B-44

MATERY OF SOIL MECHANIS -2

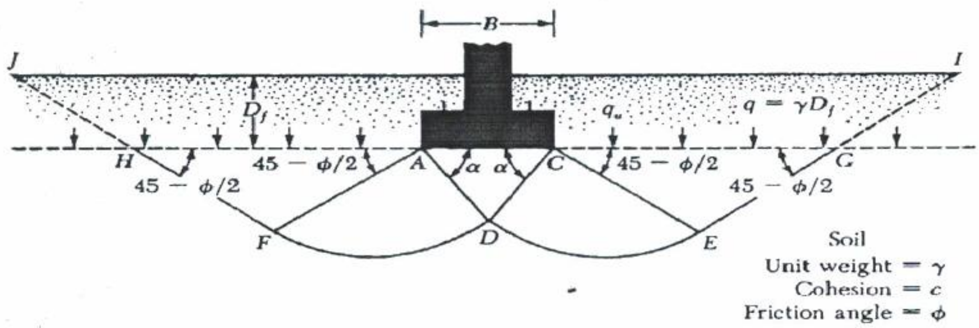
(Diktat Mekanika Tanah-2)

MODUL - 1:

Shear Strength

Bearing Capacity

Consolidation and Settlement



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1 st Edition April 2012

I. KUAT GESER TANAH / SHEAR STRENGTH OF SOIL

The *shear strength* of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. Engineers must understand the nature of shearing resistance in order to analyze soil stability problems such as bearing capacity, slope stability, and lateral pressure on earth-retaining structures.

Mohr-Coulomb Failure Criteria

Mohr (1900) presented a theory for rupture in materials. This theory contended that a material fails because of a critical combination of normal stress and shear stress, and not from either maximum normal or shear stress alone. Thus, the functional relationship between normal stress and shear stress on a failure plane can be expressed in the form

$$\tau_f = f(\sigma) \tag{8.1}$$

where

τ_f = shear stress on the failure plane
σ = normal stress on the failure plane

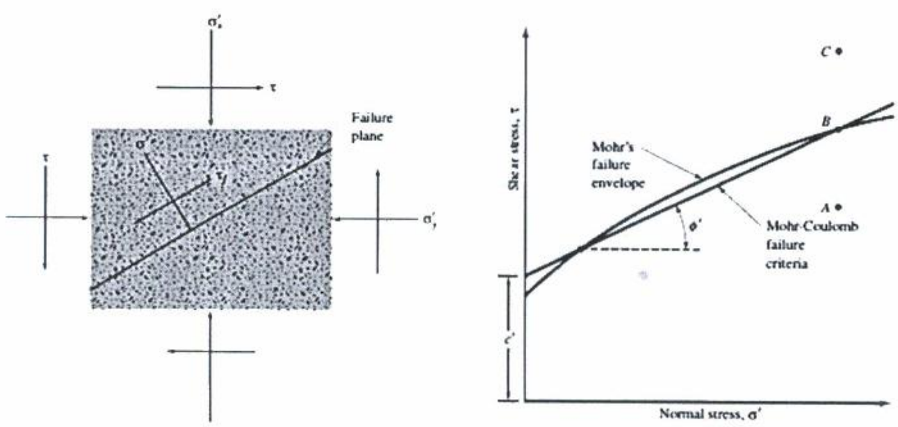
The failure envelope defined by Eq. (8.1) is a curved line. For most soil mechanics problems, it is sufficient to approximate the shear stress on the failure plane as a linear function of the normal stress (Coulomb, 1776). This relation can be written as

$$\tau_f = c + \sigma \tan \phi \tag{8.2}$$

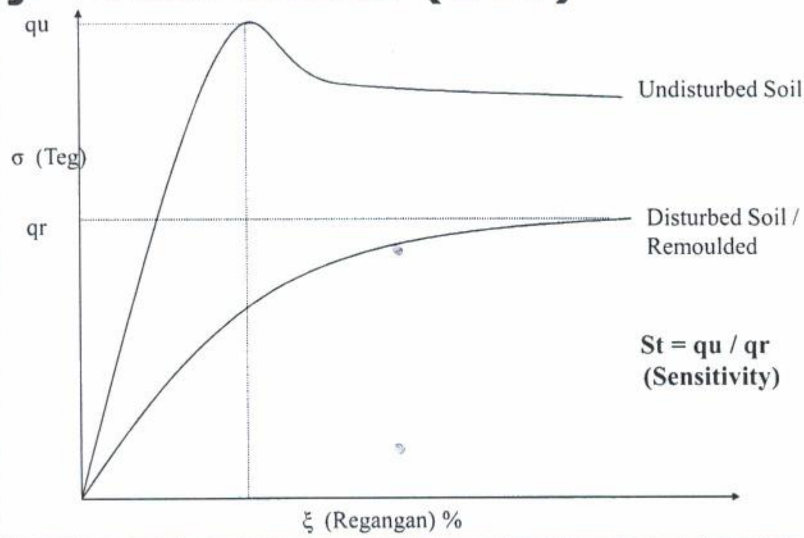
where

c = cohesion
ϕ = angle of internal friction

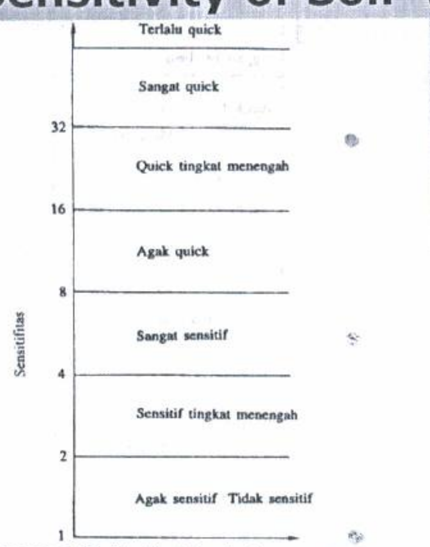
The preceding equation is called the *Mohr-Coulomb failure criteria*.



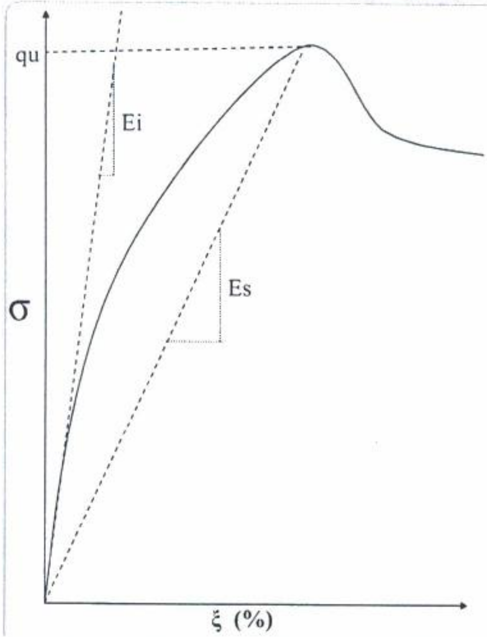
Uji Tekan Bebas (UCT)



Sensitivity of Soil (St)



Gambar 9-35 Klasifikasi tanah lempung berdasarkan kesensitifannya..



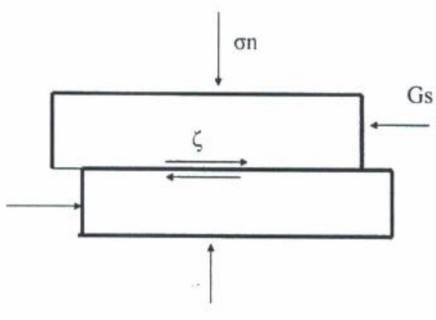
OUTPUT :

Cu = qu / 2

Ei = Initial Tangens Modulus

Es = Secant Modulus

Direct Shear Test



Pengujian Kuat Geser Tanah dng Direct Shear Test

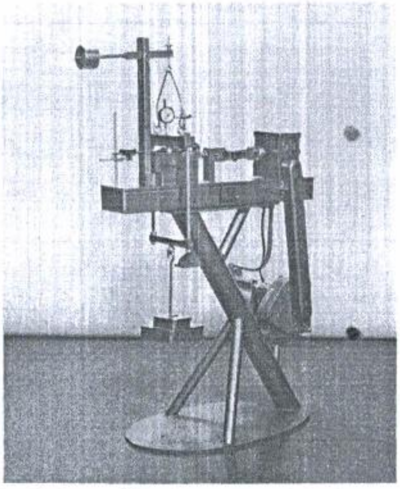
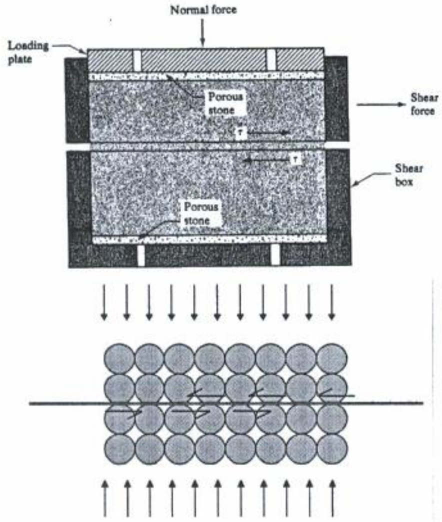


