FOUNDATION ENGINEERING 1

Shallow Foundation

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FOUNDATION ENGINEERING 1

Reference :

- Principle of Foundation Engineering, 8th Edition, Braja M Das
- An Introduction to Geotechnical Engineering (1st Ed), Holtz R.D. and Kovacs W.D.

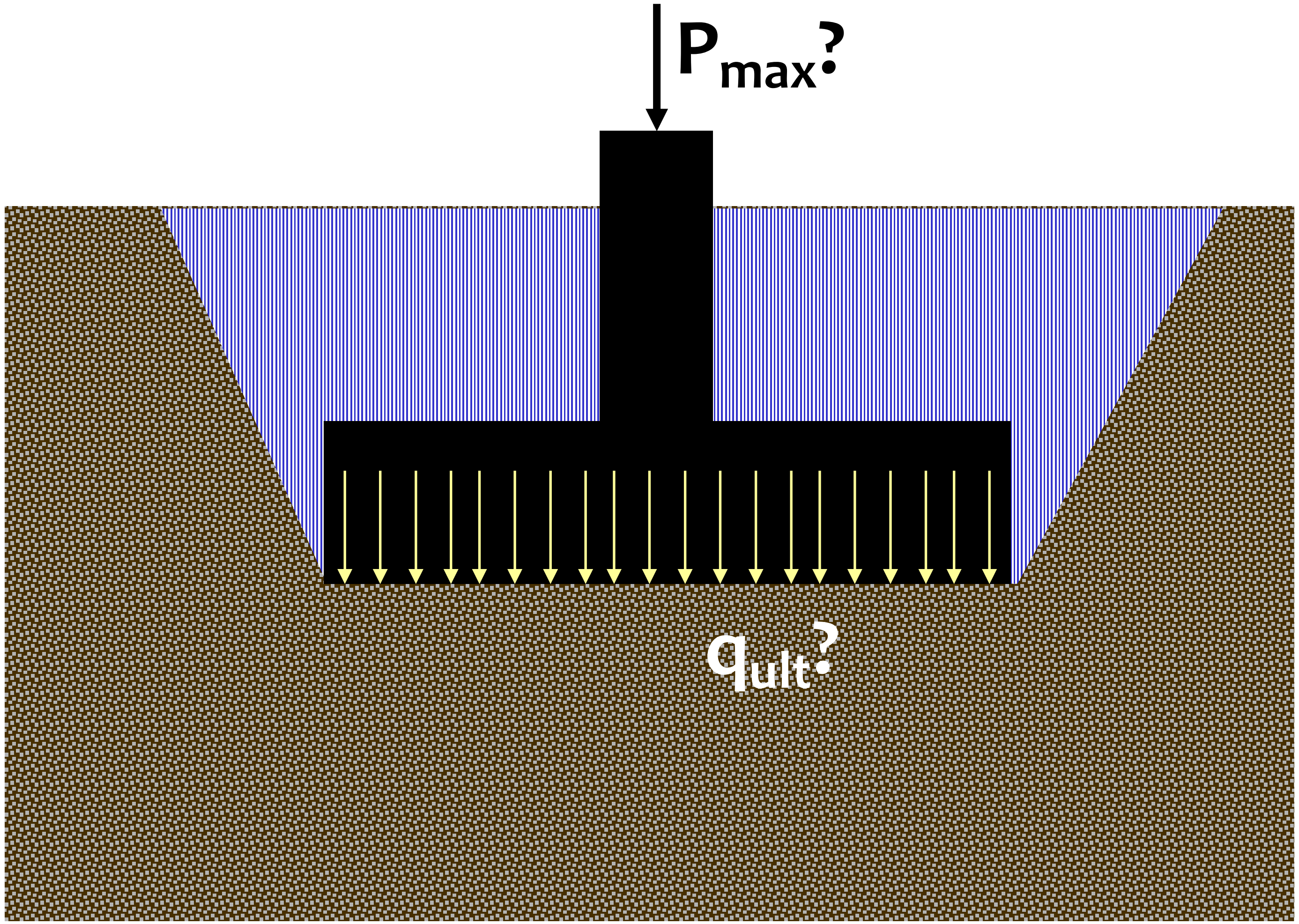
Introduction

Shallow foundation must have two main characteristic :

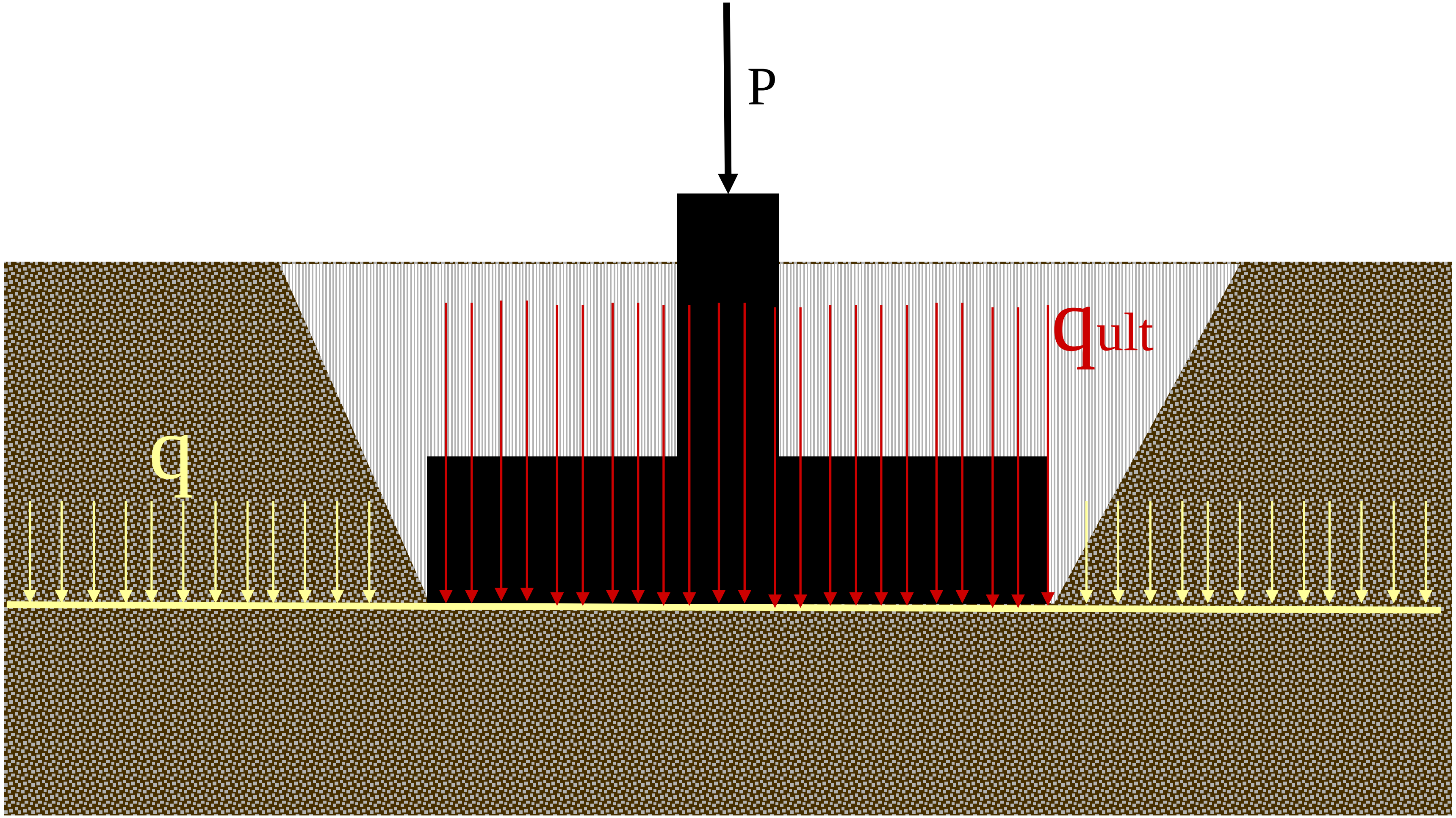
1. The foundations have to be safe against overall shear failure in the soil that support them
2. This foundations can not undergo excessive displacement or settlement

The load unit per area of the foundation at which shear failure in soil will occurs is called **Bearing Load Capacity.**

