta_s = 1$, $\eta_R = 1$

N_{60} = 20 x 80x1.05x1x1 /60 = 28

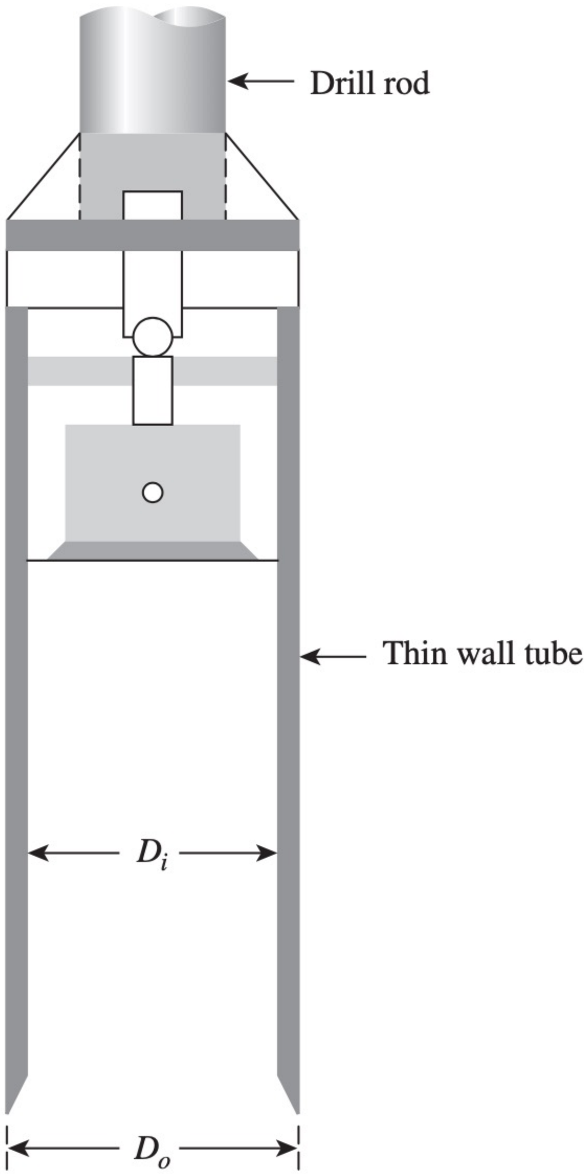
$(N_1)_{60} = C_N \times N_{60}$ = 0.7 x 28 = 19.6

or by the equation:

$$C_N = \sqrt{\left(\frac{100}{\sigma'_{vo}}\right)} = \sqrt{\frac{100}{205}} = 0.7$$

Thin-Wall Tube Sampler (Shelby Tube)

Sampling by thin-wall tube is used for obtaining fairly undisturbed soil samples. The thin- wall tubes are made of seamless, thin tubes and commonly are referred to as Shelby tubes (show the Figure). To collect samples at a given depth in a borehole, one first must remove the drilling tools. The sampler is attached to a drilling rod and lowered to the bottom of the bore- hole. After this, it is pushed hydraulically into the soil. It then is spun to shear off the base and is pulled out. The sampler with the soil inside is sealed and taken to the laboratory for testing. Most commonly used thin-wall tube samplers have outside diameters of 76.2 mm (3 in.).



Summary

The following points were presented:

- Types of samples to be extracted from the soil at the site.
- Sampling and types of samplers

1) Split spoon sampler (SS)

✓ Standard Penetration Test (SPT)

✓ Corrections

✓ $(N_1)60$

2) Thin walled sampler (Shelby Tube)


Thank
You

Foundation Engineering (1), 4th stage, Civil Engineering Dept., College of Engineering, Al-Muthanna University, 2020-2021

Instructor: Professor Dr. Hussein M. Al.Khuzaie (Ph.D., Civil Engineering, Foundation Engineering and Structures);

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Soil Investigation (3)