Figure 8.5 Strain-controlled direct shear test equipment (courtesy of Soiltest, Inc., Evanston, Illinois)



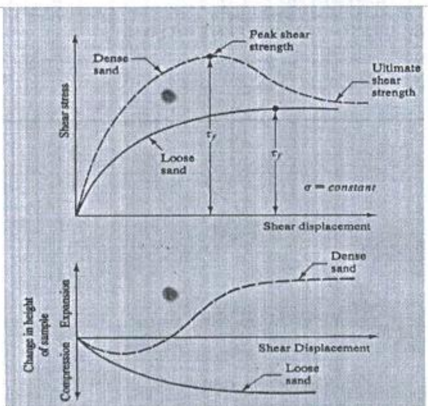


Figure 8.6 Plot of shear stress and change in height of sample against shear displacement for loose and dense dry sand (direct shear test)

$$\sigma = \text{normal stress} = \frac{\text{normal force}}{\text{are of cross-section of the sample}}$$

$$\tau = \text{shear strength} = \frac{\text{resisting shear force}}{\text{are of cross-section of the sample}}$$

Triaxial Testing

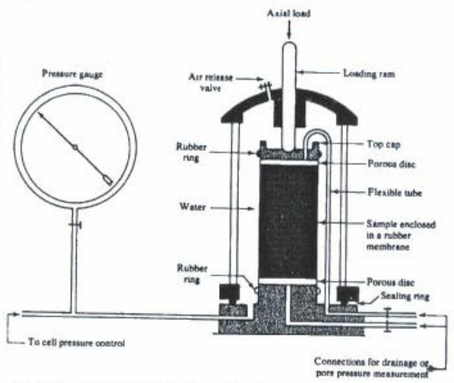


Figure 8.11 Diagram of triaxial test equipment (after Bishop and Bjerrum, 1960)

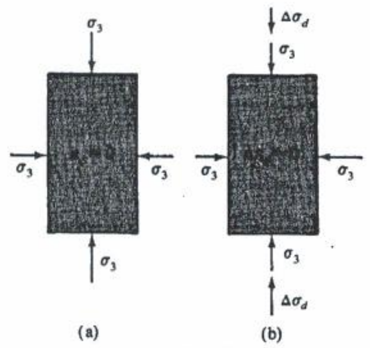


Figure 8.12 Consolidated-drained triaxial test: (a) sample under chamber confining pressure; (b) deviator stress application

Type Triaxial Test di Laboratorium

- UU Test
  Uncosolidated Undrained
- CU Test
  Consolidated Undrained
- CD Test
  Consolidated Drained

Tegangan-tegangan untuk pembuatan grafik Mohr Coulomb. Triaxial CU Test

- CU TEST

$\sigma 11' = (\Delta\sigma 11 + \sigma 31) - u1$

$\sigma 31' = \sigma 31 - u1$

u = Diukur

$\sigma 12' = (\Delta\sigma 12 + \sigma 32) - u2$

$\sigma 32' = \sigma 32 - u2$

$\sigma 13' = (\Delta\sigma 13 + \sigma 33) - u3$

$\sigma 33' = \sigma 33 - u3$

Tegangan-tegangan untuk pembuatan grafik Mohr Coulomb. Triaxial CD Test

- CD TEST

$\sigma 11' = (\Delta\sigma 11 + \sigma 31) - u1$

$\sigma 31' = \sigma 31 - u1$

u1 = u2 = u3 = 0

$\sigma 12' = (\Delta\sigma 12 + \sigma 32) - u2$ (Karena drained)

$\sigma 32' = \sigma 32 - u2$

$\sigma 13' = (\Delta\sigma 13 + \sigma 33) - u3$

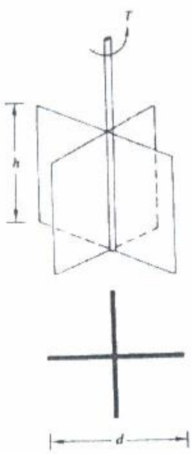
$\sigma 33' = \sigma 33 - u3$

UJI KUAT GESER DI LAPANGAN

- UJI GESER VANE (BALING-BALING)

Uji ini khusus dilakukan pada kondisi lapisan tanah lempung lunak, dimana pertimbangannya adalah bahwa bila dilakukan pengambilan sample dengan tabung, maka akan terjadi disturbance / ketergangguan sample uji, sehingga tidak akurat lagi bila dikatakan **undisturbed soil**

VANE SHAER TEST



Gambar 9-38 Gambar dari alat geser vane.

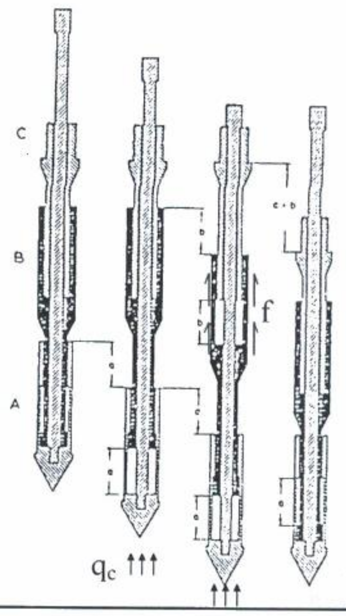
$$T = \pi c_u \left[\frac{d^2 h}{2} + \beta \frac{d^3}{4} \right]$$

$$c_u = \frac{T}{\pi \left[\frac{d^2 h}{2} + \beta \frac{d^3}{4} \right]}$$

β = 1/2 bila tahanan geser termobilisasi dianggap berbentuk segitiga
β = 2/3 bila tahanan geser termobilisasi dianggap berbentuk seragam
β = 3/5 bila tahanan geser termobilisasi dianggap berbentuk parabola

KORELASI SHEAR STRENGHT PARAMETER DENGAN DATA UJI LAPANGAN (qc dan N SPT)

Sondir



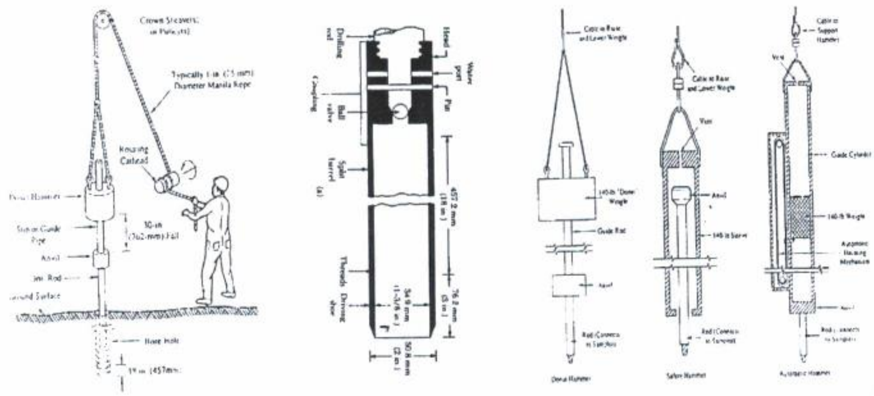
C = q_c / 20 s/d 30

C (kg/cm2) = qc(kg/cm2) / 20

C (t/m2) = qc(kg/cm2) / 2

SPT (Standard Penetration Test)

