B -45

MATERY OF SOIL MECHANIS -2

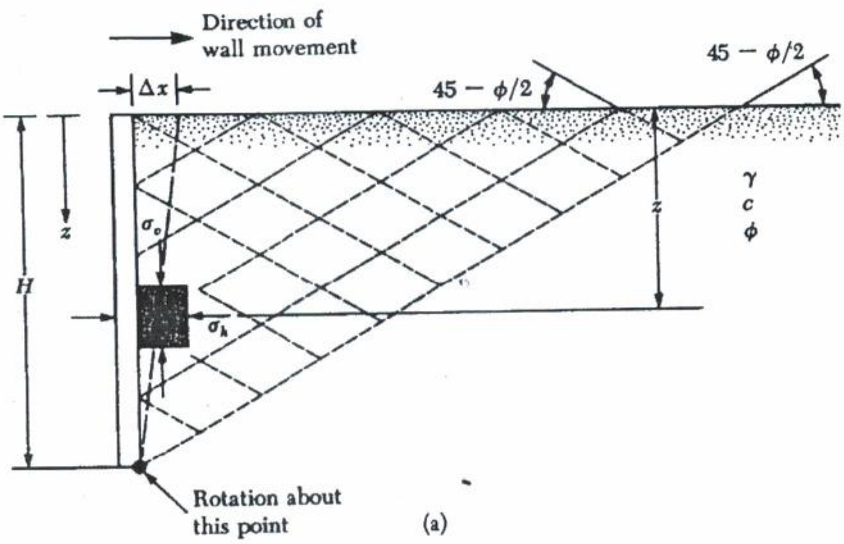
(Diktat Mekanika Tanah-2)

MODUL - 2:

LATERAL EARTH PRESSURE

SLOPE STABILITY

SITE SOIL INVESTIGATION



Idrus Ir. M.Sc

Staff Pengajar Jurusan Teknik Sipil FTSP – ISTN

1 st Edition September 2012

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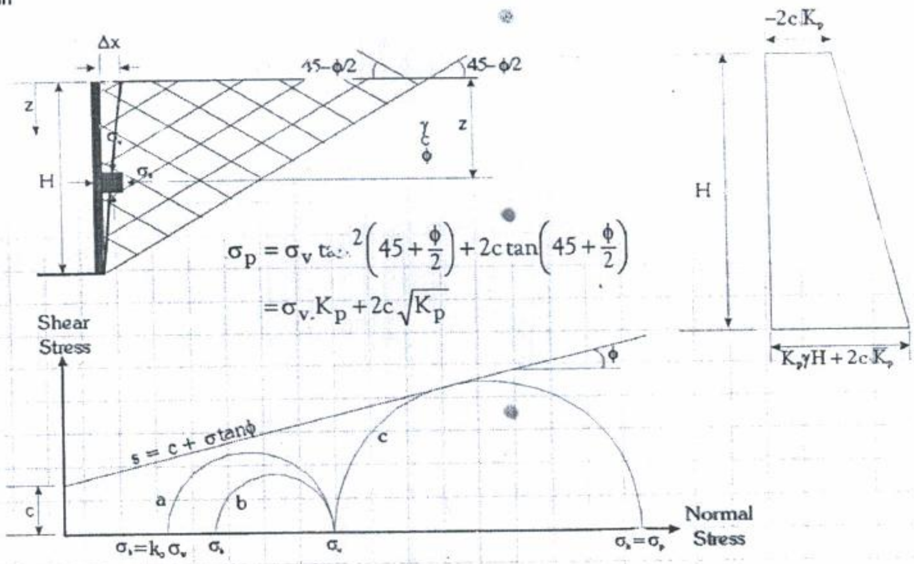
TEKANAN LATERAL PASIF RANKINE



$$\sigma_p = \sigma_v \tan^2\left(45+\frac{\phi}{2}\right) + 2c\tan\left(45+\frac{\phi}{2}\right)$$