Teori Daya Dukung Pondasi Dangkal



Teori Daya Dukung Pondasi Dangkal

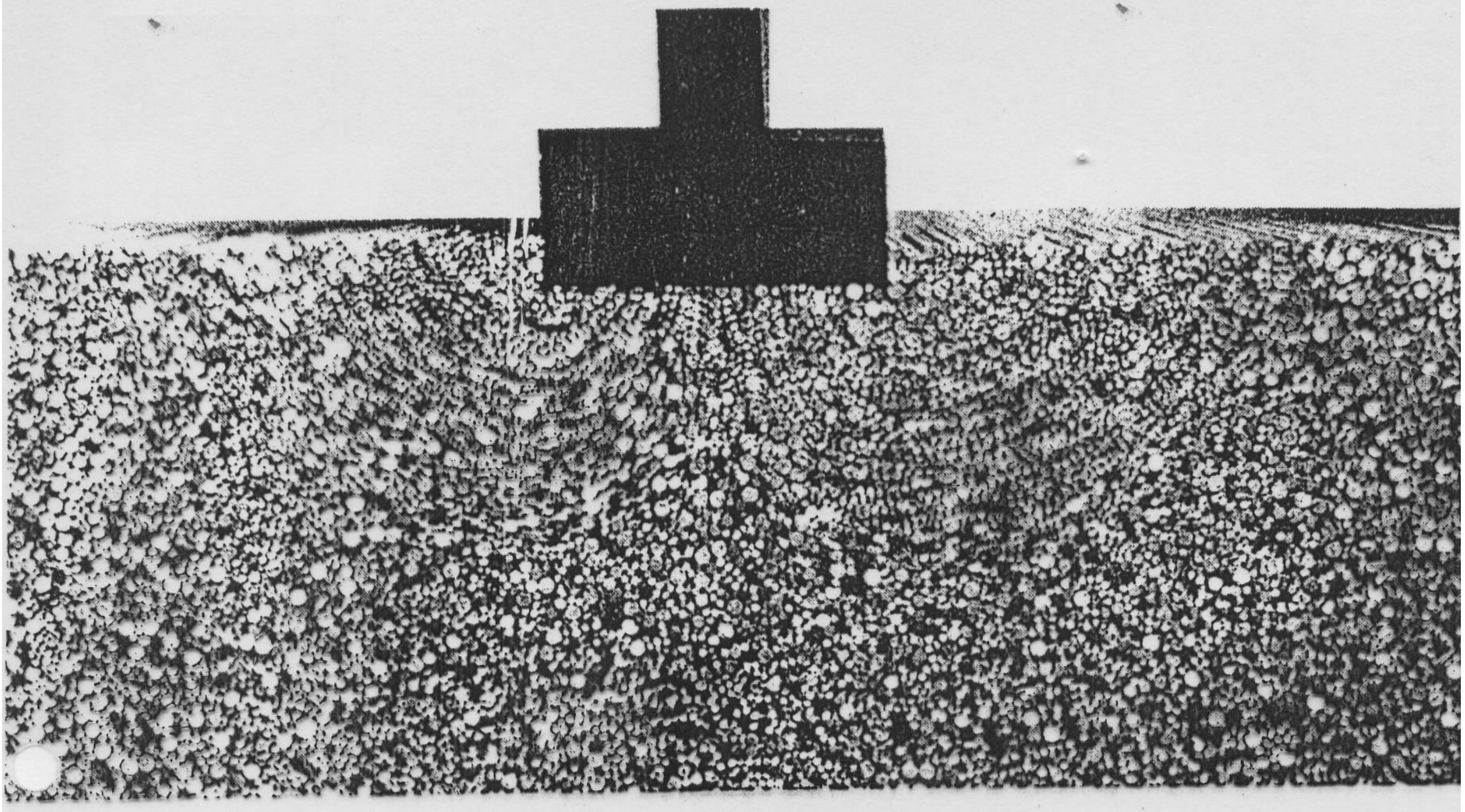


Informasi yang diketahui

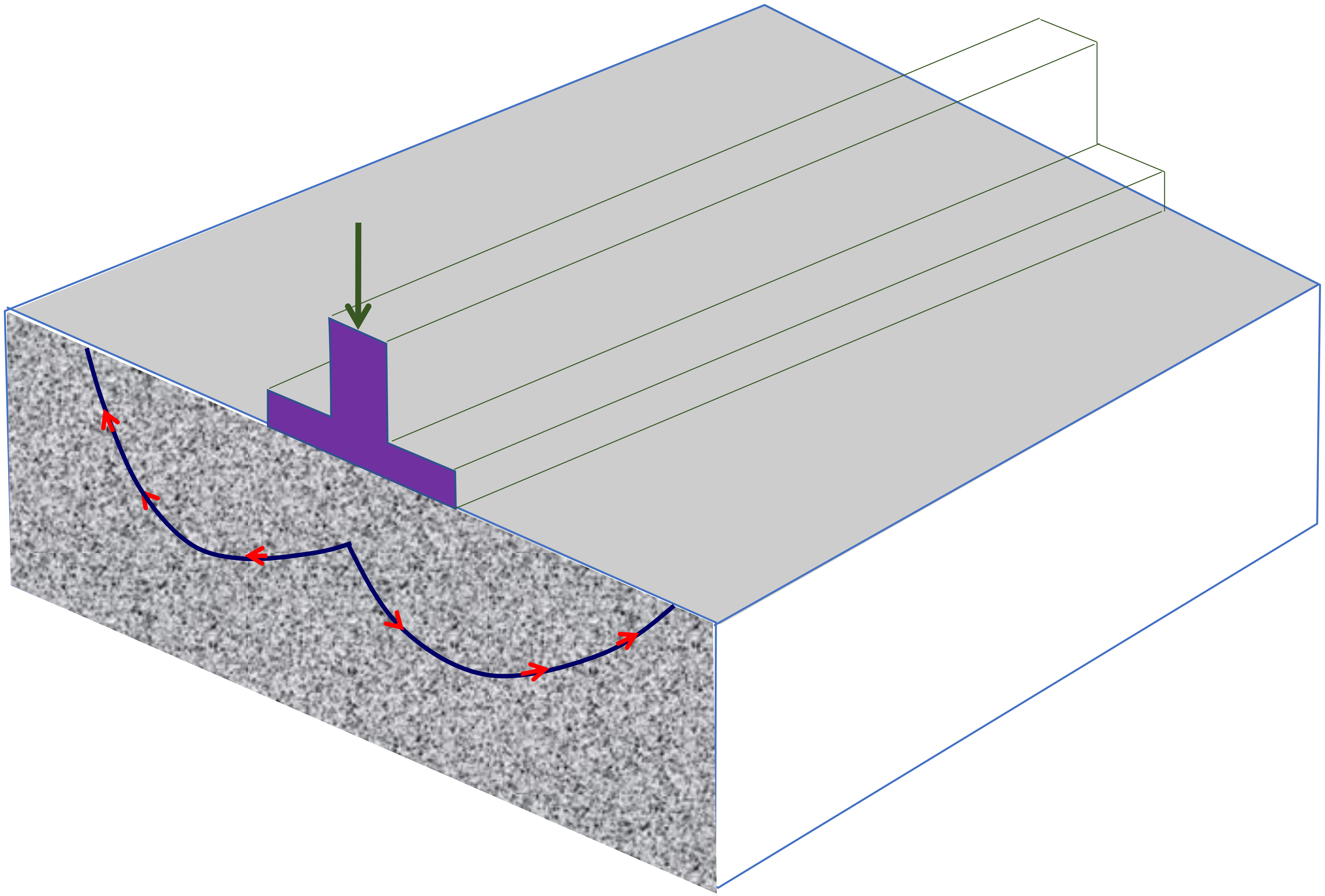
Pondasi : dimensinya (B)

Tanahnya : C, ϕ, γ

POLA KERUNTUHAN TERZAGHI



Failure zones under footing.



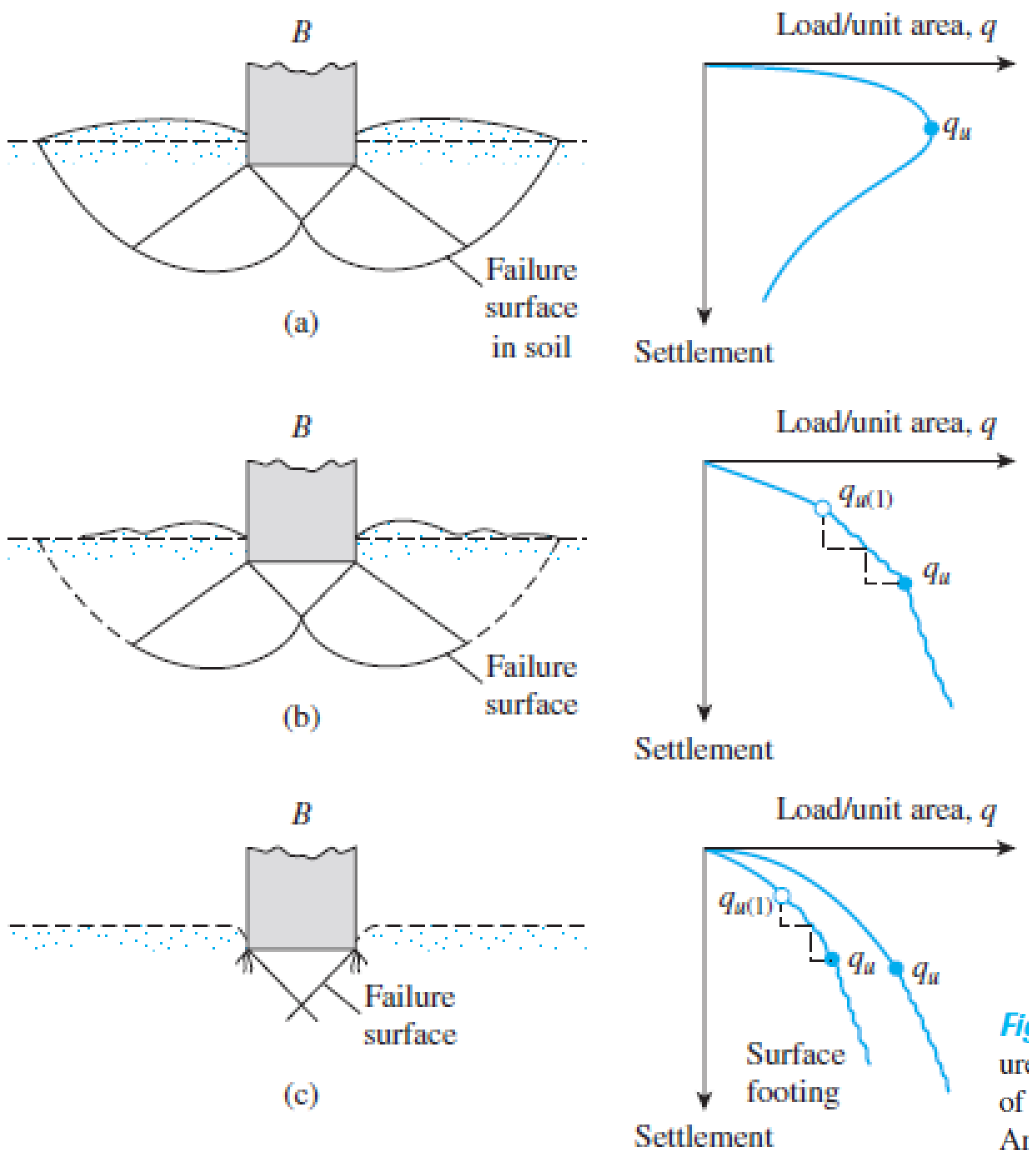
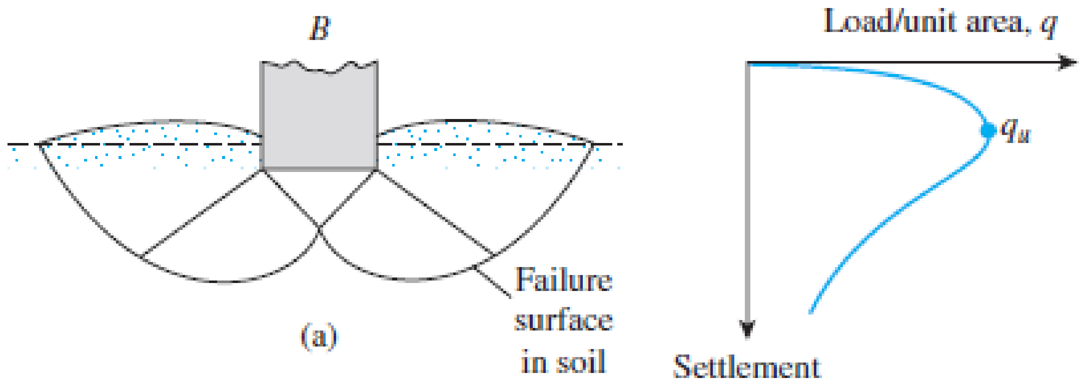


Figure 4.1 Nature of bearing capacity failure in soil: (a) general shear failure: (b) local shear failure; (c) punching shear failure (Redrawn after Vesic, 1973) (Based on Vesic, A. S. (1973). "Analysis of Ultimate Loads of Shallow Foundations," *Journal of Soil Mechanics and Foundations Division*, American Society of Civil Engineers, Vol. 99, No. SM1, pp. 45–73.)

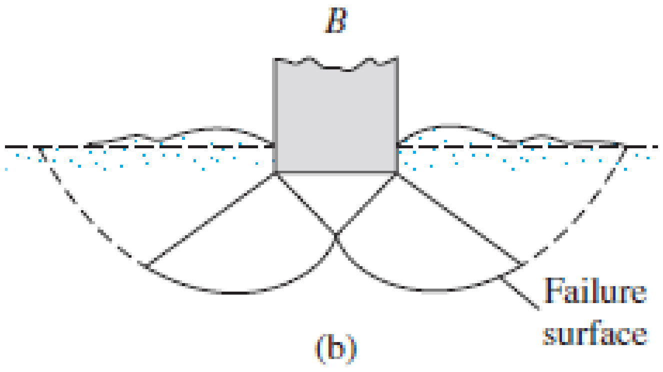
General Shear Failure

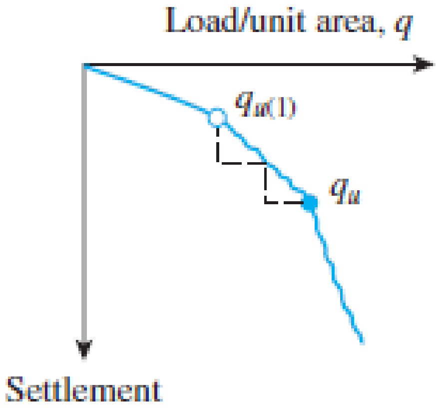


A strip foundation with a width of B resting on the surface of a dense sand or stiff cohesive soil. If a load gradually applied to the foundations, settlement will increase. At a certain point – when the load per unit are equals qu – a sudden failure in the soil supporting the foundation will take place, and the failure surface in the surface in the soil will extend to the ground surface. When such sudden failure in soil takes place, it is called general shear failure.

Local Shear Failure

If the foundation under consideration rests on sand or clayey soil of medium compaction, an increase in the load foundation will also be accompanied by an increase in settlement.



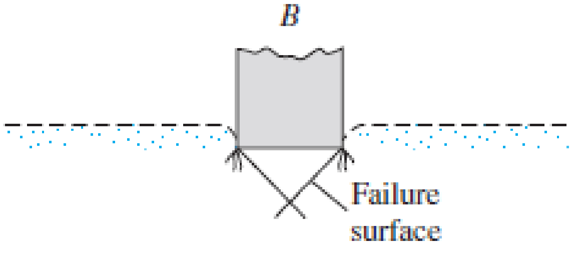


The failure surface in the soil will gradually extend outward from the foundation. When the load per unit area on the foundation equal qu(1), movement of the foundation will be accompanied by sudden jerks. A considerable movement of the foundation is then required for the failure surface in soil to extend to the ground surface. The load per unit area at which this happens is the ultimate bearing capacity, qu.