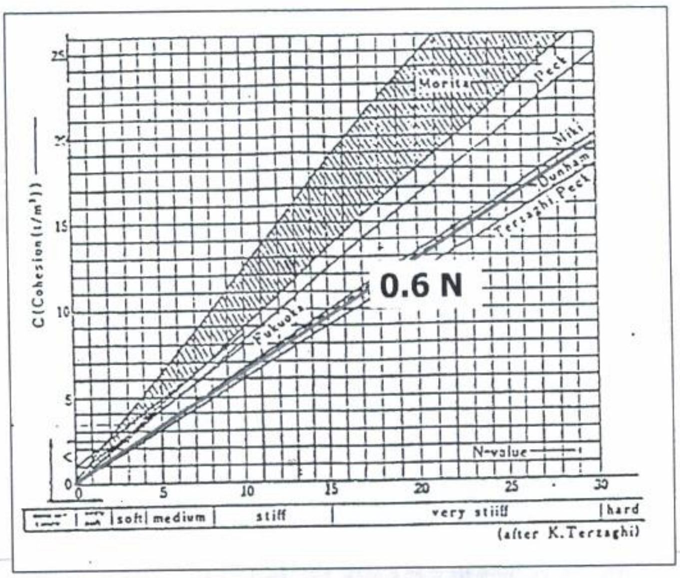
Cara uji SPT | **Jenis Hammer**

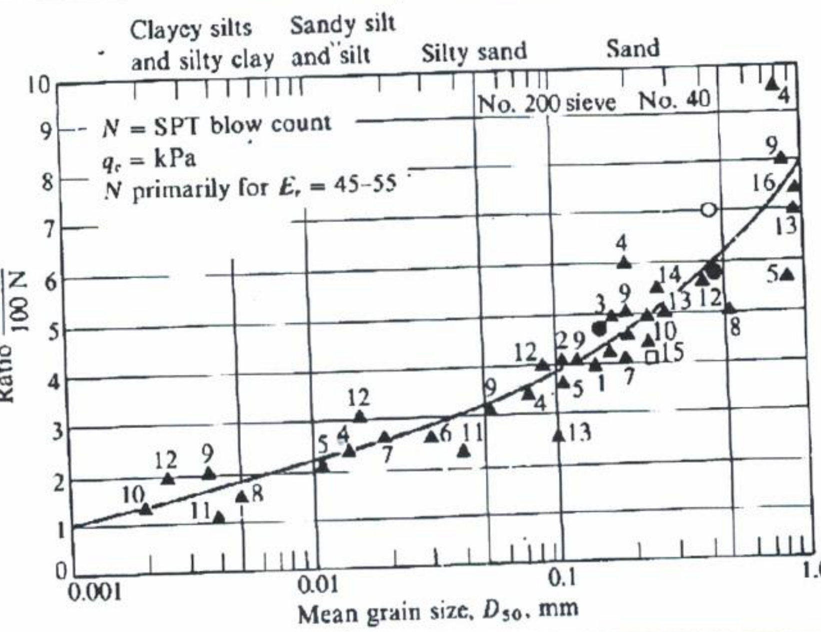


N-SPT = Jumlah pukulan untuk memasukkan split spoon sedalam 30 cm

C (t/m2) = 0.6 x N

Relationship between Cohesion and N-Value (Cohesive soil)



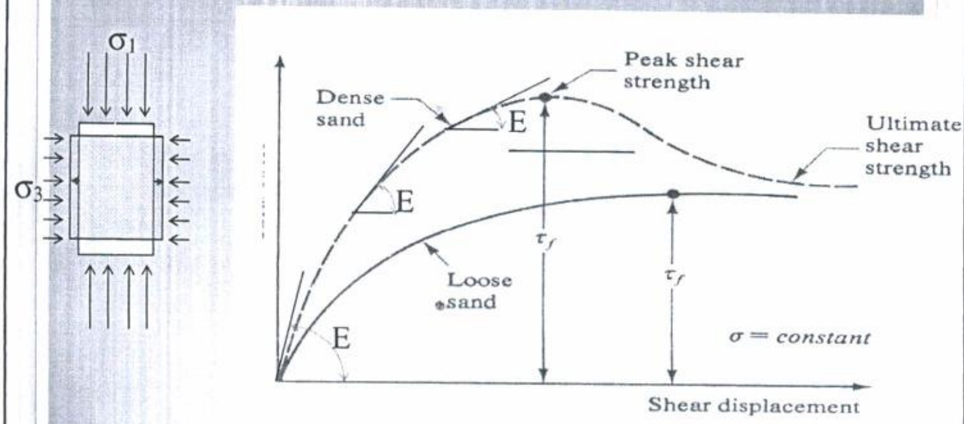


Apakah Test Lapangan mencerminkan

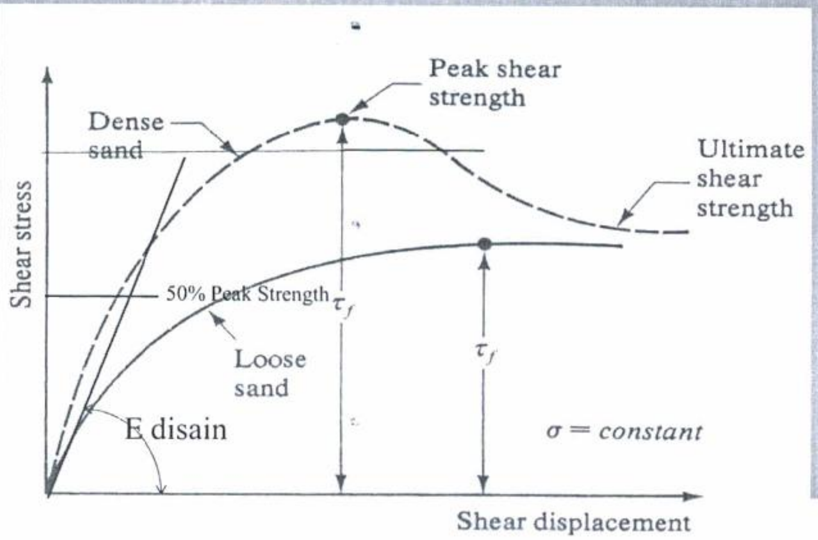
- Triaxial CD?
- Triaxial CU?
- Triaxial UU?

Mana yg paling sesuai?

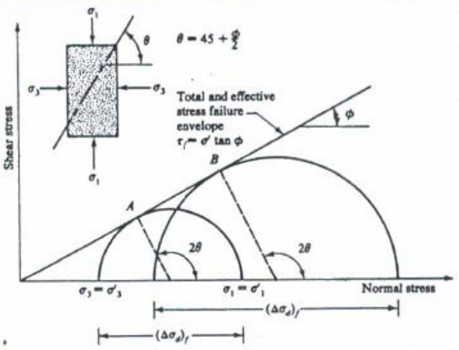
Modulus Young

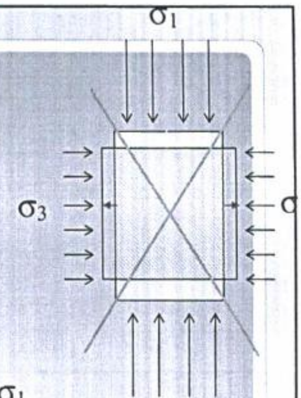


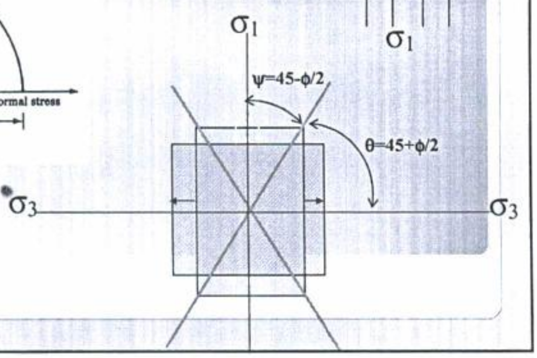
Modulus Young



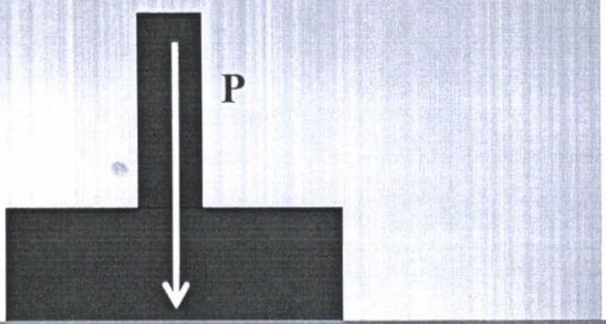
Arah Bidang Runtuh Terhadap σ_1



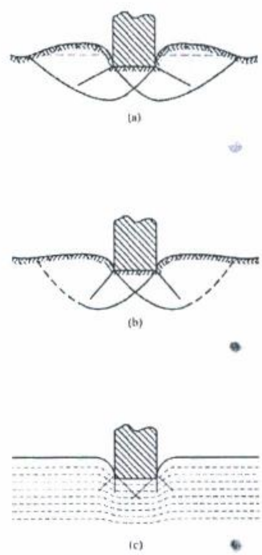




Daya Dukung Tanah (Pondasi Dangkal)



Tipe-tipe Keruntuhan



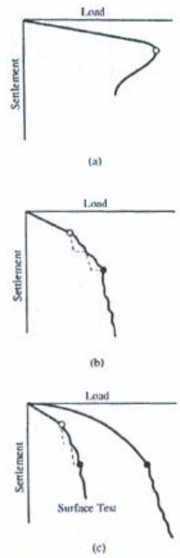
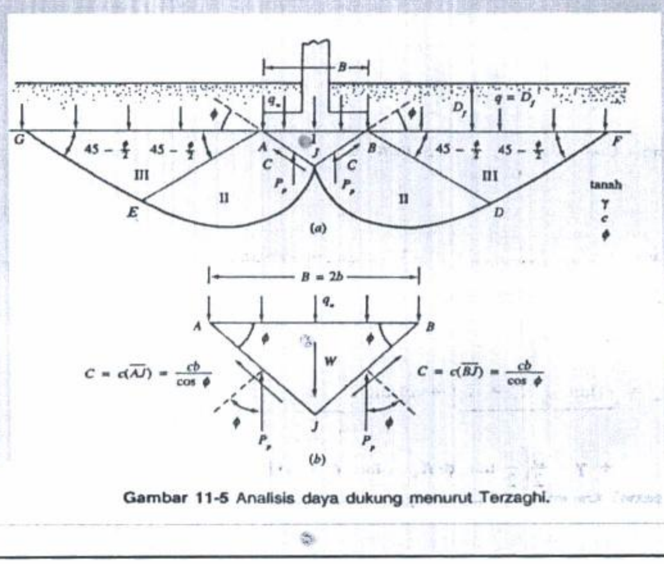


Figure 6.6 Modes of bearing capacity failures beneath spread footings: (a) general shear failure, (b) local shear failure, and (c) punching shear failure (Adapted from Vesić, 1963).