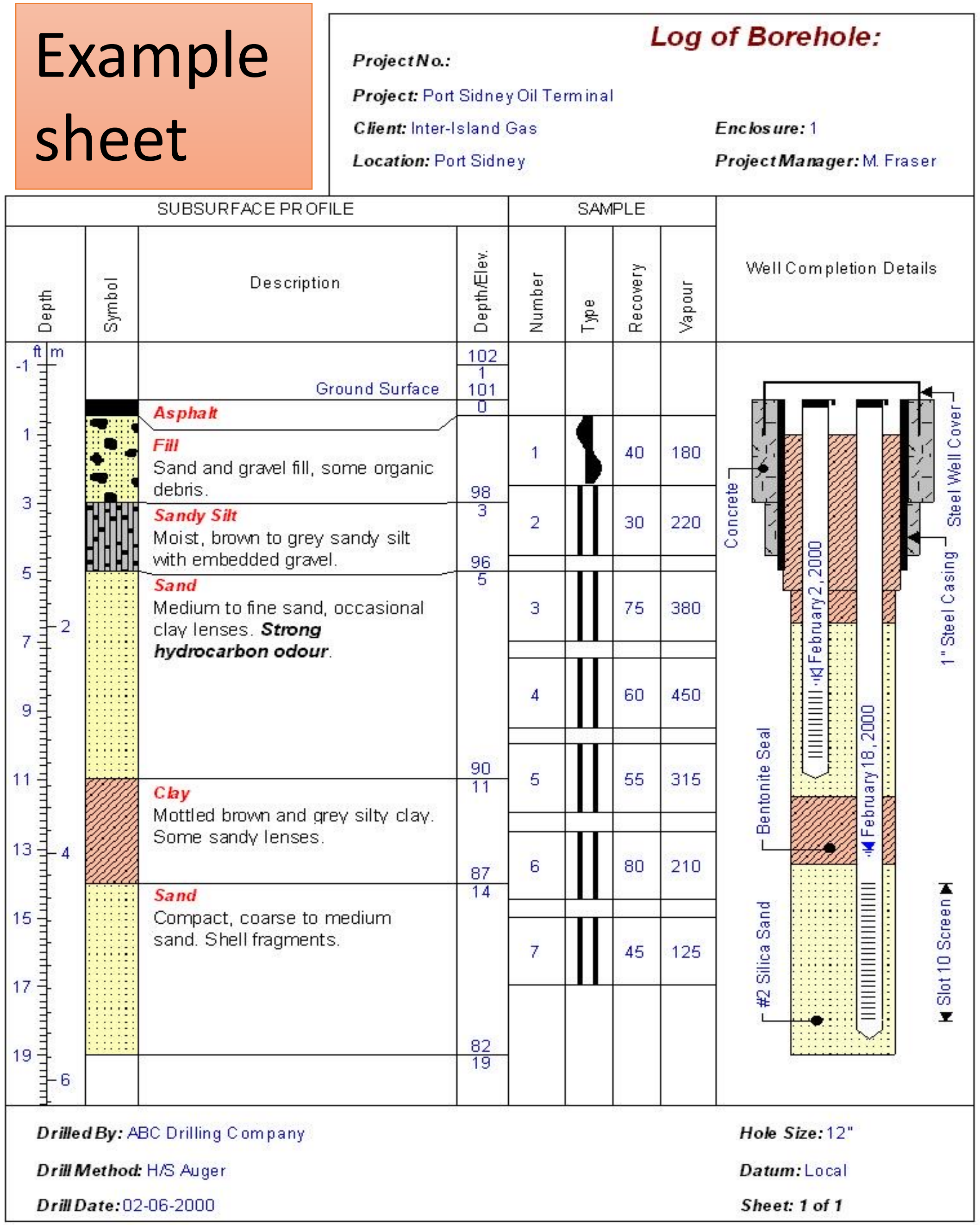
- Log of Boring
- Soil Sampling
- Laboratory Tests
- Rock Sampling
- Recovery ratio
- RQD
- Ground water table level
- Field Tests

Preparation of Boring Logs

1. Name and address of the drilling company
2. Driller’s name
3. Job description and number
4. Number, type, and location of boring
5. Date of boring
6. Subsurface stratification, which can he obtained by visual observation of the soil brought out by auger, split-spoon sampler, and thin-walled Shelby tube sampler
7. Elevation of water table and date observed, use of casing and mud losses, and so on
8. Standard penetration resistance and the depth of SPT
9. Number, type, and depth of soil sample collected
10. In case of rock coring, type of core barrel used and, for each run, the actual length of coring, length of core recovery, and RQD

Boring Logs

Example sheet



Log of Borehole:

Project No.:

Project: Port Sidney Oil Terminal

Client: Inter-Island Gas

Enclosure: 1

Location: Port Sidney

Project Manager: M. Fraser

SUBSURFACE PROFILE				SAMPLE				Well Completion Details
Depth (ft / m)	Symbol	Description	Depth/Elev.	Number	Type	Recovery	Vapour	
		Ground Surface	102 / 1 / 101 / 0					
		Asphalt						
		Fill Sand and gravel fill, some organic debris.	98 / 3	1		40	180	
		Sandy Silt Moist, brown to grey sandy silt with embedded gravel.	96 / 5	2		30	220	
		Sand Medium to fine sand, occasional clay lenses. ***Strong hydrocarbon odour***.	90 / 11	3		75	380	
				4		60	450	
				5		55	315	
		Clay Mottled brown and grey silty clay. Some sandy lenses.	87 / 14	6		80	210	
		Sand Compact, coarse to medium sand. Shell fragments.	82 / 19	7		45	125	

Drilled By: ABC Drilling Company

Hole Size: 12"

Drill Method: H/S Auger

Datum: Local

Drill Date: 02-06-2000

Sheet: 1 of 1

Test Hole No. 1

Date 96/6/1

Drilling Method Auger

Depth to water:
immediate 1.2 m

24 hours 1.0 m

Depth (m)	Soil Description	Sample			SPT Value "N"
		No.	Type	Depth (m)	
0	Topsoil, sandy (0–0.3 m)				
0.3					
0.5	Loose brown moist fine <u>sand</u> (0.3–1.2 m)	1	Auger	0.8	
1.0					
1.2	Dense silty sand 1.2–1.8 m	2	Split spoon	1.2–1.6	8/12 = 20
1.5					
1.8					
2.0	<u>Till</u>, clayey, with some silt, moist, hard (1.8–2.8 m)	3	Split spoon	2.1–2.5	15/18 = 33
2.5					
2.8					
3.0	<u>Clay</u>, soft wet (2.8–3.6 m)	4	Shelby	3.0–3.6	
3.5					
3.6	End of test hole				

Figure 2–7 Typical field notes.