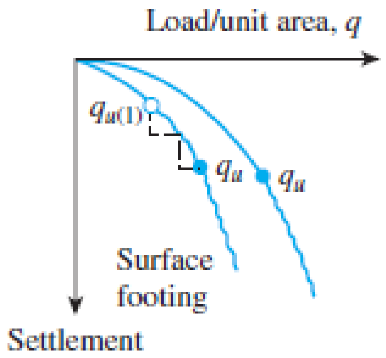
Beyond that point, an increase in load will accompanied by a large increase in foundation settlement. The load per unit area of the foundation, qu(1), is referred to as the first failure load. A peak value of q is not realized in this type of failure, which is called the local shear failure in soil.

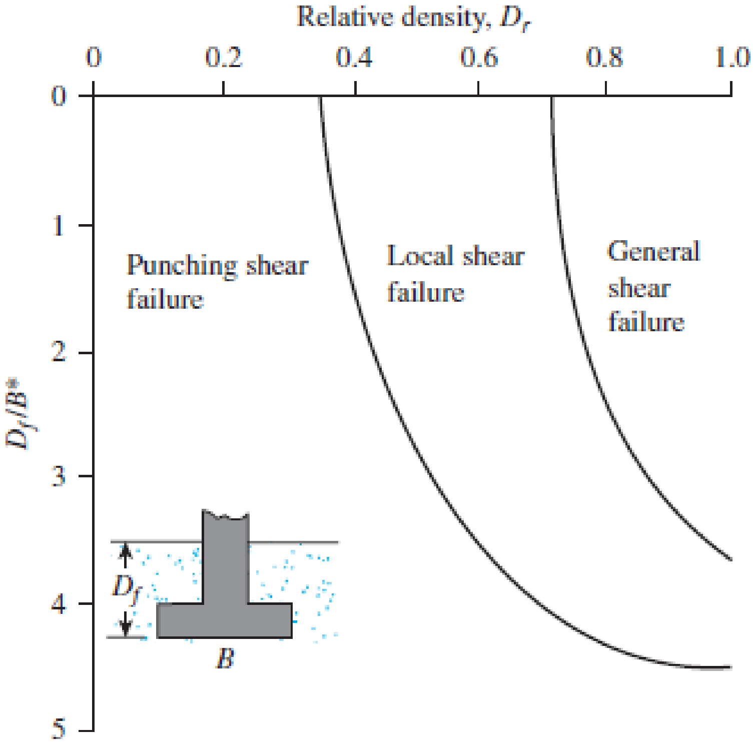
Punching Shear Failure

If the foundation supported by a fairly loose soil, the failure surface in soil will not extend to the ground surface. Beyond the ultimate failure load, qu, the load-settlement plot will be steep and practically linear. This type of failure in soil is called the punching shear failure.



(c)





Terzaghi’s Bearing Capacity Theory

Local Shear Failure

$$q_u = c'N_c + qN_q + \frac{1}{2}\gamma B N_\gamma \quad \text{(Strip foundation)}$$

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma B N_\gamma \quad \text{(square foundation)}$$

$$q_u = 1.3c'N_c + qN_q + 0.3\gamma B N_\gamma \quad \text{(circular foundation)}$$

c’ = cohesion of soil (kN/m^2)

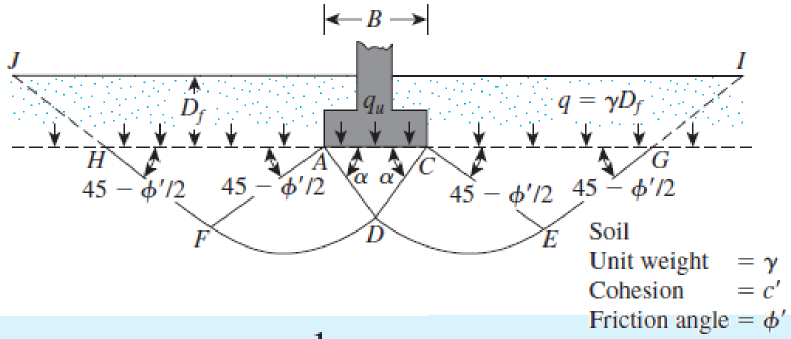
γ = unit wight of soil (kN/m^3)

q = γ Df

where N_c, N_q, and N_γ = bearing capacity factors.

$$q_{all} = \frac{q_u}{FS}$$

Terzaghi's Bearing Capacity Theory



Local Shear Failure

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

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma B N_\gamma$$ (square foundation)

$$q_u = 1.3c'N_c + qN_q + 0.3\gamma B N_\gamma$$ (circular foundation)

c′ = cohesion of soil (kN/m^2)

γ = unit wight of soil (kN/m^3)

q = γ Df

where N_c, N_q, and N_γ = bearing capacity factors.

$$q_{\text{all}} = \frac{q_u}{\text{FS}}$$

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				

Example

Problem 4.1

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.

Solution

From Eq. (4.17)

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

From Table 4.1, for $\phi' = 25°$,

$$N_c = 25.13$$

$$N_q = 12.72$$

$$N_\gamma = 8.34$$

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

Problem 4.1

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.

Example

Refer to Example before 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,

$$q_u = \frac{Q_u}{B^2} = \frac{3000}{B^2} \qquad \text{(a)}$$

From Eq. (4.17),

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

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,

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

Combining Eqs. (a) and (b),

$$\frac{3000}{B^2} = 863.26 + 55.04B \qquad \text{(c)}$$

By trial and error, we have

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

Problems

A square column foundation has to carry a gross allowable load of 1805 kN (FS = 3). Given: $D_f = 1.5$ m, $\gamma = 15.9$ kN/m^3, $\phi' = 34°$, and $c' = 0$. Use Terzaghi's equation to determine the size of the foundation (B). Assume general shear failure.

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

$$q_u = 1.3c'N_c + qN_q + 0.4\gamma B N_\gamma$$ (square foundation)

$$q_u = 1.3c'N_c + qN_q + 0.3\gamma B N_\gamma$$ (circular foundation)

c' = cohesion of soil (kN/m^2)
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q = γ Df

where N_c, N_q, and N_γ = bearing capacity factors.

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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
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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				

Assignment

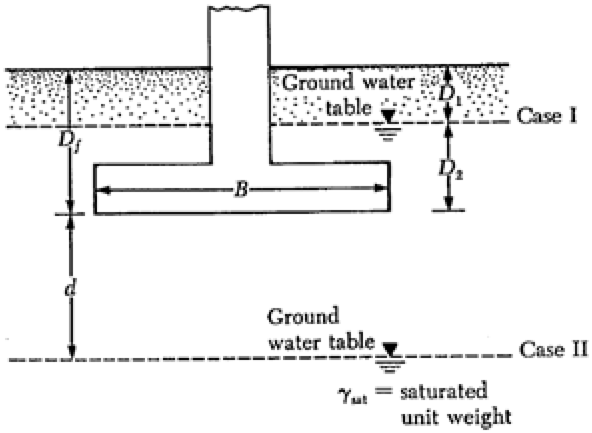
TASK #1
Rekayasa Pondasi I

Shallow Foundation
(duration of task : 3 weeks)
Lecture : Sherly Meiwa ST., MT

Problem No 1

Please explain about *Terzaghi's Bearing Capacity Equation*
Please explain about *Meyerhoff's Bearing Capacity Equation*

Problem No 2



A square foundation is B x B m in plan. The soil supporting the foundation has a friction angle of ϕ and c. The unit weight of soil γ and saturated unit weight of soil γ_{sat}. Assume that the depth of foundation is D_f and that general shear failure occurs in the soil.

B = 1.5 , 1.75, 2.0 m,
ϕ = 20°, 22°, 24°,
c = 14, 15, 16 kN/m².
γ = 17.2, 17.4, 17.6 kN/m³
and γ_{sat} = 19 kN/m³.
D_f = 1, 1.25, 1.5 m

Determine the allowable gross load on the foundation with a factor of safety (FS) of 4 using Terzaghi's Bearing Capacity Equation, if:

a) The ground water table is located in Case II with d = 10m.
b) The ground water table is located in Case II with d = 1m
c) The ground water table is located in Case I with D_1 = 0.5m

End Slide

Meyerhoff's Bearing Capacity Theory

Perhitungan bearing capacity Terzaghi dapat digunakan untuk bentuk pondasi menerus, persegi, dan lingkaran. Namun formula perhitungan ini tidak bisa merepresentasikan untuk bentuk pondasi bujursangkar (0 < B/L < 1). Selain itu beban pada beberapa kondisi terdapat inklinasi pada arah beban di pondasi. Meyerhoff (1963) telah merumuskan hal ini melalui rumus berikut :

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

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

Meyerhoff's Bearing Capacity Theory

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
ϕ'	N_c	N_q	N_γ	ϕ'	N_c	N_q	N_γ
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				

$$N_q = \tan^2\left(45 + \frac{\phi'}{2}\right)e^{\pi \tan \phi'}$$

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

$$N_\gamma = 2(N_q + 1)\tan \phi'$$

Meyerhoff's Bearing Capacity Theory

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)
Depth	$\frac{D_f}{B} \leq 1$ For $\phi = 0$: $F_{cd} = 1 + 0.4\left(\frac{D_f}{B}\right)$ $F_{qd} = 1$ $F_{\gamma d} = 1$ For $\phi' > 0$: $F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'}$ $F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right)$ $F_{\gamma d} = 1$ $\frac{D_f}{B} > 1$	Hansen (1970)

Meyerhoff's Bearing Capacity Theory

Factor	Relationship	Reference
	For $\phi = 0$: $F_{cd} = 1 + 0.4 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$ $F_{qd} = 1$ $F_{\gamma d} = 1$ For $\phi' > 0$: $F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan \phi'}$ $F_{qd} = 1 + 2 \tan \phi'(1 - \sin \phi')^2 \underbrace{\tan^{-1}\left(\frac{D_f}{B}\right)}_{\text{radians}}$ $F_{\gamma d} = 1$	
Inclination	$F_{ci} = F_{qi} = \left(1 - \frac{\beta^\circ}{90^\circ}\right)^2$ $F_{\gamma i} = \left(1 - \frac{\beta^\circ}{\phi'}\right)^2$ β = inclination of the load on the foundation with respect to the vertical	Meyerhof (1963); Hanna and Meyerhof (1981)

Meyerhoff's Bearing Capacity Theory

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.

Solve Example Problem 4.1 using Eq. (4.26).

Solution

From Eq. (4.26),

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

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,

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

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

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

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

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

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

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

Hence,

$$q_u = (20)(20.72)(1.514)(1.257)(1)$$
$$+ (1.5 \times 16.5)(10.66)(1.466)(1.233)(1)$$
$$+ \frac{1}{2}(16.5)(2)(10.88)(0.6)(1)(1)$$
$$= 788.6 + 476.9 + 107.7 = 1373.2 \text{ kN/m}^2$$

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

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

Meyerhoff's Bearing Capacity Theory

Meyerhof's Bearing Capacity Theory

Hence,

$$q_u = (20)(20.72)(1.514)(1.257)(1) + (1.5 \times 16.5)(10.66)(1.466)(1.233)(1) + \frac{1}{2}(16.5)(2)(10.88)(0.6)(1)(1)$$

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

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

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

Terzaghi's Bearing Capacity Theory

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

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

Meyerhoff's Bearing Capacity Theory

A square column foundation (Figure 4.11) is to be constructed on a sand deposit. The allowable load Q will be inclined at an angle $\beta = 20°$ with the vertical. The standard penetration numbers N_{60} obtained from the field are as follows.