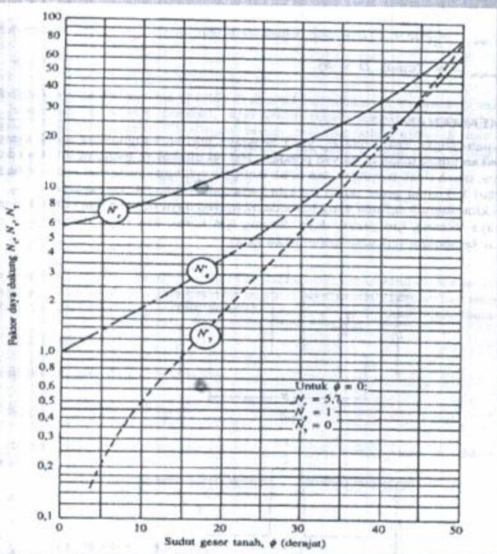
Figure 6.7 Typical load-displacement curves for different modes of bearing capacity failure: (a) general shear failure, (b) local shear failure, and (c) punching shear failure (Adapted from Vesić, 1963).

ANALISA DAYA DUKUNG MENURUT TERZAGHI

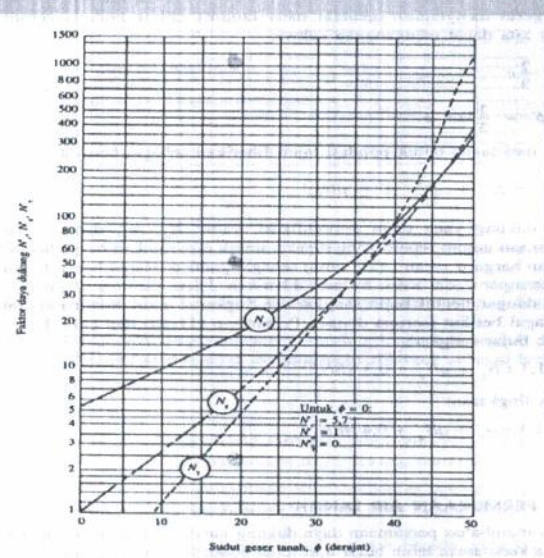


Gambar 11-5 Analisis daya dukung menurut Terzaghi.

Factor Daya Dukung Untuk Geser Setempat



Factor Daya Dukung Untuk Geser Umum



Terzaghi

Using the equilibrium analysis, Terzaghi expressed the ultimate bearing capacity in the form

$$q_u = cN_c + qN_q + \tfrac{1}{2}\gamma BN_\gamma \quad \text{(strip foundation)} \tag{3.3}$$

where c = cohesion of soil
γ = unit weight of soil
$q = \gamma D_f$
N_c, N_q, N_γ = bearing capacity factors that are nondimensional and are only functions of the soil friction angle, ϕ

The bearing capacity factors, N_c, N_q, and N_γ are defined by

$$N_c = \cot\phi \left[\frac{e^{2(3\pi/4-\phi/2)\tan\phi}}{2\cos^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)} - 1 \right] = \cot\phi(N_q - 1) \tag{3.4}$$

$$N_q = \frac{e^{2(3\pi/4-\phi/2)\tan\phi}}{2\cos^2\left(45+\frac{\phi}{2}\right)} \tag{3.5}$$

$$N_\gamma = \frac{1}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi} - 1\right)\tan\phi \tag{3.6}$$

where $K_{p\gamma}$ = passive pressure coefficient

$$q_u = cN_c + qN_q + \tfrac{1}{2}\gamma BN_\gamma \quad \text{(strip foundation)} \tag{3.3}$$

where c = cohesion of soil
γ = unit weight of soil
$q = \gamma D_f$
N_c, N_q, N_γ = bearing capacity factors that are nondimensional and are only functions of the soil friction angle, ϕ

The bearing capacity factors, N_c, N_q, and N_γ are defined by

$$N_c = \cot\phi \left[\frac{e^{2(3\pi/4-\phi/2)\tan\phi}}{2\cos^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)} - 1 \right] = \cot\phi(N_q - 1) \tag{3.4}$$

$$q_u = 1.3cN_c + qN_q + 0.4\gamma BN_\gamma \quad \text{(square foundation)} \tag{3.7}$$

$$q_u = 1.3cN_c + qN_q + 0.3\gamma BN_\gamma \quad \text{(circular foundation)} \tag{3.8}$$

Example
3.2

Repeat Example Problem 3.1, assuming local shear failure occurs in the soil supporting the foundation.

Solution

From Eq. (3.10)

$$q_u = 0.867cN'_c + qN'_q + 0.4\gamma BN'_\gamma$$

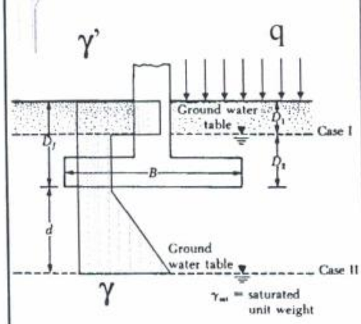
From Figure 3.5, for $\phi = 20°$

$$N'_c = 12$$
$$N'_q = 4$$
$$N'_\gamma = 1.7$$

So

$$q_u = (0.867)(15.2)(12) + (1 \times 17.8)(4) + (0.4)(17.8)(1.5)(1.7)$$
$$= 158.1 + 71.2 + 18.2 = 247.5 \text{ kN/m}^2$$
$$q_{all} = \frac{247.5}{4} = 61.9 \text{ kN/m}^2$$

Allowable gross load $= Q = (q_{all})(B^2) = (61.9)(1.5^2) = \underline{139 \text{ kN}}$



Case I

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

$$q = \text{effective surcharge} = D_1\gamma + D_2(\gamma_{sat} - \gamma_w) \qquad (3.12)$$

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

Case II

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

$$q = \gamma D_f \qquad (3.13)$$

The factor γ in the last term of the bearing capacity equations must be replaced by the factor

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma') \qquad (3.14)$$

Case III

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

Bearing Capacity Factors

Based on laboratory and field studies of bearing capacity, the basic nature of the failure surface in soil suggested by Terzaghi now appears to be correct (Vesic, 1973). However, the angle α as shown in Figure 3.5 is closer to $45 + \phi/2$ than to ϕ. If this change is accepted, the values of N_c, N_q, and N_γ for a given soil friction angle will also change from those given in Table 3.1. With $\alpha = 45 + \phi/2$, the relations for N_c and N_q can be derived as

$$N_q = \tan^2\left(45 + \frac{\phi}{2}\right) e^{\pi \tan\phi} \qquad (3.26)$$

$$N_c = (N_q - 1)\cot\phi \qquad (3.27)$$

The equation for N_c given by Eq. (3.27) was originally derived by Prandtl (1921), and the relation for N_q [Eq. (3.26)] was presented by Reissner (1924). Caquot and Kerisel (1953) and Vesic (1973) gave the relation for N_γ as