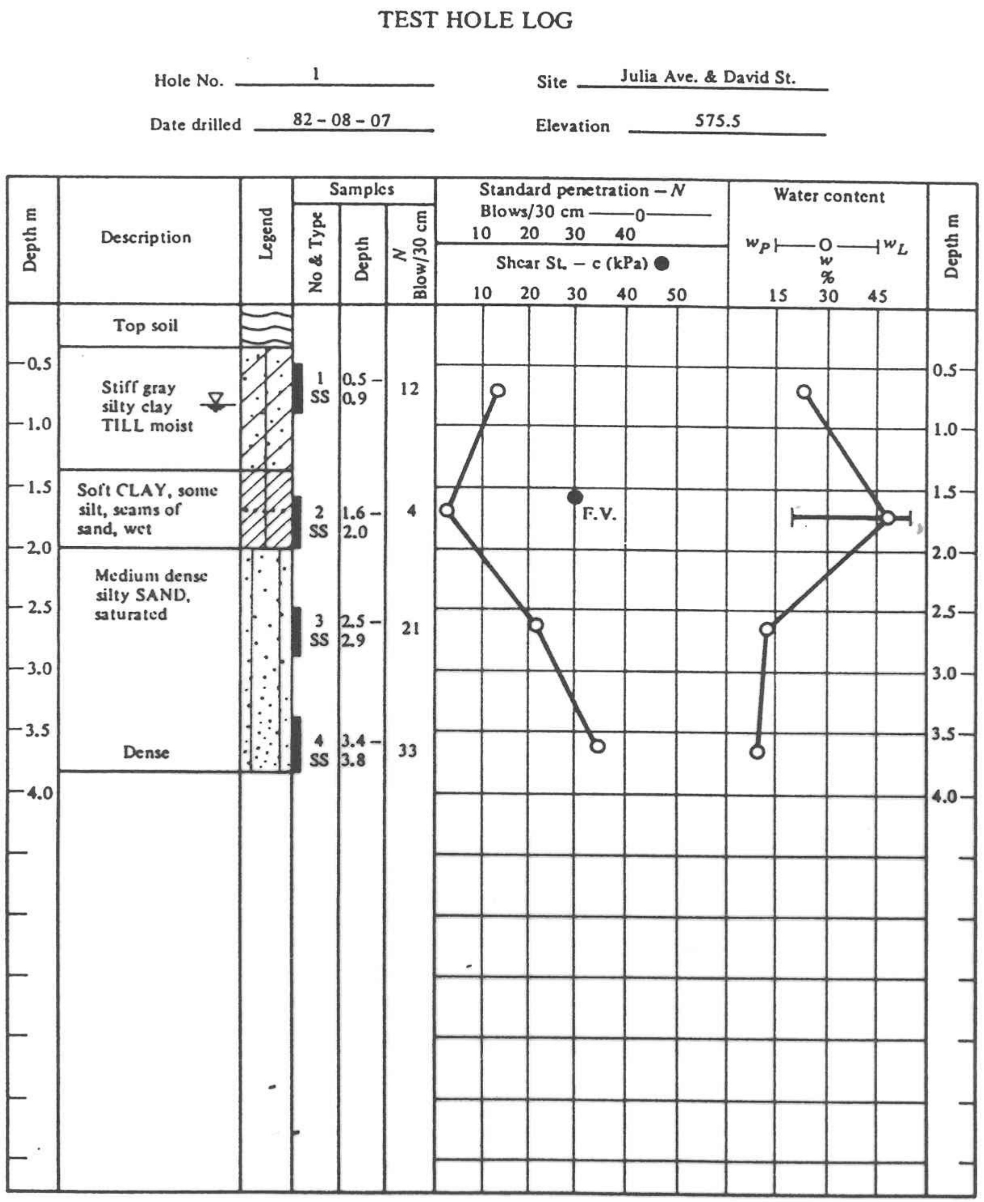


Figure 2–10 Test-hole log.