Depth (m)	N_{60}
1.5	3
3.0	6
4.5	9
6.0	10
7.5	10
9.0	8

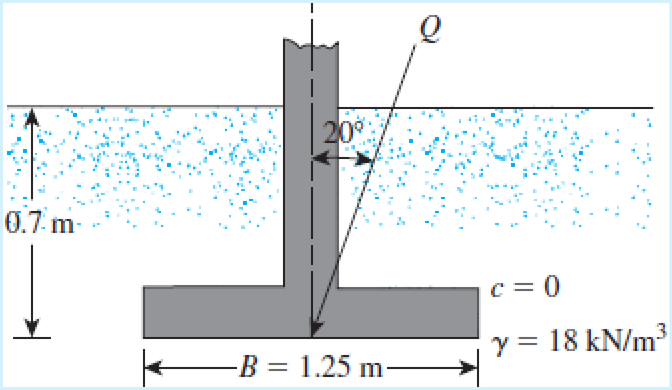


Figure 4.11

Determine Q. Use FS = 3, Eq. (3.29), and Eq. (4.26).

Solution

From Eq. (3.29),

$$\phi' \text{ (deg)} = 27.1 + 0.3N_{60} - 0.00054(N_{60})^2$$

The following is an estimation of ϕ' in the field using Eq. (3.29).

Depth (m)	N_{60}	ϕ' (deg)
1.5	3	28
3.0	6	29
4.5	9	30
6.0	10	30
7.5	10	30
9.0	8	29

Average = 29.4° ≈ 30°

With $c' = 0$, the ultimate bearing capacity [Eq. (4.26)] becomes

$$q_u = qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

$$q = (0.7)(18) = 12.6 \text{ kN/m}^2$$

$$\gamma = 18 \text{ kN/m}^3$$

Meyerhof's Bearing Capacity Theory

From Table 4.2 for $\phi' = 30°$,

$$N_q = 18.4$$
$$N_\gamma = 22.4$$

From Table 4.3, (*Note: B = L*)

$$F_{qs} = 1 + \left(\frac{B}{L}\right) \tan \phi' = 1 + 0.577 = 1.577$$

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

$$F_{qd} = 1 + 2 \tan \phi'(1 - \sin \phi')^2 \frac{D_f}{B} = 1 + \frac{(0.289)(0.7)}{1.25} = 1.162$$

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

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

$$F_{\gamma i} = \left(1 - \frac{\beta°}{\phi'}\right)^2 = \left(1 - \frac{20}{30}\right)^2 = 0.11$$

Hence,

$$q_u = (12.6)(18.4)(1.577)(1.162)(0.605) + \left(\frac{1}{2}\right)(18)(1.25)(22.4)(0.6)(1)(0.11)$$
$$= 273.66 \text{ kN/m}^2$$

$$q_{all} = \frac{q_u}{FS} = \frac{273.66}{3} = 91.22 \text{ kN/m}^2$$

Now,

$$Q \cos 20 = q_{all} B^2 = (91.22)(1.25)^2$$
$$Q \approx \mathbf{151.7 \text{ kN}}$$

Meyerhof's Bearing Capacity Theory

Problem 4.4

The applied load on a shallow square foundation makes an angle of 20° with the vertical. Given: $B = 5$ ft, $D_f = 3$ ft, $\gamma = 115$ lb/ft^3, $\phi' = 25°$, and $c' = 600$ lb/ft^2. Use FS = 3 and determine the gross inclined allowable load. Use Eq. (4.26).

Modification of Bearing Capacity Equations for Water Table

If the water table is close to the foundation, some modification of the beraing capacity equation will be necessary :

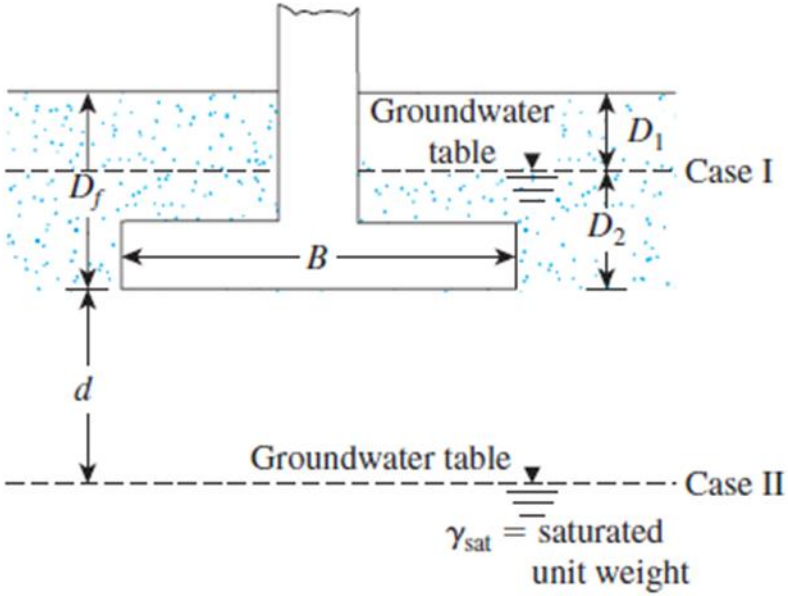
Case I. If the water table is located so that $0 \leq D_1 \leq D_f$, the factor q in the bearing capacity equations takes the form

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

$$\gamma' = \gamma_{sat} - \gamma_w$$

where

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



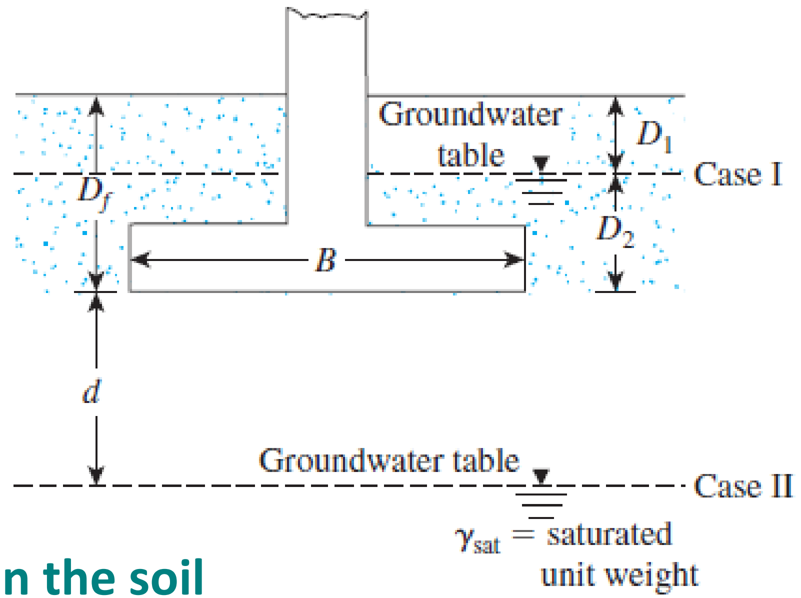
Modification of Bearing Capacity Equations for Water Table

Case II. For a water table located so that $0 \le d \le B$,

$$q = \gamma D_f$$

In this case, the factor gin the last term of the bearing capacity equations must be replace by the factor :

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



Based the assumption no seepage force in the soil

Case III. When the water table is located so that $d \ge B$, the water will have no effect on the ultimate bearing capacity.

Eccentrically Loaded Foundations

In several cases, foundation are subjected to moments in addition to the vertical load.

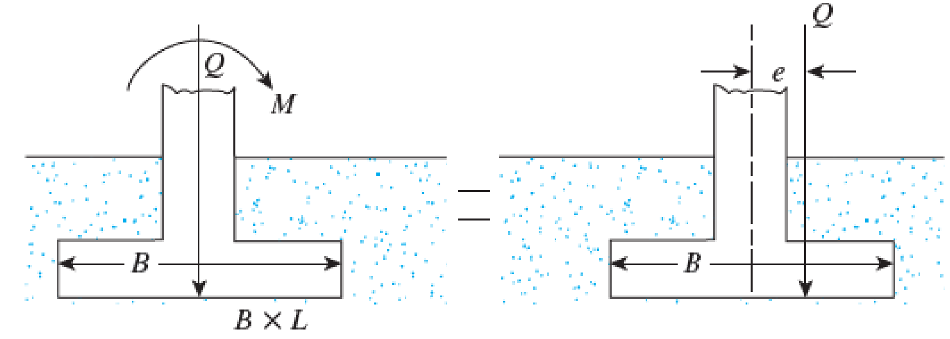
The nominal distribution of pressure :

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

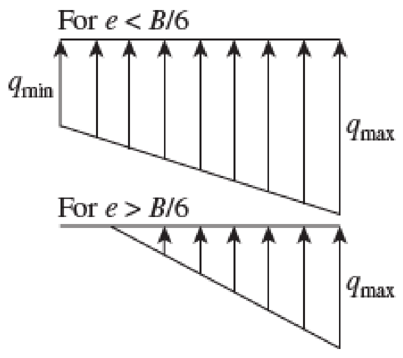
Where :

Q = total vertical load

M = Moment on the foundation



(b)



(a)

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

Eccentrically Loaded Foundations

The Distance : $e = \frac{M}{Q}$ is the eccentricity. So :

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

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

For e > B/6. q_{min} will be negative, 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 value of q_{max} :

$$q_{max} = \frac{4Q}{3L(B - 2e)}$$

The factor of safety for such types of loading against bearing capacity failure can be evaluated by using the procedure suggested by Meyerhoff (1953), Which is generally referred to as the effective area method.

The following is Meyerhof's step-by-step procedure for determining the ultimate load that soil can support and the factor of safety against bearing capacity failure :

STEP 1

Determine the effective dimensions of the foundation :

B' = Effective width = B – 2e

L' = effective length = L

(if the eccentricity were in the direction of the length of the foundation, the value of L' = L -2e and B' = B

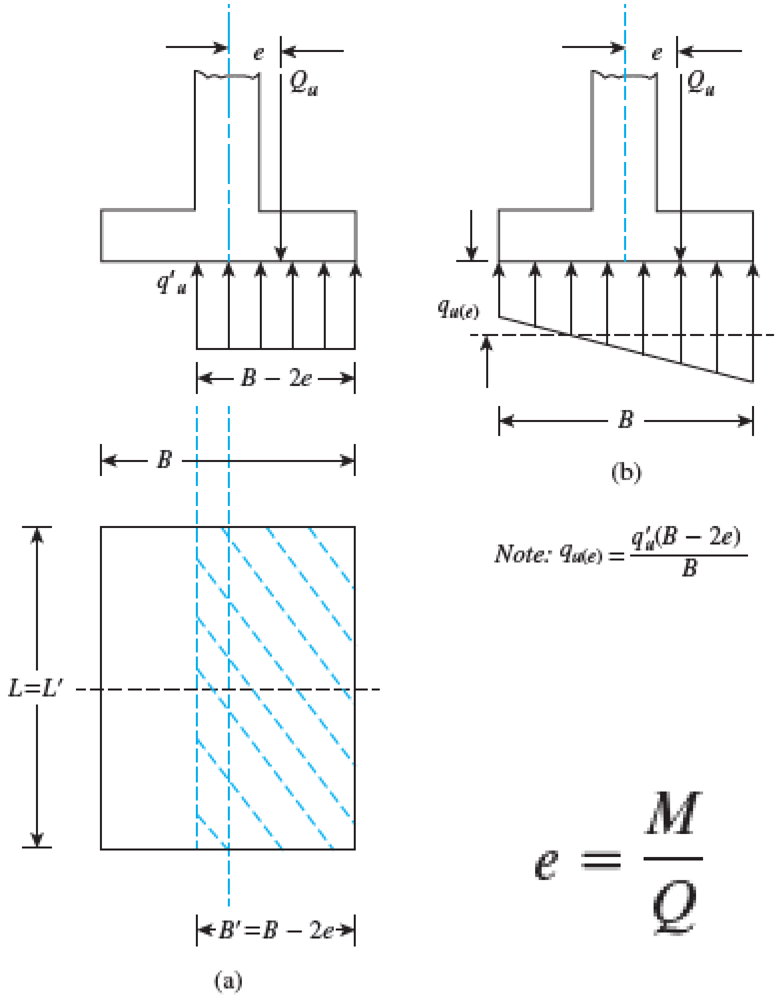


Figure 4.19 Definition of q'_u and $q_{u(e)}$

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

Ultimate Bearing Capacity under Eccentric Loading One-Way Eccentricity

STEP 2

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

To evaluate of shape factor with effective length and effective width dimension instead of L and B, respectively. To determine depth factor do not replace B with B' .