$$N_\gamma = 2(N_q + 1)\tan\phi \qquad (3.28)$$

Harga-harga Faktor bentuk, Faktor Kedalaman dan Faktor Kemiringan Beban

TABEL 11-2 Harga-harga Faktor Bentuk, Faktor Kedalaman dan Faktor Kemiringan

Faktor bentuk untuk pondasi bentuk persegi (B = lebar pondasi; L = panjang pondasi)
$\lambda_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$ $\lambda_{qs} = 1 + \left(\frac{B}{L}\right)(\tan\phi)$ $\lambda_{\gamma s} = 1 - 0{,}4\left(\frac{B}{L}\right)$
Faktor bentuk untuk pondasi bentuk lingkaran dan bujur sangkar
$\lambda_{cs} = 1 + \frac{N_q}{N_c}$ $\lambda_{qs} = 1 + \tan\phi$ $\lambda_{\gamma s} = 0{,}6$
Faktor kedalaman $\frac{D_f}{B} \leq 1$
$\lambda_{qd} = 1 + 2\tan\phi(1 - \sin\phi)^2\left(\frac{D_f}{B}\right)$ $\lambda_{cd} = \lambda_{qd} - \frac{1 - \lambda_{qd}}{N_q \cdot \tan\phi}$ $\lambda_{\gamma d} = 1$
Faktor kedalaman untuk $\phi = 0$
$\lambda_{cd} = 1 + 0{,}4\left(\frac{D_f}{B}\right)$
Faktor kedalaman untuk $\frac{D_f}{B} > 1$
$\lambda_{qd} = 1 + 2\tan\phi(1 - \sin\phi)^2\tan^{-1}\left(\frac{D_f}{B}\right)$ $\lambda_{cd} = \lambda_{qd} - \frac{1 - \lambda_{qd}}{N_q \cdot \tan\phi}$ $\lambda_{\gamma d} = 1$
Faktor kedalaman untuk $\phi = 0$
$\lambda_{cd} = 1 + 0{,}4\tan^{-1}\left(\frac{D_f}{B}\right)$
Faktor kemiringan
$\lambda_{ci} = \left(1 - \frac{\alpha^\circ}{90^\circ}\right)^2$ $\lambda_{qi} = \left(1 - \frac{\alpha^\circ}{90^\circ}\right)^2$ $\lambda_{\gamma i} = \left(1 - \frac{\alpha^\circ}{\phi^\circ}\right)$

Example 3.3

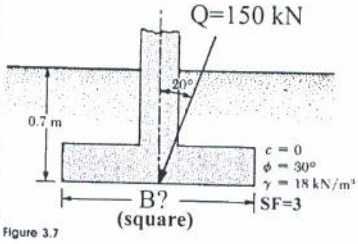


Figure 3.7

Solution

With $c = 0$, the ultimate bearing capacity becomes

$$q_u = qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i} \quad (3.1$$

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

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

Table 3.2, for $\phi = 30°$

$$N_q = 18.4$$

$$N_\gamma = 22.4$$

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

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

$$F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2\frac{D_f}{B} = 1 + \frac{(0.289)(0.7)}{B} = 1 + \frac{0.202}{B}$$

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

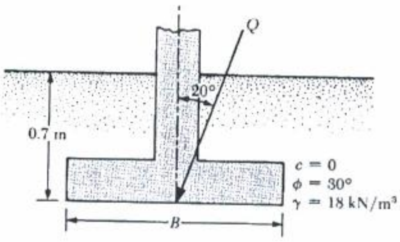
$$F_{qi} = \left(1 - \frac{\beta°}{90°}\right)^2 = \left(1 - \frac{20}{90}\right)^2 = 0.605$$

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

Hence

$$q_u = (12.6)(18.4)(1.577)\left(1 + \frac{0.202}{B}\right)(0.605) + (0.5)(18)(B)(22.4)(0.6)(1)(0.11)$$

$$= 221.2 + \frac{44.68}{B} + 13.3B$$



Thus

$$q_{all} = \frac{q_u}{3} = 73.73 + \frac{14.89}{B} + 4.43B \quad \text{(b)}$$

Given Q = total allowable load = $q_{all} \times B^2$ or

$$q_{all} = \frac{150}{B^2} \quad \text{(c)}$$

Equating the right-hand sides of Eqs. (b) and (c)

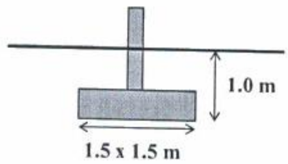
$$\frac{150}{B^2} = 73.73 + \frac{14.89}{B} + 4.43B$$

By trial and error, $B \approx$ 1.3 m

Example
3.1

A square foundation is 1.5 m × 1.5 m in plan. The soil supporting the foundation has a friction angle of $\phi = 20°$ and $c = 15.2$ kN/m². The unit weight of soil, γ, is 17.8 kN/m³. Determine the allowable gross load on the foundation with a factor of safety (*FS*) of 4. Assume the depth of the foundation (D_f) to be one meter, and general shear failure occurs in soil.

Solution



From Eq. (3.7)

$$q_u = 1.3cN_c + qN_q + 0.4\gamma BN_\gamma$$

From Figure 3.4, for $\phi = 20°$

$$N_c = 17.7$$
$$N_q = 7.4$$
$$N_\gamma = 5$$

Thus

$$q_u = (1.3)(15.2)(17.7) + (1 \times 17.8)(7.4) + (0.4)(17.8)(1.5)(5)$$
$$= 349.75 + 131.72 + 53.4 = 534.87 \approx 535 \text{ kN/m}^2$$

So, allowable load per unit area of the foundation =

$$q_{all} = \frac{q_u}{FS} = \frac{535}{4} = 133.75 \text{ kN/m}^2$$

Thus, the total allowable gross load

$$Q = (133.75)B^2 = (133.75)(1.5 \times 1.5) = 300.9 \approx \underline{300 \text{ kN}}$$

The factor of safety as defined by Eq. (3.40) may be referred to as the *net allowable bearing capacity*. This should be kept at least about 3 in all cases.

Another type of factor of safety for the bearing capacity of shallow foundations is often used. This is the factor of safety with respect to shear failure (FS_{shear}). In most cases, a value of FS_{shear} = 1.4–1.6 is desirable along with a *minimum* factor of safety of 3–4 against gross or net ultimate bearing capacity. In order to calculate the net allowable load on the basis of a given FS_{shear}, the following procedure should be adopted:

1. Let c and ϕ be the cohesion and the angle of friction of soil, and let FS_{shear} be the required factor of safety with respect to shear failure. So, the developed cohesion and the angle of friction can be given as

$$c_d = \frac{c}{FS_{shear}} \qquad (3.41)$$

$$\phi_d = \tan^{-1}\left(\frac{\tan\phi}{FS_{shear}}\right) \qquad (3.42)$$

2. The gross allowable bearing capacity can now be calculated according to Eqs. (3.3), (3.7), (3.8) or the general bearing capacity equation [Eq. (3.16)] using c_d and ϕ_d as the shear strength parameters of the soil. For example, the gross allowable bearing capacity of a continuous foundation according to Terzaghi's equation can be written as

$$q_{all} = c_d N_c + qN_q + \tfrac{1}{2}\gamma BN_\gamma \qquad (3.43)$$

where N_c, N_q, and N_γ = bearing capacity factors for friction angle, ϕ_d

3. The net allowable bearing capacity is thus

$$q_{net(all)} = q_{all} - q = c_d N_c + q(N_q - 1) + \tfrac{1}{2}\gamma BN_\gamma \qquad (3.44)$$

pressure distribution on the soil will be as shown in Figure 3.8a. The value of q_{max} can be given by the expression

$$q_{max} = \frac{4Q}{3L(B - 2e)} \tag{3.49}$$

2. Determine the effective dimensions of the foundation as

B' = effective width = $B - 2e$
L' = effective length = L

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

3. Use Eq. (3.16) for the ultimate bearing capacity as

$$q'_u = cN_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma B'N_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i} \tag{3.50}$$

For the evaluation of F_{cs}, F_{qs}, $F_{\gamma s}$, F_{ci}, F_{qi}, and $F_{\gamma i}$, Equations (3.20) to (3.22) and Eqs. (3.29) to (3.33) have to be used with *effective length* and *effective width* dimensions in place of L and B, respectively.

For determination of F_{cd}, F_{qd}, and $F_{\gamma d}$, use Equations (3.23) to (3.28) (do not replace B with B').

4. The total ultimate load that the foundation can sustain is

$$Q_{ult} = q'_u(B')(L') \tag{3.51}$$

5. The factor of safety against bearing capacity failure is given as

$$FS = \frac{Q_{ult}}{Q} \tag{3.52}$$

As we can see, eccentricity tends to decrease the load-bearing capacity of a foundation. In such cases, it is probably advantageous to place the foundation columns off center, as shown in Figure 3.9. This, in effect, produces a centrally loaded foundation with uniformly distributed pressure.