Boring Log

Name of the Project: Two-story apartment building

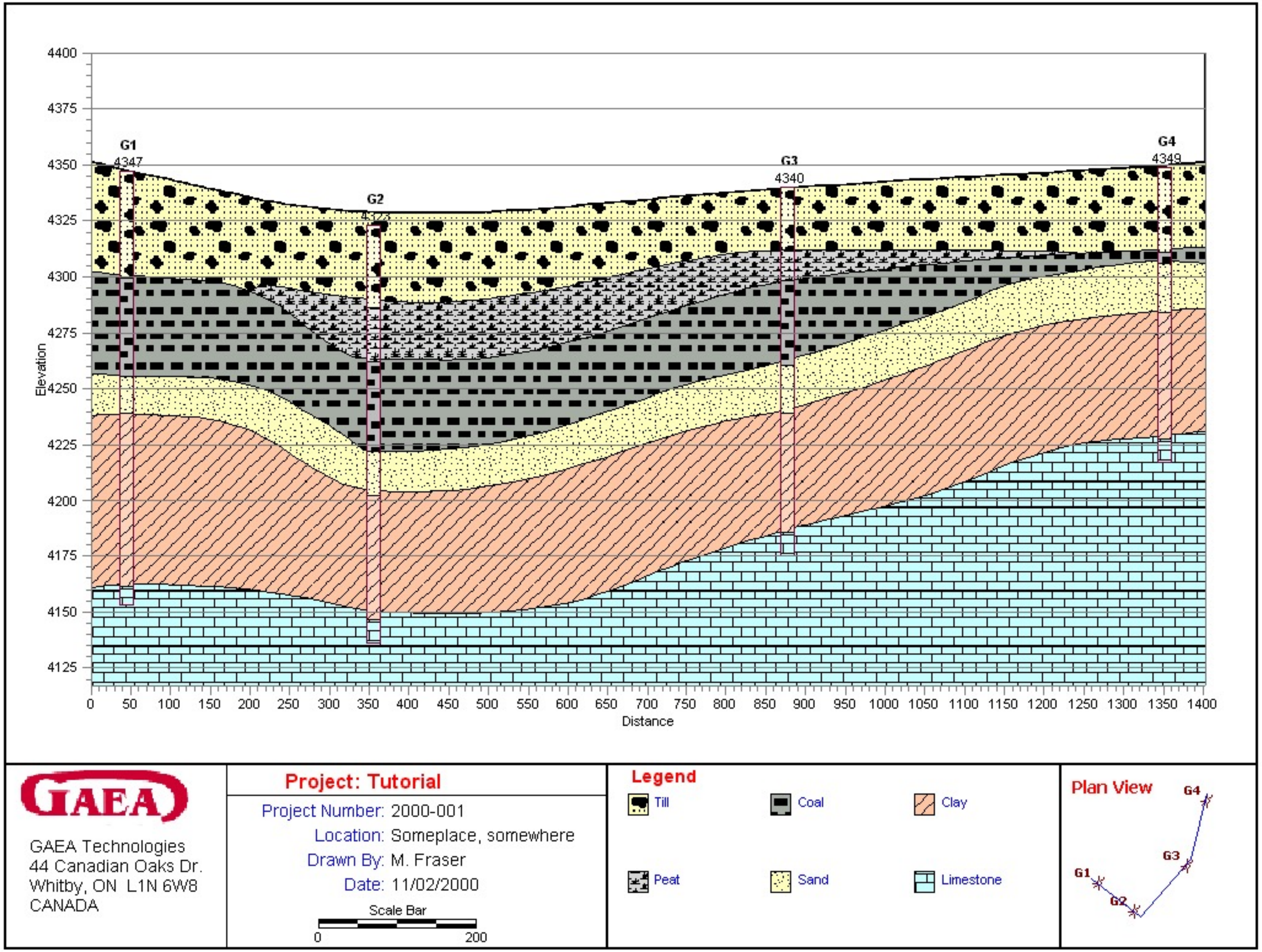
Location: Johnson & Olive St. Date of Boring: March 2, 1982