STEP 3

The total ultimate load that the foundation can sustain is : $Q_u = q'_u \overbrace{(B')(L')}^{A'}$

STEP 4

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

STEP 5

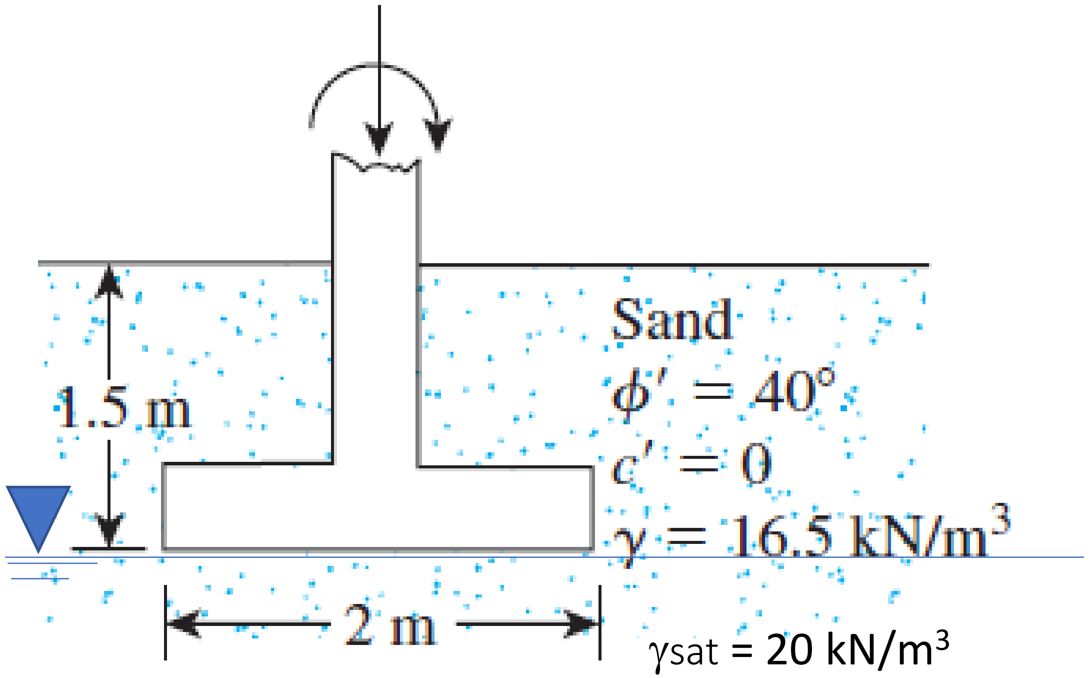
Check the factor of safety against q_{max} or FS = q_u'/q_{max}

Eccentrically Loaded Foundations

Example #4

A square foundation is shown in figure. If the Load = 10 ton and moment = 15 ton/m , determine the size of footing. Use formula from Terzaghi ?

What next ??



γ_{sat} = 20 kN/m^3

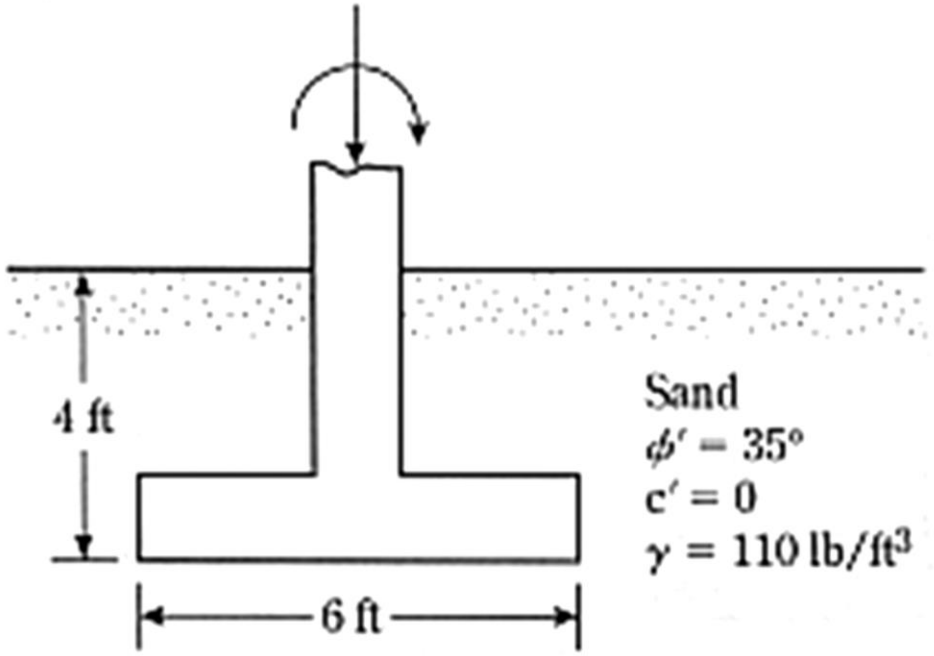
C' = 20 kN/m^2

ϕ = 20^o

Eccentrically Loaded Foundations

Example #4

A continues foundation is shown in figure. If the load eccentricity is 0.5ft, determine the ultimate load, Q_{ult} per unit length of the foundation



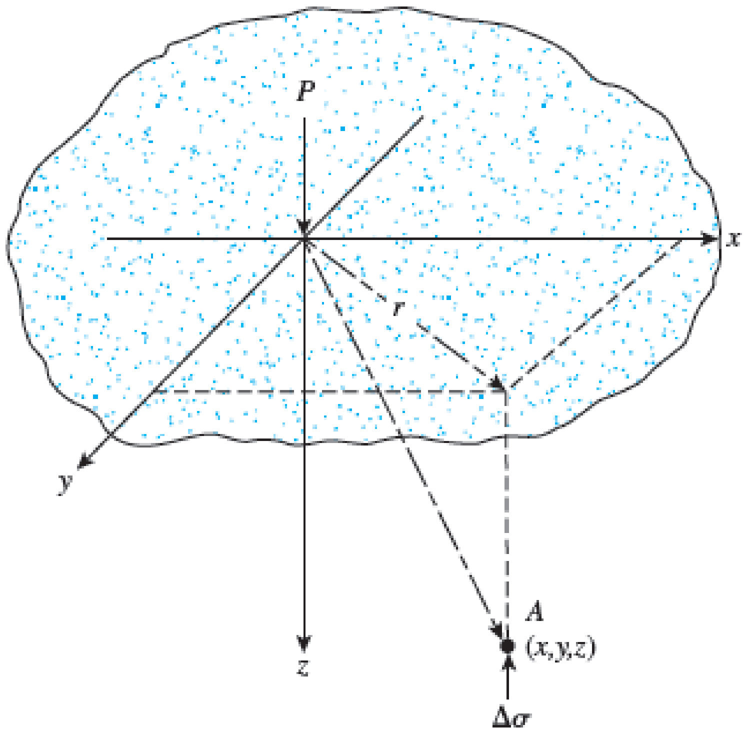
$$q'_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma B'N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

What next ??

Note : Because the foundation in question is a continues, B'/L' will be Zero

Tegangan Akibat Beban Terkonsentrasi

Tahun 1885, Boussinesq menemukan sebuah hubungan matematis untuk menentukan tegangan normal dan tegangan geser untuk kondisi tanah homogen, elastic, dan isotropic akibat beban terkonsentrasi di permukaan tanah.



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

where

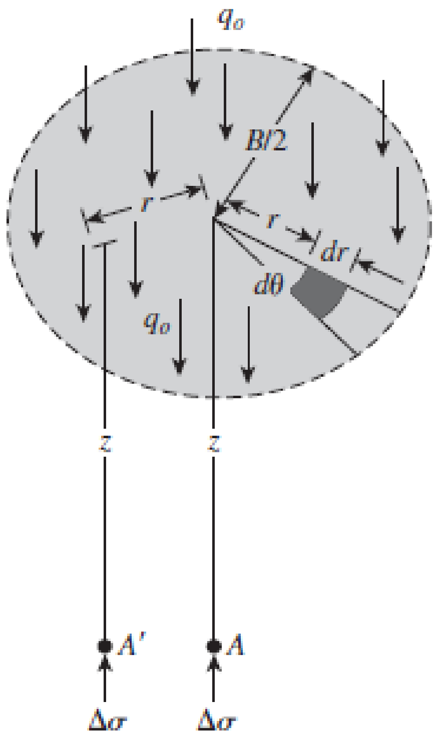
$$r = \sqrt{x^2 + y^2}$$

x, y, z = coordinates of the point A

Tegangan Akibat Beban Lingkaran

Di asumsikan radius = B/2, dan qo adalah beban yang terdistribusi seragam per unit area. Untuk menentukan tegangan pada titik A yang berlokasi pada kedalaman z, Z harus diasumsikan persis dibawah titik pusat beban lingkaran.

$$\Delta\sigma = q_o\left\{1 - \frac{1}{\left[1 + \left(\frac{B}{2z}\right)^2\right]^{3/2}}\right\}$$



Tegangan Akibat Beban Lingkaran

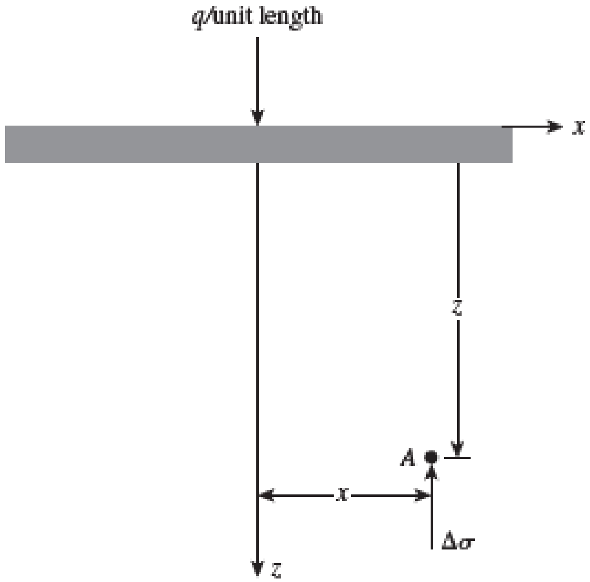
Table 6.1 Variation of $\Delta\sigma/q_o$ for a Uniformly Loaded Flexible Circular Area

	$r/(B/2)$					
$z/(B/2)$	0	0.2	0.4	0.6	0.8	1.0
0	1.000	1.000	1.000	1.000	1.000	1.000
0.1	0.999	0.999	0.998	0.996	0.976	0.484
0.2	0.992	0.991	0.987	0.970	0.890	0.468
0.3	0.976	0.973	0.963	0.922	0.793	0.451
0.4	0.949	0.943	0.920	0.860	0.712	0.435
0.5	0.911	0.902	0.869	0.796	0.646	0.417
0.6	0.864	0.852	0.814	0.732	0.591	0.400
0.7	0.811	0.798	0.756	0.674	0.545	0.367
0.8	0.756	0.743	0.699	0.619	0.504	0.366
0.9	0.701	0.688	0.644	0.570	0.467	0.348
1.0	0.646	0.633	0.591	0.525	0.434	0.332
1.2	0.546	0.535	0.501	0.447	0.377	0.300
1.5	0.424	0.416	0.392	0.355	0.308	0.256
2.0	0.286	0.286	0.268	0.248	0.224	0.196
2.5	0.200	0.197	0.191	0.180	0.167	0.151
3.0	0.146	0.145	0.141	0.135	0.127	0.118
4.0	0.087	0.086	0.085	0.082	0.080	0.075