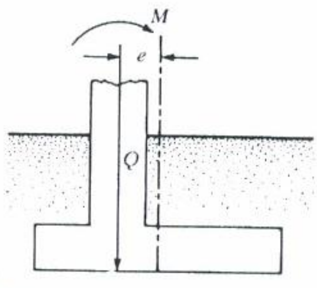
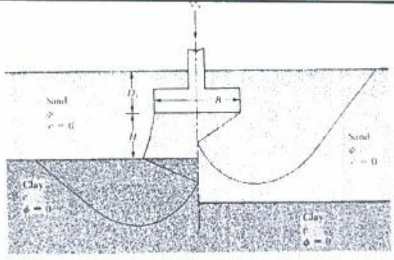
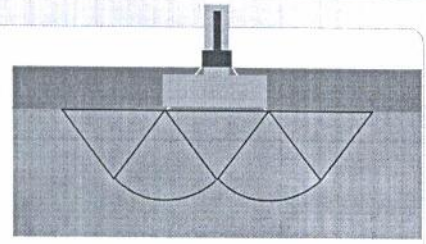


Figure 3.9 Foundation of columns with off-center loading





Pondasi Lajur

$$q_u = \boxed{cN_c} + \boxed{\gamma H^2\left(1 + \frac{2D_f}{H}\right) K_s \frac{\tan\phi}{B} + \gamma D_f}$$

Clay portion / **Sand portion**

with a maximum of

$$q_u = \frac{1}{2}\gamma B N_\gamma + \gamma D_f N_q$$

where ϕ = angle of friction of top sand layer
γ = unit weight of sand
K_s = punching shear resistance coefficient

Table 3.3 Values of K_s for Use in Eqs. (3.54) and (3.56)

Friction angle of sand, ϕ (deg)	K_s
20	1.89
25	2.22
30	3.06
35	4.45
40	6.95
45	11.12
50	19.15

Pondasi segi empat

with a maximum of

$$q_u = \frac{1}{2}\left(1 - 0.4\frac{B}{L}\right)\gamma B N_\gamma + \gamma D_f N_q$$

Settlement

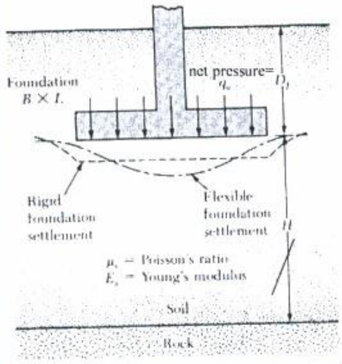


Figure 3.12 Elastic settlement of flexible and rigid foundations

$$S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\frac{\alpha}{2} \quad \text{(corner of the flexible foundation)} \qquad (3.58)$$

$$S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\alpha \quad \text{(center of the flexible foundation)} \qquad (3.59)$$

$$\text{where } \alpha = \frac{1}{\pi}\left[\ln\left(\frac{\sqrt{1+m^2}+m}{\sqrt{1+m^2}-m}\right) + m\ln\left(\frac{\sqrt{1+m^2}+1}{\sqrt{1+m^2}-1}\right)\right] \qquad (3.60)$$

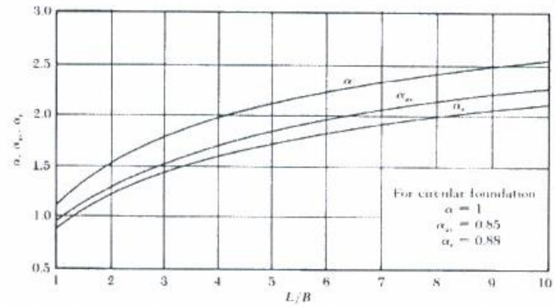
$m = B/L$ (3.61)

B = width of foundation

L = length of foundation

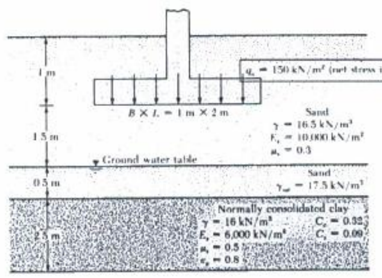
$$S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\alpha_{av} \quad \text{(average for flexible foundation)} \qquad (3.62)$$

$$S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\alpha_r \quad \text{(rigid foundation)} \qquad (3.63)$$



3.12

A foundation 1 m × 2 m in plan is shown in Figure 3.33. Estimate the total settlement of the foundation.



Elastic Settlement

The clay layer is located at a depth of 2 m—that is, $2B$ below the foundation. From Figure 3.15 on p. 128, it can be seen that the soil located at a depth $z > 2B$ has very little influence on the elastic settlement. Hence, if Eq. (3.63) is used for the elastic settlement calculation, it is reasonable to use the Young's modulus and Poisson's ratio values of the sand layer. Thus

$$S_e = \frac{Bq_o}{E_s}(1-\mu_s^2)\alpha_r \longleftarrow \textbf{rigid foundation}$$

Given: $q_o = 150\ \text{kN/m}^2$, $E_s = 10{,}000\ \text{kN/m}^2$, $\mu_s = 0.3$, and $\alpha_r \approx 1.2$ (Figure 3.13b). So

$$S_e = \frac{(1)(150)}{10{,}000}(1-0.3^2)(1.2) = 0.0163\ \text{m} = \underline{16.38\ \text{mm}}$$

For Saturated Clay: $\upsilon=0.5$

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

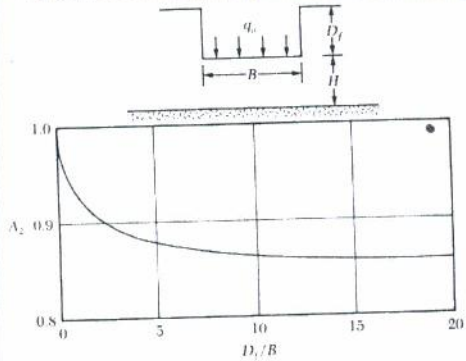


Figure 3.14 Values of A_1 and A_2 for elastic settlement calculation—Eq. (3.64) (after Christian and Carrier, 1978)

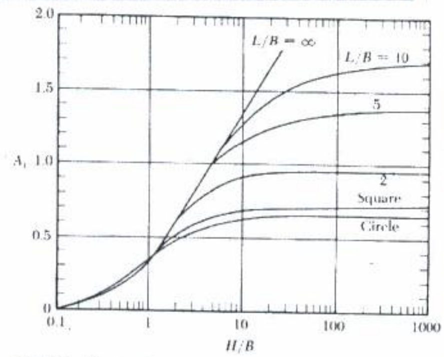
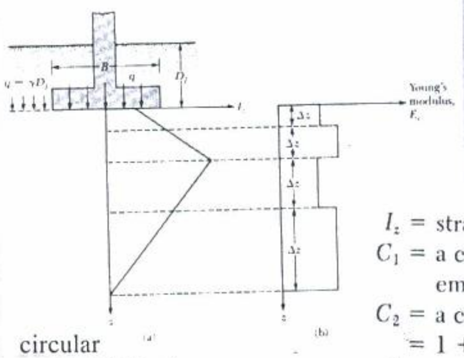


Figure 3.14 (Continued)

For Sandy Soil (Schmertmann, 1978)



$$S_e = C_1 C_2(\overline{q} - q) \sum_{0}^{2B} \frac{I_z}{E_s} \Delta z$$

I_z = strain influence factor

C_1 = a correction factor for the depth of foundation embedment = $1 - 0.5[q/(\overline{q} - q)]$

C_2 = a correction factor to account for creep in soil = $1 + 0.2 \log$ (time in years/0.1)

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

circular

$I_z = 0.1$ at $z = 0$
$I_z = 0.5$ at $z = 0.5B$
$I_z = 0$ at $z = 2B$

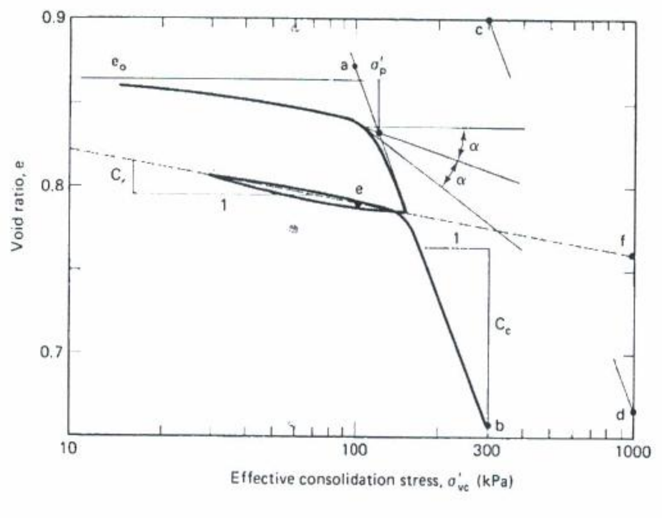
Similarly, for foundations with $L/B \geq 10$,

$I_z = 0.2$ at $z = 0$
$I_z = 0.5$ at $z = B$
$I_z = 0$ at $z = 4B$

PEMODELAN KONSOLIDASI PRIMER

Akibat pertambahan beban → kenaikan tekanan air pori

Keluarnya air dari pori → tekanan air pori kembali lagi (tanah settle)



CONSOLIDATION SETTLEMENT

- Total Settlement (St) = Si + Scp + Scs
- Si = Immediately Settlement
- Scp = Primary Consolidation Settlement
- Scs = Secondary Consolidation Settlement