Boring No. 3 Type of Boring: Hollow stem auger Ground Elevation: 60.8 m

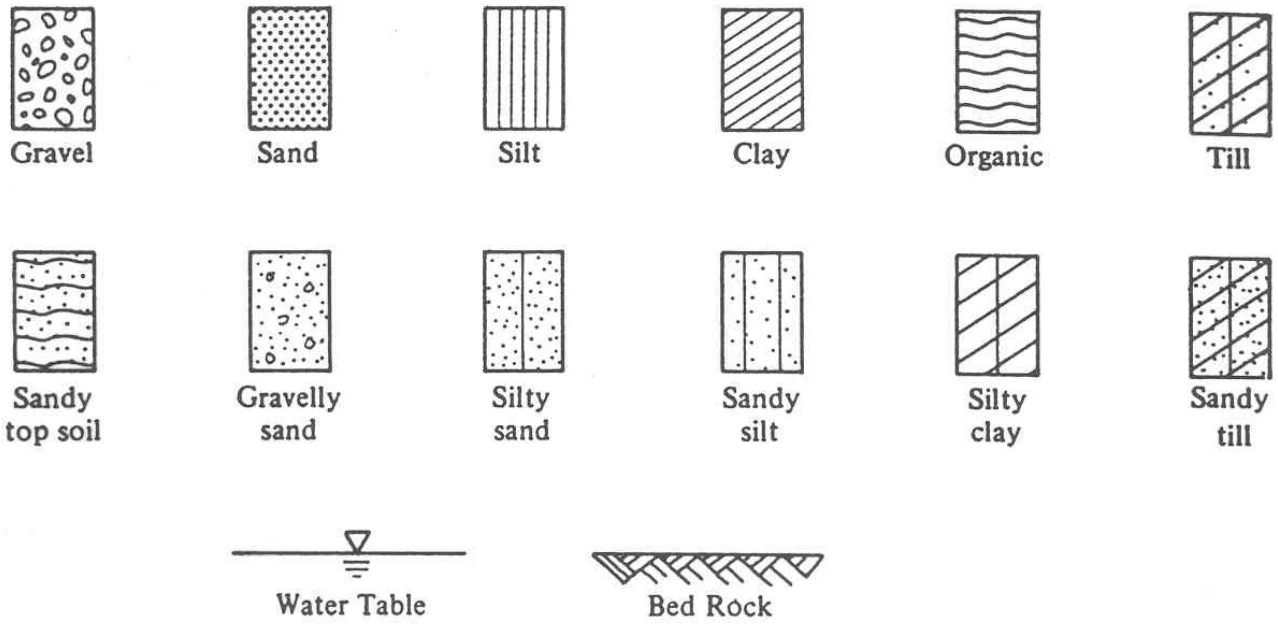
Soil description	Depth (m)	Soil sample type and number	N	w_n (%)	Comments
Light brown clay (fill)					
Silty sand (SM)	1				
		SS-1	9	8.2	
	2				
	3				
*G.W.T. 3.5 m		SS-2	12	17.6	$LL = 38$ $PI = 11$
	4				
Light gray silty clay (ML)		ST-1		20.4	$LL = 36$ $q_u = 112\ \text{kN/m}^2$
	5				
	6				
		SS-3	11	20.6	
Sand with some gravel (SP)	7				
End of boring @ 8 m	8	SS-4	27	9	

N = standard penetration number (below/304.8 mm)
w_n = natural moisture content
LL = liquid limit; PI = plasticity index
q_u = unconfined compression strength
SS = split-spoon sample; ST = Shelby tube sample

*Ground water table observed after one week of drilling


G1
4347
G2
G3
4340
G4
4349
Elevation
4400
4375
4350
4325
4300
4275
4250
4225
4200
4175
4150
4125
Distance
0 50 100 150 200 250 300 350 400 450 500 550 600 650 700 750 800 850 900 950 1000 1050 1100 1150 1200 1250 1300 1350 1400
GAEA
GAEA Technologies
44 Canadian Oaks Dr.
Whitby, ON L1N 6W8
CANADA
Project: Tutorial
Project Number: 2000-001
Location: Someplace, somewhere
Drawn By: M. Fraser
Date: 11/02/2000
Scale Bar
0
200
Legend
Till
Coal
Clay
Peat
Sand
Limestone
Plan View
G1
G2
G3
G4

Soils Investigation



Type of Sample

S.S. – Split Spoon
S.T. – Shelby Tube
A.S. – Auger Sample
W.S. – Washed Sample
R.C. – Rock Core
V.S. – Vane Shear Sample

Soil Tests

F.V. – Field Vane
L.V. – Lab Vane
Qu – Unconfined Compression
Qq – Undrained (quick) Triaxial
C – Dynamic Cone (Blows per foot)

Typical test-hole symbols and abbreviations.

SOIL SAMPLING

Two types of soil samples can be obtained during sampling disturbed and undisturbed. The most important engineering properties required for foundation design are strength, compressibility, and permeability. Reasonably good estimates of these properties for cohesive soils can be made by laboratory tests on undisturbed samples which can be obtained with moderate difficulty. It is nearly impossible to obtain a truly undisturbed sample of soil; so in general usage the term "undisturbed" means a sample where some precautions have been taken to minimize disturbance or remolding effects. In this context, the quality of an "undisturbed" sample varies widely between soil laboratories.