Tegangan Akibat Beban Garis

Table 6.2 Variation of $\Delta\sigma/(q/z)$ with x/z [Eq. (6.5)]

x/z	$\Delta\sigma/(q/z)$	x/z	$\Delta\sigma/(q/z)$
0	0.637	1.3	0.088
0.1	0.624	1.4	0.073
0.2	0.589	1.5	0.060
0.3	0.536	1.6	0.050
0.4	0.473	1.7	0.042
0.5	0.407	1.8	0.035
0.6	0.344	1.9	0.030
0.7	0.287	2.0	0.025
0.8	0.237	2.2	0.019
0.9	0.194	2.4	0.014
1.0	0.159	2.6	0.011
1.1	0.130	2.8	0.008
1.2	0.107	3.0	0.006



$$\Delta\sigma = \frac{2q}{\pi z[(x/z)^2 + 1]^2}$$

$$\frac{\Delta\sigma}{(q/z)} = \frac{2}{\pi[(x/z)^2 + 1]^2}$$

$$\Delta\sigma = \frac{2qz^3}{\pi(x^2 + z^2)^2}$$

Tegangan Vertikal dalam Tanah

Tegangan Akibat Beban Area Persegi Panjang

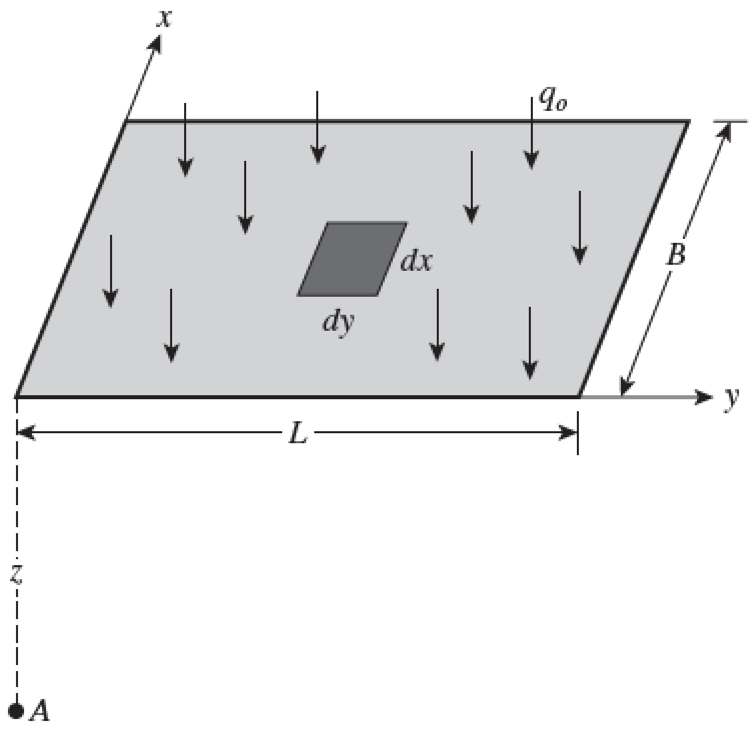
Formula Boussinesq

$$\Delta\sigma = \int_{y=0}^{L}\int_{x=0}^{B}\frac{3q_o\,(dx\,dy)z^3}{2\pi(x^2+y^2+z^2)^{5/2}} = q_o I$$

$$I = \text{influence factor} = \frac{1}{4\pi}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\cdot\frac{m^2+n^2+2}{m^2+n^2+1} + \tan^{-1}\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)$$

$$m = \frac{B}{z}$$

$$n = \frac{L}{z}$$



Tegangan Vertikal dalam Tanah

Tegangan Akibat Beban Area Persegi Panjang

Table 6.4 Variation of Influence Value I [Eq. (6.10)][a]

	n											
m	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4
0.1	0.00470	0.00917	0.01323	0.01678	0.01978	0.02223	0.02420	0.02576	0.02698	0.02794	0.02926	0.03007
0.2	0.00917	0.01790	0.02585	0.03280	0.03866	0.04348	0.04735	0.05042	0.05283	0.05471	0.05733	0.05894
0.3	0.01323	0.02585	0.03735	0.04742	0.05593	0.06294	0.06858	0.07308	0.07661	0.07938	0.08323	0.08561
0.4	0.01678	0.03280	0.04742	0.06024	0.07111	0.08009	0.08734	0.09314	0.09770	0.10129	0.10631	0.10941
0.5	0.01978	0.03866	0.05593	0.07111	0.08403	0.09473	0.10340	0.11035	0.11584	0.12018	0.12626	0.13003
0.6	0.02223	0.04348	0.06294	0.08009	0.09473	0.10688	0.11679	0.12474	0.13105	0.13605	0.14309	0.14749
0.7	0.02420	0.04735	0.06858	0.08734	0.10340	0.11679	0.12772	0.13653	0.14356	0.14914	0.15703	0.16199
0.8	0.02576	0.05042	0.07308	0.09314	0.11035	0.12474	0.13653	0.14607	0.15371	0.15978	0.16843	0.17389
0.9	0.02698	0.05283	0.07661	0.09770	0.11584	0.13105	0.14356	0.15371	0.16185	0.16835	0.17766	0.18357
1.0	0.02794	0.05471	0.07938	0.10129	0.12018	0.13605	0.14914	0.15978	0.16835	0.17522	0.18508	0.19139
1.2	0.02926	0.05733	0.08323	0.10631	0.12626	0.14309	0.15703	0.16843	0.17766	0.18508	0.19584	0.20278
1.4	0.03007	0.05894	0.08561	0.10941	0.13003	0.14749	0.16199	0.17389	0.18357	0.19139	0.20278	0.21020
1.6	0.03058	0.05994	0.08709	0.11135	0.13241	0.15028	0.16515	0.17739	0.18737	0.19546	0.20731	0.21510
1.8	0.03090	0.06058	0.08804	0.11260	0.13395	0.15207	0.16720	0.17967	0.18986	0.19814	0.21032	0.21836
2.0	0.03111	0.06100	0.08867	0.11342	0.13496	0.15326	0.16856	0.18119	0.19152	0.19994	0.21235	0.22058
2.5	0.03138	0.06155	0.08948	0.11450	0.13628	0.15483	0.17036	0.18321	0.19375	0.20236	0.21512	0.22364
3.0	0.03150	0.06178	0.08982	0.11495	0.13684	0.15550	0.17113	0.18407	0.19470	0.20341	0.21633	0.22499
4.0	0.03158	0.06194	0.09007	0.11527	0.13724	0.15598	0.17168	0.18469	0.19540	0.20417	0.21722	0.22600
5.0	0.03160	0.06199	0.09014	0.11537	0.13737	0.15612	0.17185	0.18488	0.19561	0.20440	0.21749	0.22632
6.0	0.03161	0.06201	0.09017	0.11541	0.13741	0.15617	0.17191	0.18496	0.19569	0.20449	0.21760	0.22644
8.0	0.03162	0.06202	0.09018	0.11543	0.13744	0.15621	0.17195	0.18500	0.19574	0.20455	0.21767	0.22652
10.0	0.03162	0.06202	0.09019	0.11544	0.13745	0.15622	0.17196	0.18502	0.19576	0.20457	0.21769	0.22654
∞	0.03162	0.06202	0.09019	0.11544	0.13745	0.15623	0.17197	0.18502	0.19577	0.20458	0.21770	0.22656

Tegangan Akibat Beban Area Persegi Panjang

Table 6.4 (*Continued*)

	n										
m	1.6	1.8	2.0	2.5	3.0	4.0	5.0	6.0	8.0	10.0	∞
0.1	0.03058	0.03090	0.03111	0.03138	0.03150	0.03158	0.03160	0.03161	0.03162	0.03162	0.03162
0.2	0.05994	0.06058	0.06100	0.06155	0.06178	0.06194	0.06199	0.06201	0.06202	0.06202	0.06202
0.3	0.08709	0.08804	0.08867	0.08948	0.08982	0.09007	0.09014	0.09017	0.09018	0.09019	0.09019
0.4	0.11135	0.11260	0.11342	0.11450	0.11495	0.11527	0.11537	0.11541	0.11543	0.11544	0.11544
0.5	0.13241	0.13395	0.13496	0.13628	0.13684	0.13724	0.13737	0.13741	0.13744	0.13745	0.13745
0.6	0.15028	0.15207	0.15326	0.15483	0.15550	0.15598	0.15612	0.15617	0.15621	0.15622	0.15623
0.7	0.16515	0.16720	0.16856	0.17036	0.17113	0.17168	0.17185	0.17191	0.17195	0.17196	0.17197
0.8	0.17739	0.17967	0.18119	0.18321	0.18407	0.18469	0.18488	0.18496	0.18500	0.18502	0.18502
0.9	0.18737	0.18986	0.19152	0.19375	0.19470	0.19540	0.19561	0.19569	0.19574	0.19576	0.19577
1.0	0.19546	0.19814	0.19994	0.20236	0.20341	0.20417	0.20440	0.20449	0.20455	0.20457	0.20458
1.2	0.20731	0.21032	0.21235	0.21512	0.21633	0.21722	0.21749	0.21760	0.21767	0.21769	0.21770
1.4	0.21510	0.21836	0.22058	0.22364	0.22499	0.22600	0.22632	0.22644	0.22652	0.22654	0.22656
1.6	0.22025	0.22372	0.22610	0.22940	0.23088	0.23200	0.23236	0.23249	0.23258	0.23261	0.23263
1.8	0.22372	0.22736	0.22986	0.23334	0.23495	0.23617	0.23656	0.23671	0.23681	0.23684	0.23686
2.0	0.22610	0.22986	0.23247	0.23614	0.23782	0.23912	0.23954	0.23970	0.23981	0.23985	0.23987
2.5	0.22940	0.23334	0.23614	0.24010	0.24196	0.24344	0.24392	0.24412	0.24425	0.24429	0.24432
3.0	0.23088	0.23495	0.23782	0.24196	0.24394	0.24554	0.24608	0.24630	0.24646	0.24650	0.24654
4.0	0.23200	0.23617	0.23912	0.24344	0.24554	0.24729	0.24791	0.24817	0.24836	0.24842	0.24846
5.0	0.23236	0.23656	0.23954	0.24392	0.24608	0.24791	0.24857	0.24885	0.24907	0.24914	0.24919
6.0	0.23249	0.23671	0.23970	0.24412	0.24630	0.24817	0.24885	0.24916	0.24939	0.24946	0.24952
8.0	0.23258	0.23681	0.23981	0.24425	0.24646	0.24836	0.24907	0.24939	0.24964	0.24973	0.24980
10.0	0.23261	0.23684	0.23985	0.24429	0.24650	0.24842	0.24914	0.24946	0.24973	0.24981	0.24989
∞	0.23263	0.23686	0.23987	0.24432	0.24654	0.24846	0.24919	0.24952	0.24980	0.24989	0.25000