$$= \sigma_v . K_p + 2c\sqrt{K_p}$$

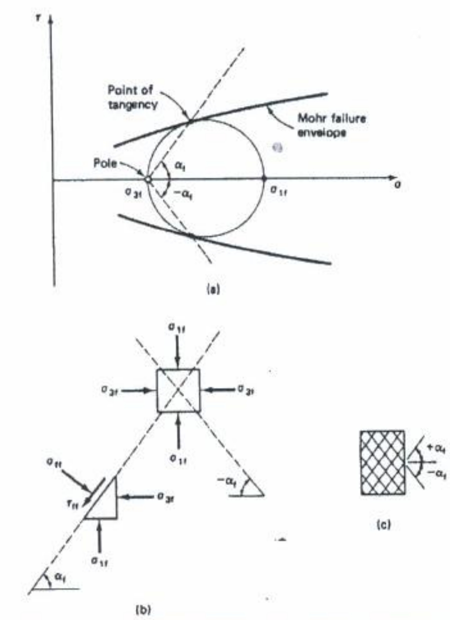


Fig. 10.7 (a) Mohr failure hypothesis for determining the angle of the failure plane in the (b) element; (c) conjugate failure planes.

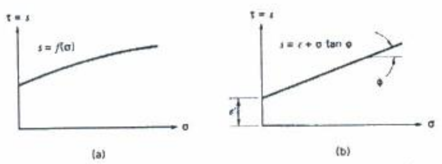


Figure 2.42 Shear strength according to (a) Mohr and (b) Coulomb.

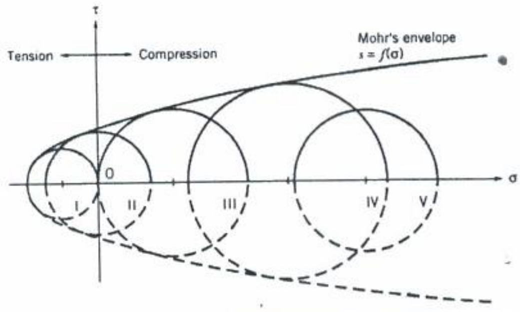


Figure 2.43 Mohr's circles for various cases of stress: I, simple tension; II, pure shear; III, simple compression; IV and V, biaxial compression.

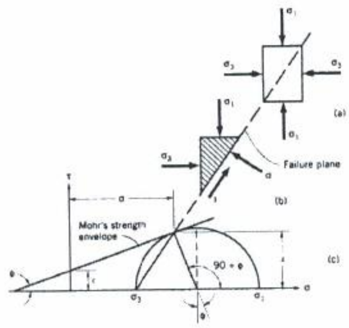


Figure 2.44 (a) Element, $\sigma_1 > \sigma_3$. (b) Normal and shear stresses on plane of failure. (c) Mohr circle for stress condition shown in (a).

- The refering to Eq. (1.76), the equation relating the principal stresses for a Mohr"s circle that touches the Mohr-Coulomb failure envelope can be given by

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

- For the Mohr' circle c in Figure 5.5b,
  major principal stress, σ1 = σv
  and minor principal stress, σ3 = σa
  thus

$$\sigma_v = \sigma_a \tan^2\left(45+\frac{\phi}{2}\right) + 2C\tan\left(45+\frac{\phi}{2}\right)$$

$$\sigma_a = \frac{\sigma_v}{\tan^2\left(45+\frac{\phi}{2}\right)} - \frac{2c}{\tan\left(45+\frac{\phi}{2}\right)}$$

$$\sigma_a = \sigma_v \tan^2\left(45-\frac{\phi}{2}\right) - 2c\tan\left(45-\frac{\phi}{2}\right)$$

$$= \sigma_v K_a - 2c\sqrt{K_a}$$

$$K_a = \tan^2(45-\phi/2)$$

Where = Rankine active earth pressure coefficient

For estimating the ultimate bearing capacity of *square* or *circular foundations*, Eq. (3.1) may be modified to

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

and

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

In Eq. (3.7), B equals the dimension of each side of the foundation; in Eq. (3.8), B equals the diameter of the foundation.