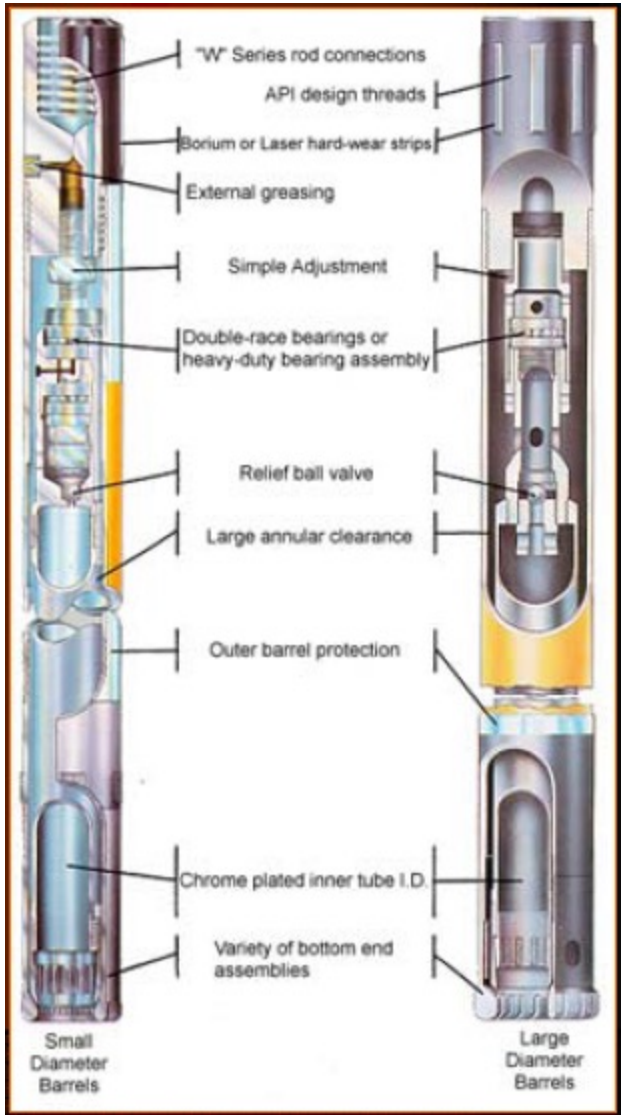
Soils Investigation

Table 2–4
LABORATORY TESTS RELATED TO A SOILS INVESTIGATION

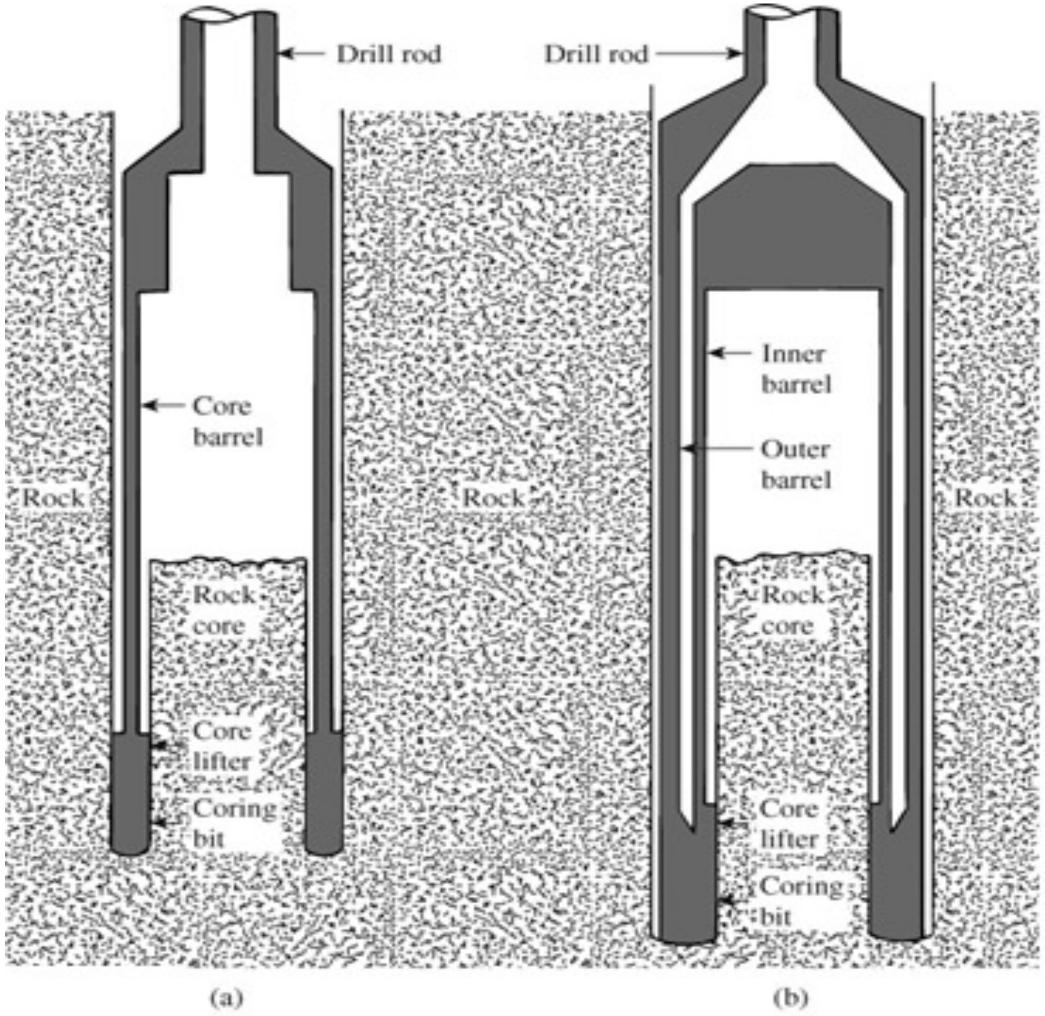
Test	Sample Required: Disturbed or Undisturbed	Sample Required: Undisturbed	Soils: Cohesive	Soils: Granular
Moisture content	X		X	X
Grain size	X		X	X
Atterberg Limits	X		X	
Relative density (specific gravity)	X		X	X
Density (unit weight)		X	X	X
Unconfined compression		X	X	
Triaxial compression		X	X	X
Direct shear		X	X	X
Consolidation		X	X	
Vane shear		X	X	
Permeability		X		X