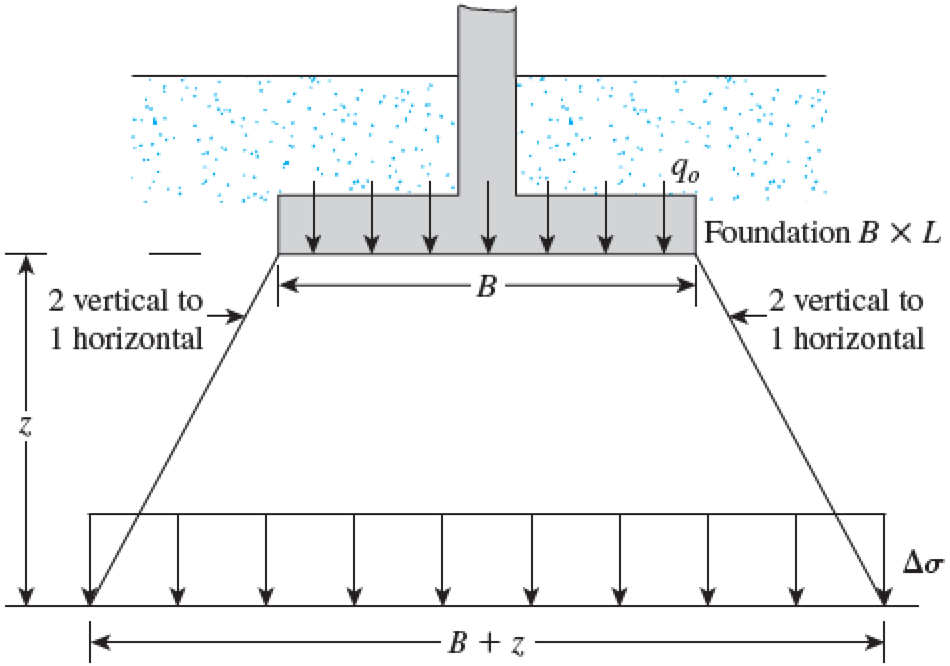
[a]Based on Saika, 2012

Tegangan Akibat Beban Area Persegi Panjang

Metode 2:1

Formula Boussinesq

$$\Delta\sigma = \frac{q_o \times B \times L}{(B+z)(L+z)}$$



Tegangan Akibat Beban Timbunan

$$\Delta\sigma = \frac{q_o}{\pi}\left[\left(\frac{B_1 + B_2}{B_2}\right)(\alpha_1 + \alpha_2) - \frac{B_1}{B_2}(\alpha_2)\right]$$

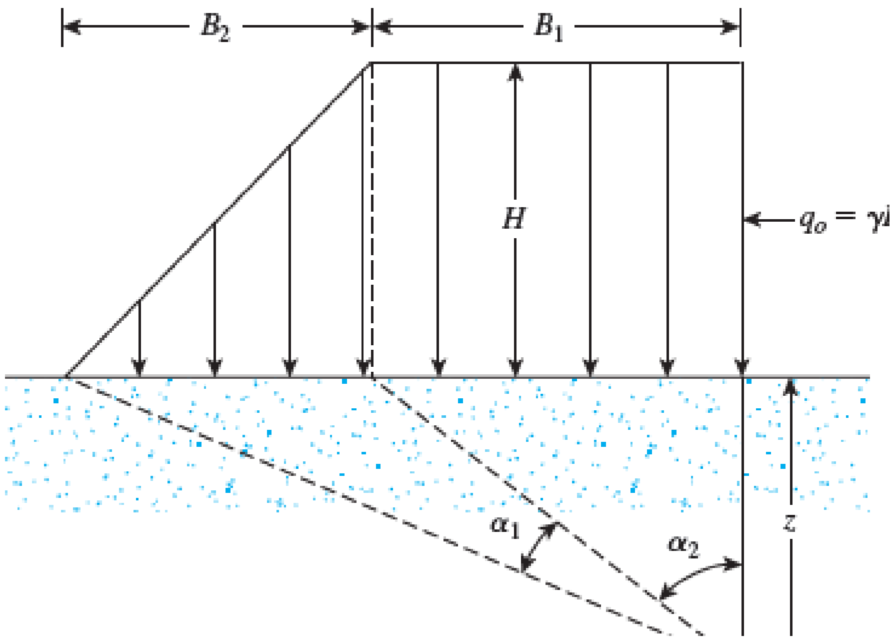
where

$q_o = \gamma H$
γ = unit weight of the embankment soil
H = height of the embankment

$$\alpha_1 = \tan^{-1}\left(\frac{B_1 + B_2}{z}\right) - \tan^{-1}\left(\frac{B_1}{z}\right)$$

$$\alpha_2 = \tan^{-1}\left(\frac{B_1}{z}\right)$$

(Note that α_1 and α_2 are in radians.)



Penurunan Elastis Pondasi Dangkal di Tanah Lempung Jenuh (Poisson Ratio = 0.5)

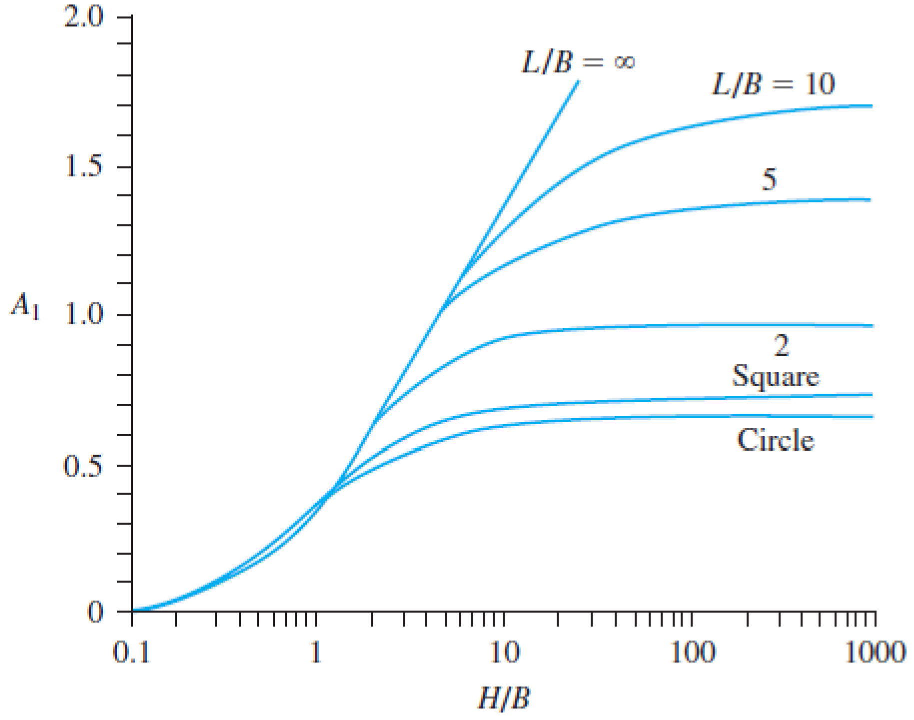
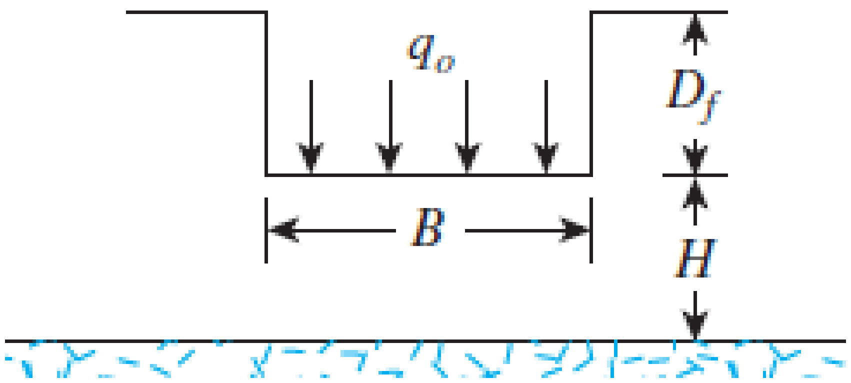
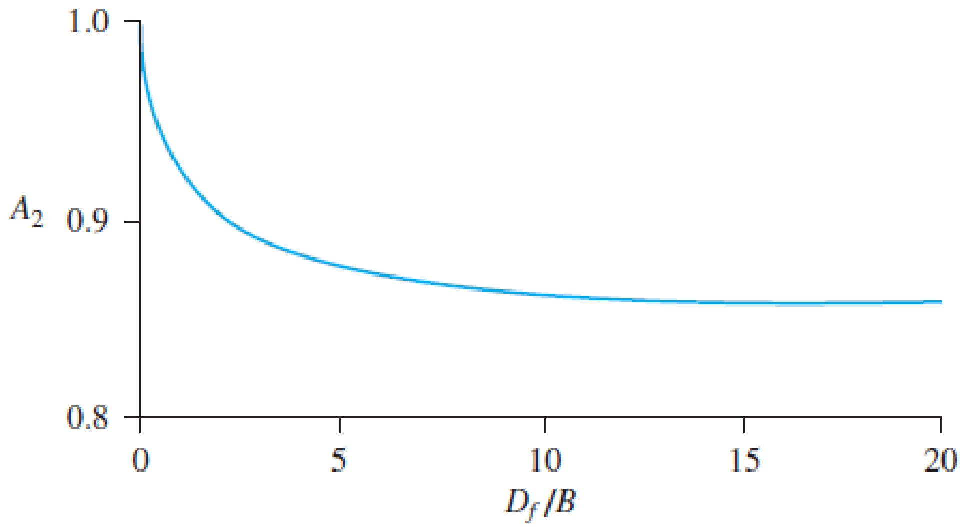


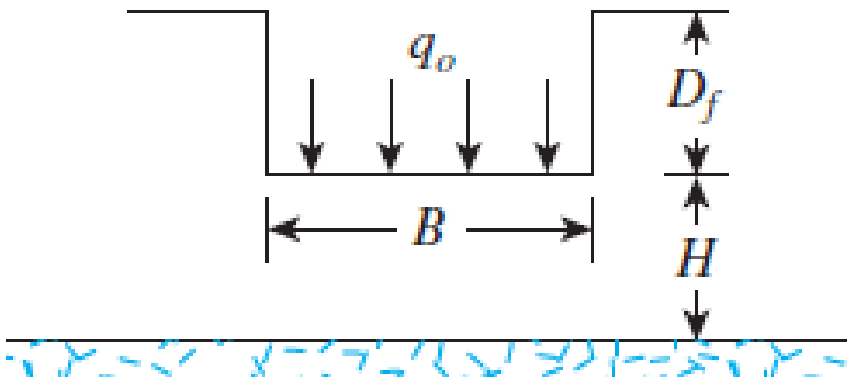
Figure 7.1 Values of A_1 and A_2 for elastic settlement calculation—Eq. (7.1) (After Christian and Carrier, 1978) (Based on Christian, J. T. and Carrier, W. D. (1978). "Janbu, Bjerrum and Kjaernsli's chart reinterpreted," *Canadian Geotechnical Journal*, Vol. 15, pp. 123–128.)



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

Penurunan Elastis Pondasi Dangkal di Tanah Lempung Jenuh (Poisson Ratio = 0.5)





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

Analisis Penurunan di Pondasi Dangkal

Penurunan Elastis Pondasi Dangkal di Tanah Lempung Jenuh (Poisson Ratio = 0.5)

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)

$$E_s = \beta c_u$$

where c_u = undrained shear strength.