Untuk lempung yang terkonsolidasi normal (Normally Consolidation) dimana

Pc < Po, maka

Scp = (Cc.H/1+eo) log (Po+Δ P/Po)

Bila Pc> Po (Lempung yang Over Konsolidasi) maka terdapat 2 (dua) kemungkinan

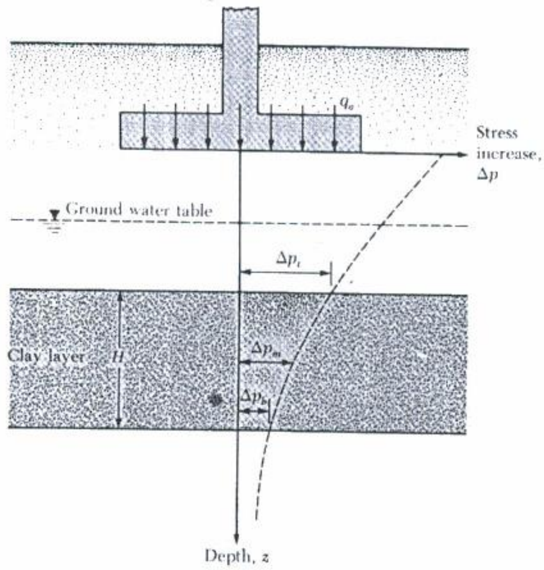
Bila Po+ ΔP < Pc dan Po+ ΔP > Pc

Po+ ΔP < Pc

Scp =(Cs.H/1+eo) log (Po+Δ P/Po)

Po+ ΔP > Pc

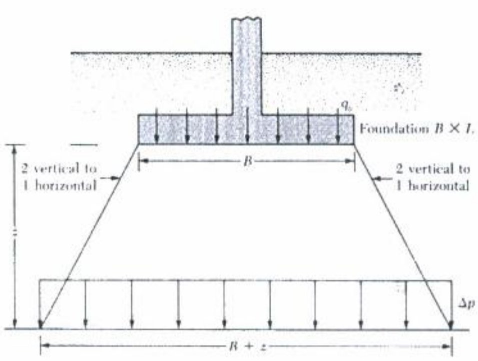
Scp =(Cs.H/1+eo) log (Pc/Po) + (Cc.H/1+eo) log (Po+Δ P/Pc)



$$\Delta p_{av} = \frac{1}{6}(\Delta p_t + 4\,\Delta p_m + \Delta p_b)$$

Δp?

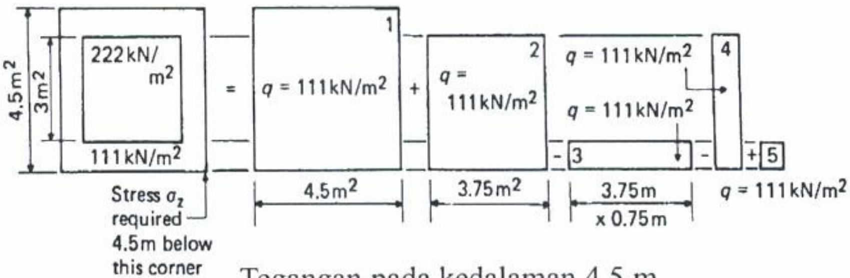
1. Diasumsikan beban pondasi menyebar secara linier



$$\Delta p_{av} = \frac{1}{6}(\Delta p_t + 4\,\Delta p_m + \Delta p_b)$$

Using the 2:1 method

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



Tegangan pada kedalaman 4.5 m

Shape	*m*	*n*	I_r	$\sigma_z = qI_r$ (kN/m²)
1	1	1	0.175	19.42
2	0.833	0.833	0.152	16.88
3	0.833	0.167	0.042	4.66
4	0.167	0.833	0.042	4.66
5	0.167	0.167	0.013	1.44

$\therefore\ \sigma_z = \mathbf{28.42\ kN/m^2}$

For irregularly shaped figures, the Newmark chart is more convenient to use than the Fadum chart (Fig. 3.4). It is constructed in such a way that each sub-division, bounded by two adjacent radial lines and two adjacent circles, represents an *influence value* of 0.005. The scale line *AB* is equal to the depth below ground level *z* and, at that depth, a pressure of *q* kN/m² on the surface will produce a vertical stress $\sigma_z = 0.005q$ kN/m² at point *N*.

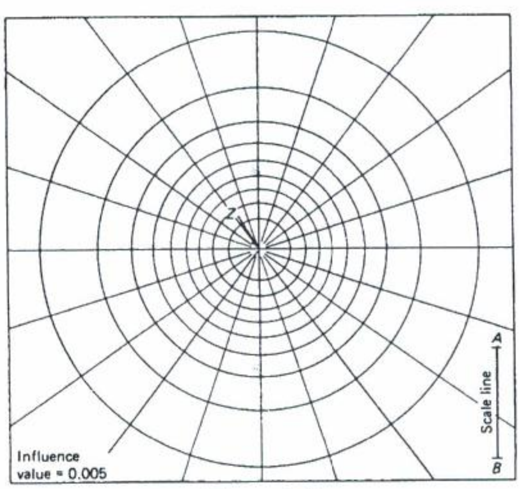
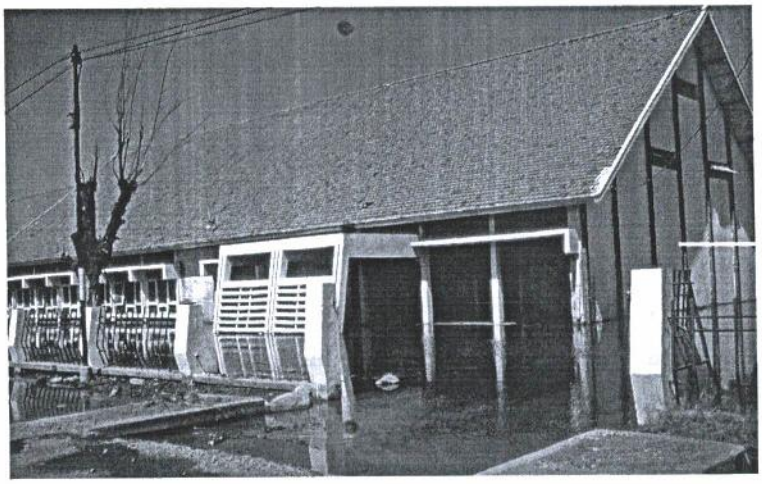