ROCK SAMPLING

- Rock cores are necessary if the soundness of the rock is to be established.
- small cores tend to break up inside the drill barrel.
- Larger cores also have a tendency to break up (rotate inside the barrel and degrade), especially if the rock is soft or fissured.



Rock coring



Rock coring: (a) single-tube core barrel; (b) double-tube core barrel

ROCK SAMPLING - Definition

$$\text{Recovery Ratio} = \frac{\sum \text{Lengths of intact pieces of core}}{\text{Length of core advance}}$$

$$\text{RQD} = \frac{\sum \text{Lengths of intact pieces of core} \geq 10.16 \text{ cm}}{\text{Length of core advance}}$$

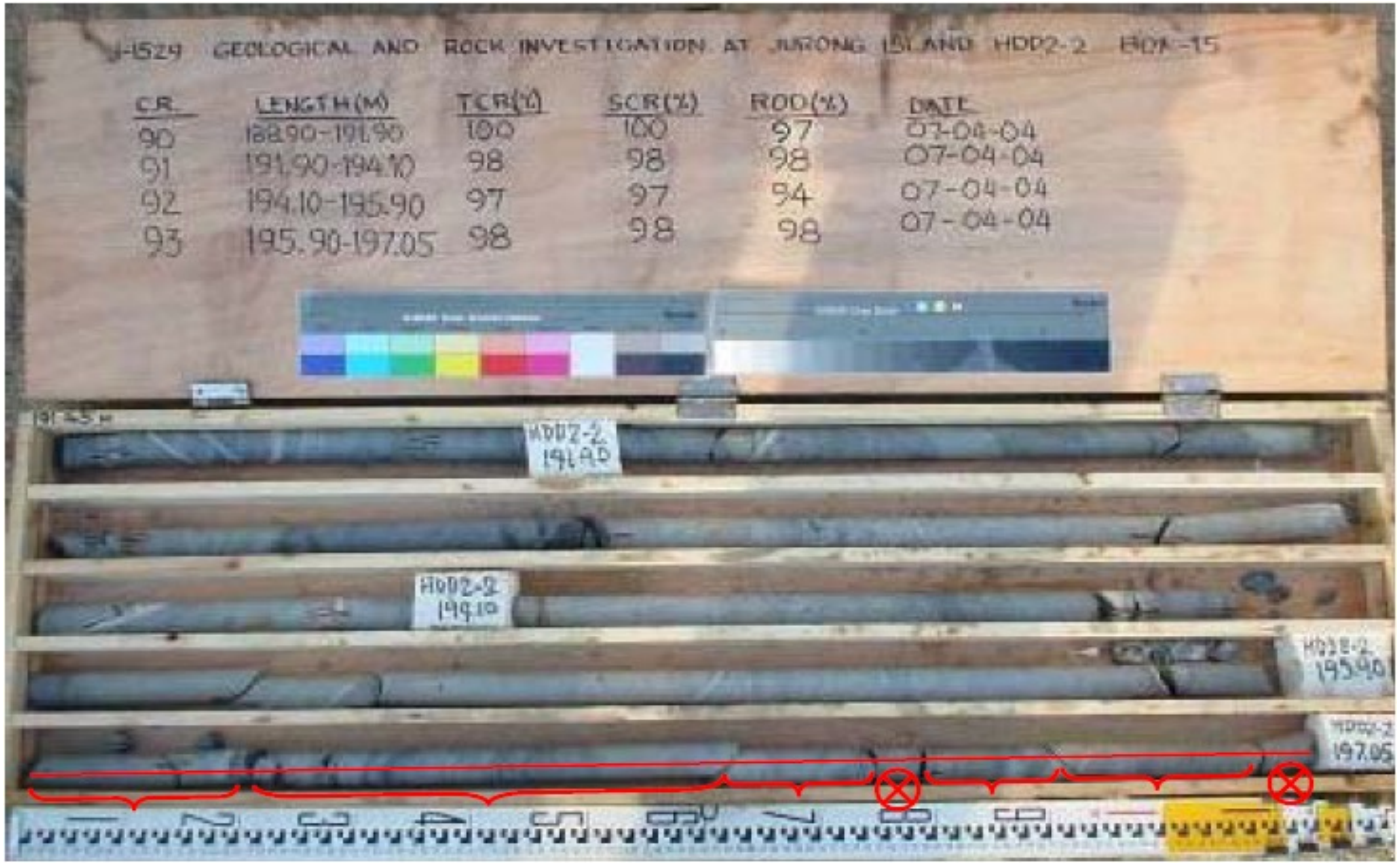
Rock Core Drilling

- Done with either tungsten carbide or diamond core bits
- Use a double or triple tube core barrel when sampling weathered or fractured rock
- Used to determine Rock Quality Designation