Analisis Penurunan di Pondasi Dangkal

Penurunan Elastis Pondasi Dangkal di Tanah Lempung Jenuh (Poisson Ratio = 0.5)

Example 7.1

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

From Eq. (7.1),

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

Given:

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

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

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

$$E_s = \beta c_u$$

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

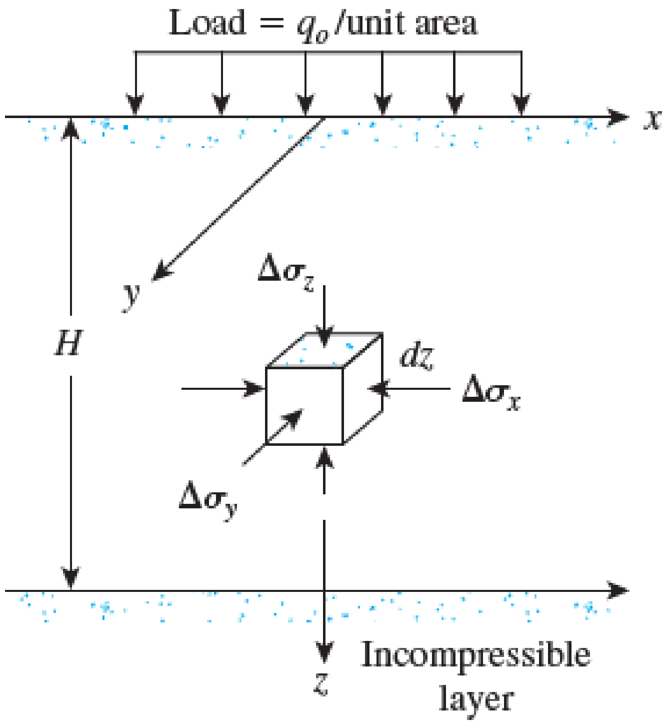
$$E_s = (480)(150) = 72{,}000 \text{ kN/m}^2$$

Also, from Figure 7.1, $A_1 = 0.9$ and $A_2 = 0.92$. Hence,

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

Penurunan Elastis Pondasi Dangkal di Tanah Pasir

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



Penurunan Elastis Pondasi Dangkal di Tanah Pasir

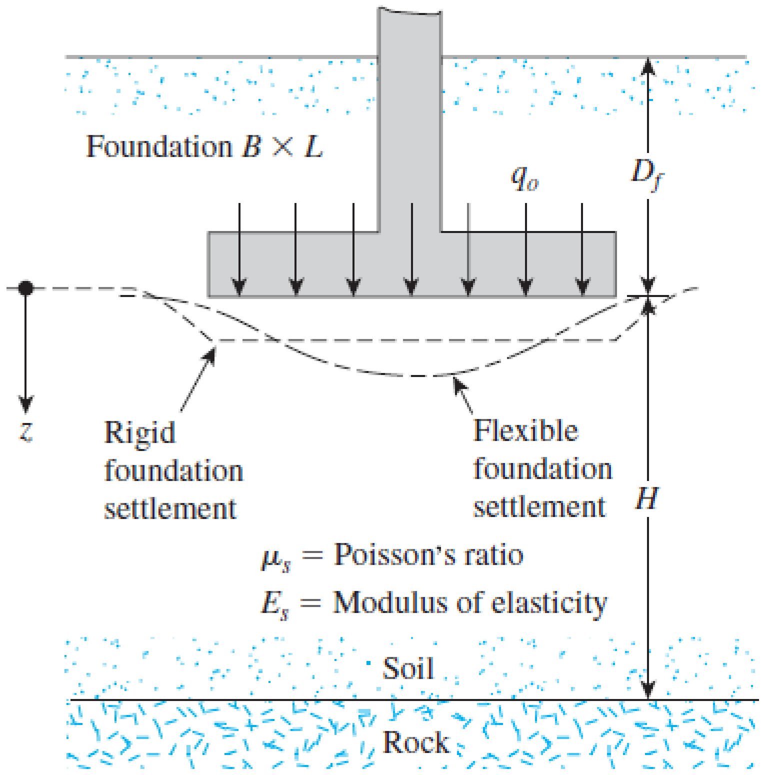


Figure 7.3 Elastic settlement of flexible and rigid foundations

where

q_o = 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 $z = 5B$
$B' = B/2$ for center of foundation
$= B$ for corner of foundation
I_s = shape factor (Steinbrenner, 1934)

Penurunan Elastis Pondasi Dangkal di Tanah Pasir

I_s = shape factor (Steinbrenner, 1934)

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

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

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

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

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

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

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

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

To calculate settlement at the *center* of the foundation, we use

$$\alpha = 4$$

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

and

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

Penurunan Elastis Pondasi Dangkal di Tanah Pasir

To calculate settlement at the *center* of the foundation, we use

$$\alpha = 4$$

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

and

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

To calculate settlement at a *corner* of the foundation,

$$\alpha = 1$$

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

and

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

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

where

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

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

Table 7.4 Variation of I_f with D_f/B, B/L, and μ_s

		B/L		
μ_s	D_f/B	0.2	0.5	1.0
0.3	0.2	0.95	0.93	0.90
	0.4	0.90	0.86	0.81
	0.6	0.85	0.80	0.74
	1.0	0.78	0.71	0.65
0.4	0.2	0.97	0.96	0.93
	0.4	0.93	0.89	0.85
	0.6	0.89	0.84	0.78
	1.0	0.82	0.75	0.69
0.5	0.2	0.99	0.98	0.96
	0.4	0.95	0.93	0.89
	0.6	0.92	0.87	0.82
	1.0	0.85	0.79	0.72

Penurunan Elastis Pondasi Dangkal di Tanah Pasir

Example 7.2

A rigid shallow foundation 1 m × 2 m is shown in Figure 7.4. Calculate the elastic settlement at the center of the foundation.

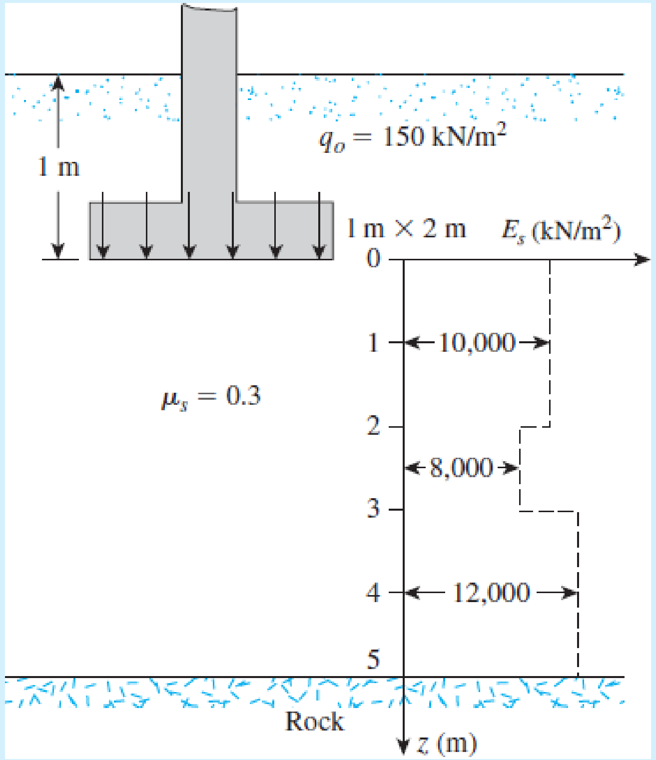


Figure 7.4 Elastic settlement below the center of a foundation

Solution

We are given that $B = 1$ m and $L = 2$ m. Note that $\bar{z} = 5$ m $= 5B$. From Eq. (7.13)

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

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

For the *center of the foundation,*

$$\alpha = 4$$

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

and

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

Analisis Penurunan di Pondasi Dangkal

Penurunan Elastis Pondasi Dangkal di Tanah Pasir

From Tables 7.2 and 7.3, $F_1 = 0.641$ and $F_2 = 0.031$. From Eq. (7.5),

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

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

Again, $D_f/B = 1/1 = 1$, $B/L = 0.5$, and $\mu_s = 0.3$. From Table 7.4, $I_f = 0.71$.

Hence,

$$S_{e(\text{flexible})} = q_0(\alpha B')\frac{1 - \mu_s^2}{E_s} I_s I_f$$

$$= (150)\left(4 \times \frac{1}{2}\right)\left(\frac{1 - 0.3^2}{10{,}400}\right)(0.716)(0.71) = 0.0133\,\text{m} = 13.3\,\text{mm}$$

Since the foundation is rigid, from Eq.(7.12) we obtain

$$S_{e(\text{rigid})} = (0.93)(13.3) = \mathbf{12.4\,mm}$$

Analisis Penurunan di Pondasi Dangkal

Penurunan Konsolidasi

$$S_{c(p)} = \frac{C_c H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$ (for normally consolidated clays) [Eq. (2.65)]

$$S_{c(p)} = \frac{C_s H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}$$ (for overconsolidated clays with $\sigma'_o + \Delta\sigma'_{av} < \sigma'_c$) [Eq. (2.67)]

$$S_{c(p)} = \frac{C_s H_c}{1+e_o} \log \frac{\sigma'_c}{\sigma'_o} + \frac{C_c H_c}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_c}$$ (for overconsolidated clays with $\sigma'_o < \sigma'_c < \sigma'_o + \Delta\sigma'_{av}$) [Eq. (2.69)]

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

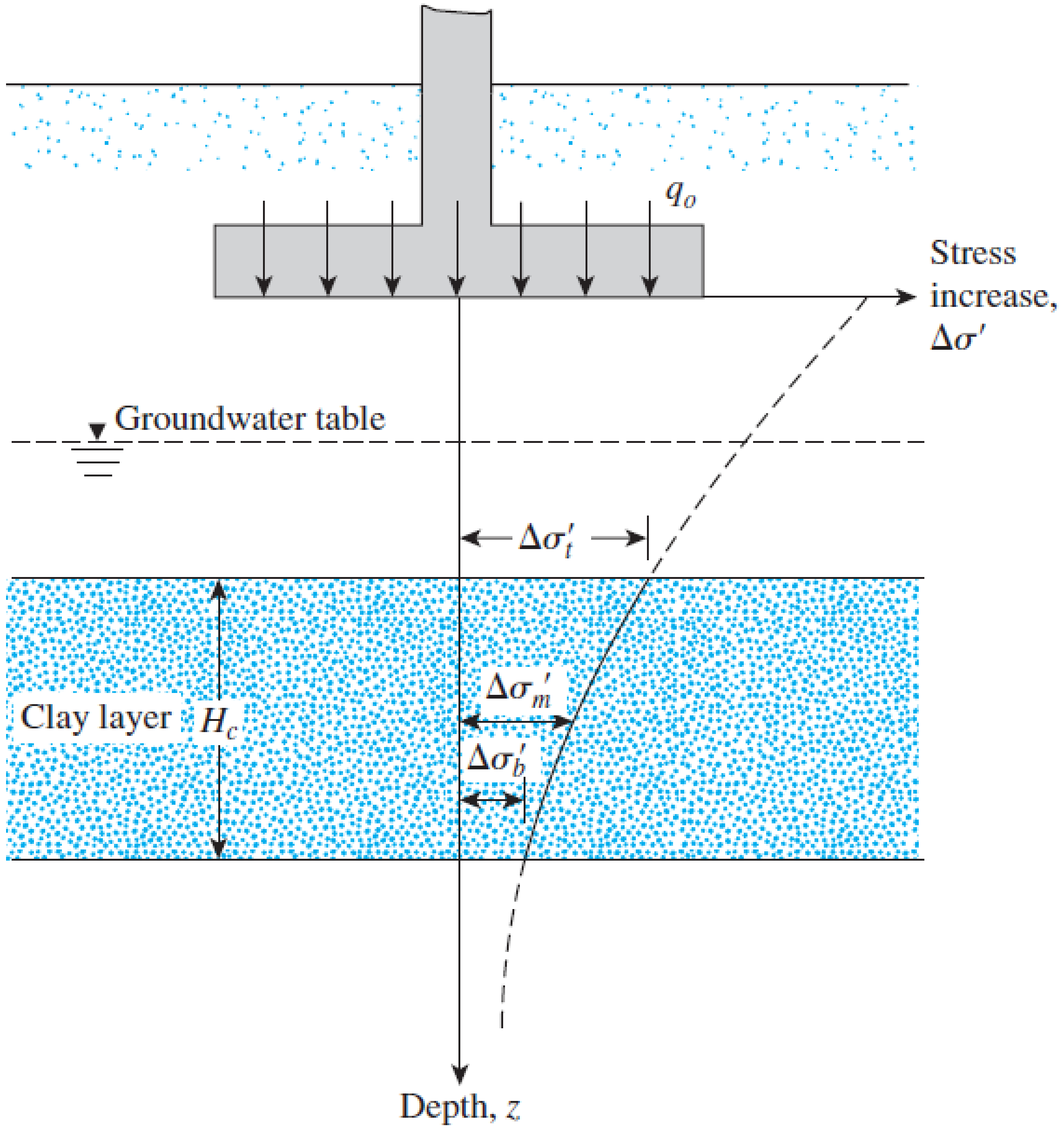


Figure 7.20 Consolidation settlement calculation

The increase in effective pressure, Δσ' on the clay layer is not Constant with depth : the magnitude of Δσ' will decrease with increase in depth measured from the bottom of the foundation.

The average in pressure may be approximated by :

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

The method of determining the pressure increase caused by various types of foundation load using Boussinesq's solution

Penurunan Total

Penurunan Total = Penurunan Elastik + Penurunan Konsolidasi