Figure 3.10

Settelement of Building Supported by Shallow Foundation

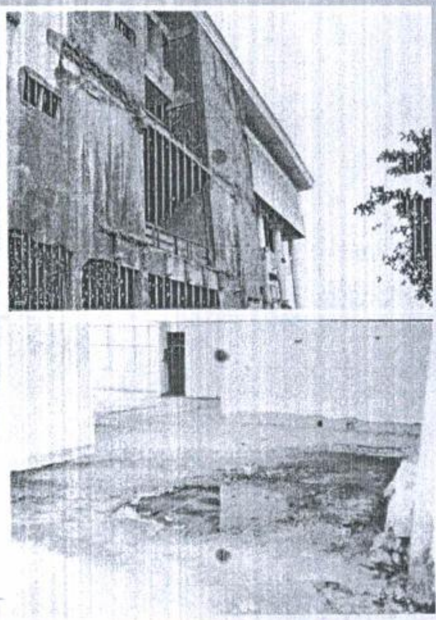


Office of Tanjung Mas Port

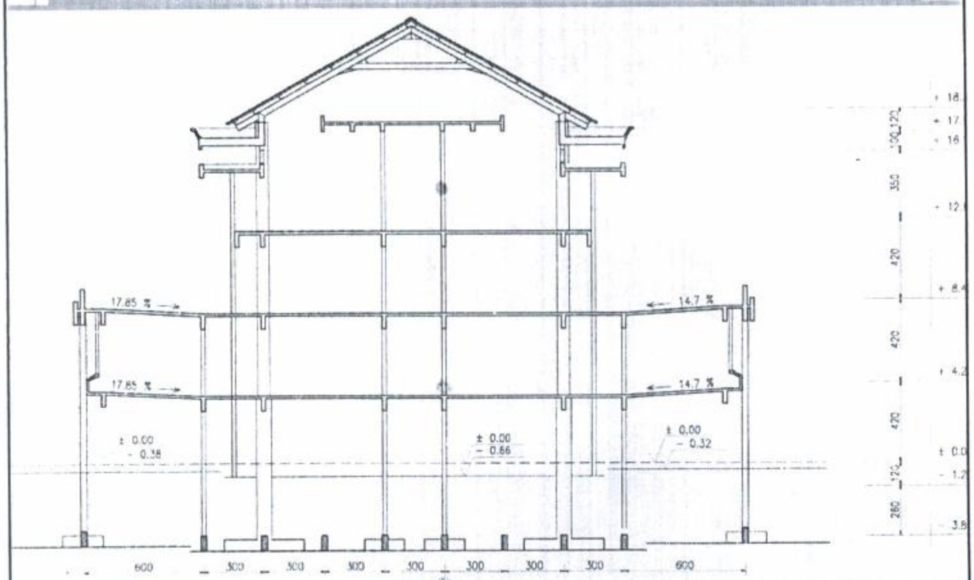
Tawang Railway Station

School Building

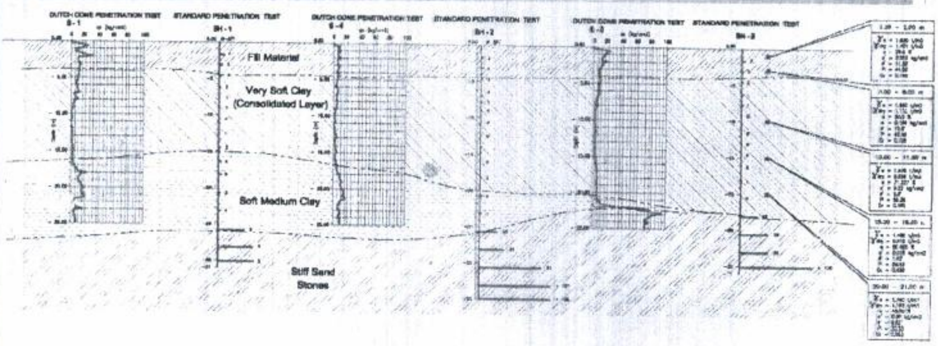
Kondisi Kerusakan Gedung

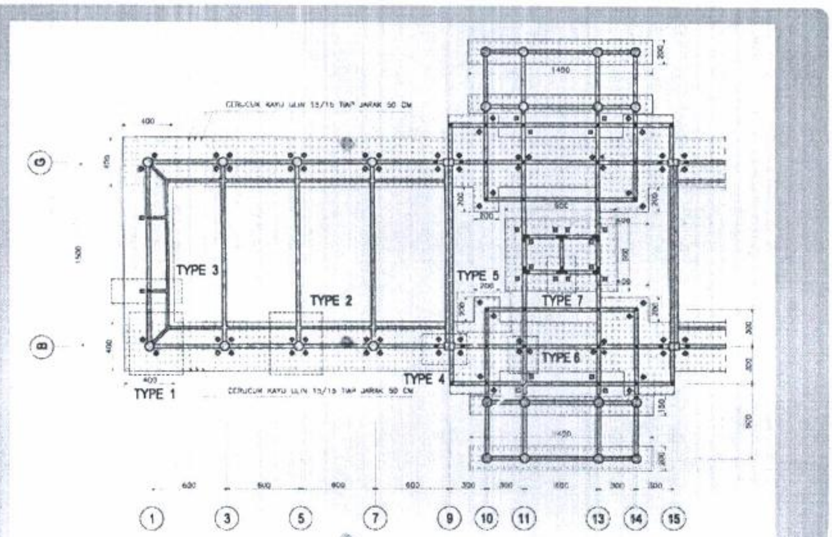


POTONGAN MELINTANG BANGUNAN



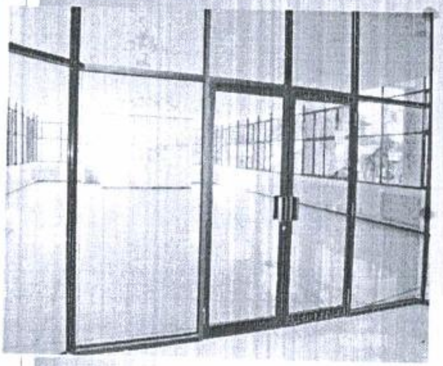
IL PROFIL BELOW BUILDING

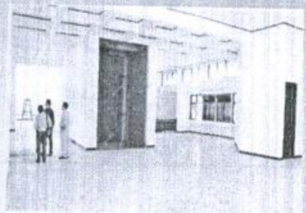




DENAH PILE TAMBAHAN DAN PERBAIKAN PONDASI

Gedung setelah Perbaikan





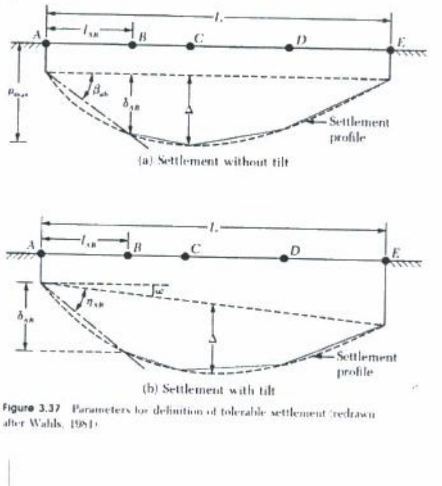


Figure 3.37 Parameters for definition of tolerable settlement (redrawn after Wahls, 1981)

Polshin and Tokar (1957) have presented the settlement criteria of the 1955 U.S.S.R. Building Code. These were based on the experience gained from the observation of foundation settlement over 25 years. Tables 3.8 and 3.9 present their findings.

Table 3.7 Limiting Angular Distortion As Recommended by Bjerrum[a]

Category of potential damage	η
Danger to machinery sensitive to settlement	1/750
Danger to frames with diagonals	1/600
Safe limit for no cracking of buildings[b]	1/500
First cracking of panel walls	1/300
Difficulties with overhead cranes	1/300
Tilting of high rigid buildings becomes visible	1/250
Considerable cracking of panel and brick walls	1/150
Danger of structural damage to general buildings	1/150
Safe limit for flexible brick walls, $L/H > 4$[b]	1/150

[a]After Wahls, 1981
[b]Safe limits include a factor of safety.

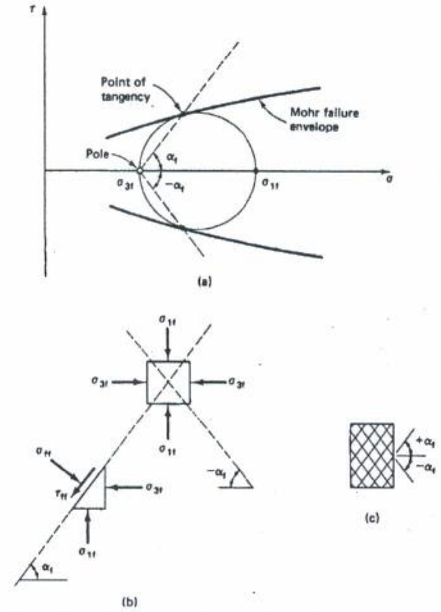


Fig. 10.7 (a) Mohr failure hypothesis for determining the angle of the failure plane in the (b) element; (c) conjugate failure planes.

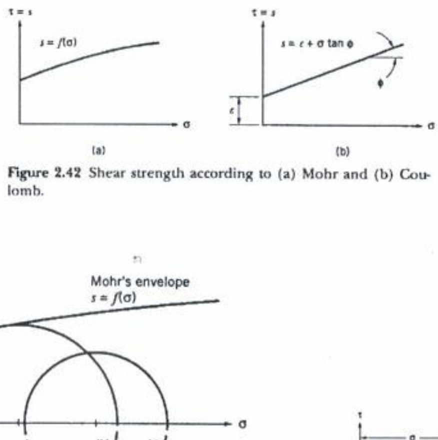


Figure 2.42 Shear strength according to (a) Mohr and (b) Coulomb.

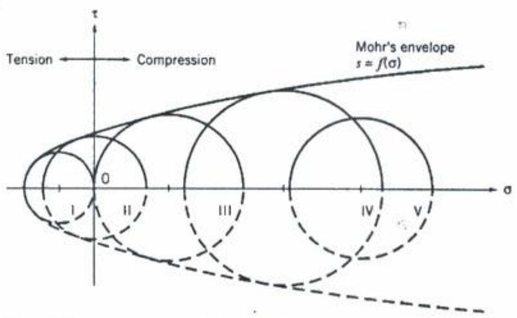


Figure 2.43 Mohr's circles for various cases of stress: I, simple tension; II, pure shear; III, simple compression; IV and V, biaxial compression.

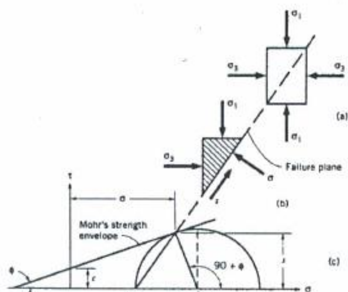


Figure 2.44 (a) Element; $\sigma_1 > \sigma_3$. (b) Normal and shear stresses on plane of failure. (c) Mohr circle for stress condition shown in (a).