Diamond coring bit

core barrel

Rock Quality Designation (RQD)

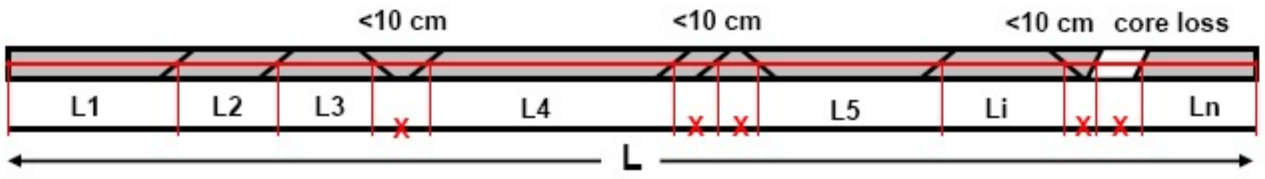


Rock Quality Designation

RQD

Rock Quality Designation (RQD) is defined as the percentage of rock cores that have length equal or greater than 10 cm over the total drill length.

$$RQD = \Sigma Li / L \times 100\%, \quad Li > 10 \text{ cm}$$



$$RQD = (L1 + L2 + \ldots + Ln) / L \times 100\%$$

RQD	Rock Mass Quality
< 25	Very poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
99 – 100	Excellent

Example on Core Recovery & RQD

- Core run of 150 cm
- Total core recovery = 125 cm
- Core recovery ratio = 125/150 = 83%
- On modified basis, 95 cm are counted

RQD = 95/150=63 %

Core Recovery cm	Modified Core Recovery, cm
25	25
5	0
5	0
7.5	0
10	10
12.5	12.5
7.5	0
10	10
15	15
10	10
5	0
12.5	12.5
125	95

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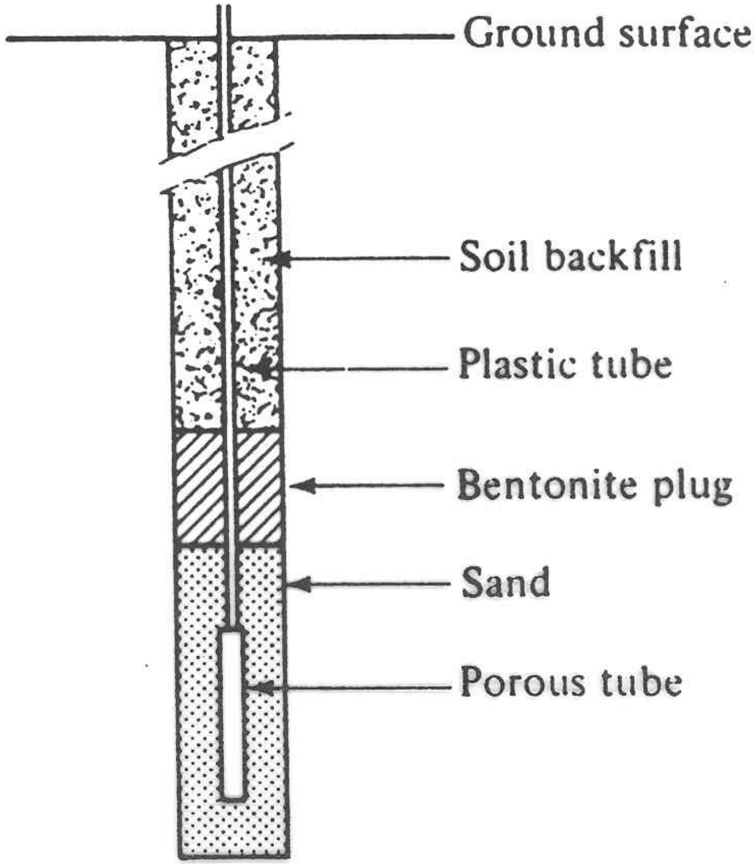
GROUND WATER TABLE LEVEL

Groundwater conditions and the potential for groundwater seepage are fundamental factors in virtually all geotechnical analyses and design studies. Accordingly, the evaluation of groundwater conditions is a basic element of almost all geotechnical investigation programs. Groundwater investigations are of two types as follows:

- Determination of groundwater levels and pressures.
- Measurement of the permeability of the subsurface materials.



Piezometer installation.

FIELD STRENGTH TESTS

The following are the major field tests for determining the soil strength:

1. Vane shear test (VST).
2. Standard Penetration Test (SPT).
3. Cone Penetration Test (CPT).
4. The Borehole Shear Test (BST).
5. The Flat Dilatometer Test (DMT).
6. The Pressure-meter Test (PMT).
7. The Plate Load Test (PLT).

Thank you