For foundations that exhibit the local shear failure mode in soils, Terzaghi suggested modifications to Eqs. (3.3), (3.7), and (3.8) as follows:

$$q_u = \frac{2}{3}cN'_c + qN'_q + \tfrac{1}{2}\gamma BN'_\gamma \quad \text{(strip foundation)} \tag{3.9}$$

$$q_u = 0.867cN'_c + qN'_q + 0.4\gamma BN'_\gamma \quad \text{(square foundation)} \tag{3.10}$$

$$q_u = 0.867cN'_c + qN'_q + 0.3\gamma BN'_\gamma \quad \text{(circular foundation)} \tag{3.11}$$

N'_c, N'_q, and N'_γ are the *modified bearing capacity factors*. They can be calculated by using the bearing capacity factor equations (for N_c, N_q, and N_γ) by replacing ϕ by $\phi' = \tan^{-1}(\frac{2}{3}\tan\phi)$. The variation of N'_c, N'_q, and N'_γ with the soil friction angle, ϕ, is given in Table 3.2.

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

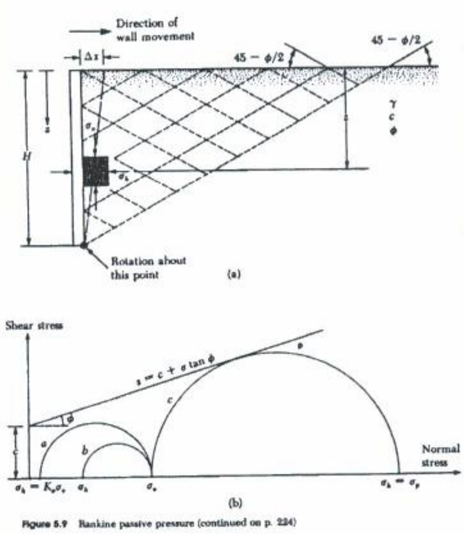
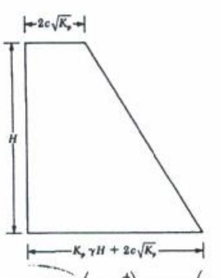


Figure 5.9 Rankine passive pressure (continued on p. 224)



$$\sigma_p = \sigma_v \tan^2\left(45 + \frac{\phi}{2}\right) + 2c\tan\left(45 + \frac{\phi}{2}\right)$$

Now, let

$$K_p = \text{Rankine passive earth pressure coefficient} = \tan^2\left(45 + \frac{\phi}{2}\right) \tag{5.20}$$

Hence, from Eq. (5.19)

$$\sigma_p = \sigma_v K_p + 2c\sqrt{K_p} \tag{5.21}$$

Using the preceding equation, Figure 5.9c shows the passive pressure diagram for the wall shown in Figure 5.9a. Note that,

at $z = 0$, $\sigma_v = 0$, $\sigma_p = 2c\sqrt{K_p}$

and

at $z = H$, $\sigma_v = \gamma H$, $\sigma_p = \gamma HK_p + 2c\sqrt{K_p}$

The passive force per unit length of the wall can be determined from the area of the pressure diagram, or

$$P_p = \tfrac{1}{2}\gamma H^2 K_p + 2cH\sqrt{K_p} \tag{5.22}$$

LATERAL EARTH PRESSURE

SANGAT DIKENAL 2 (DUA) TEORI TEGANGAN LATERAL, YAITU :

- Teori RANKINE
- Teori COULOMB

Koefisien tegangan latreral dari Rankine

Ka = (1-sin Ø) / (1+ sin Ø) atau Ka = tan2 (45 – Ø/2)
Kp = (1+sin Ø) / (1 - sin Ø) atau Ka = tan2 (45 + Ø/2)

Koefisien Tekanan Lateral Aktif Dan Pasif dari Coulomb

Koefisien tegangan lateral dari Coulomb

$$K_a = \frac{\cos^2(\varnothing - \theta)}{\cos^2\theta . \cos(\delta + \theta) \left[1 + \sqrt{\dfrac{\sin(\varnothing + \theta)\ \sin(\varnothing - \alpha)}{\cos(\delta + \theta) . \cos(\theta - \alpha)}} \right]^2}$$

$$K_p = \frac{\cos^2(\varnothing + \theta)}{\cos^2\theta . \cos(\delta - \theta) \left[1 + \sqrt{\dfrac{\sin(\varnothing - \theta) . \sin(\varnothing + \alpha)}{\cos(\delta - \theta) . \cos(\alpha - \theta)}} \right]^2}$$

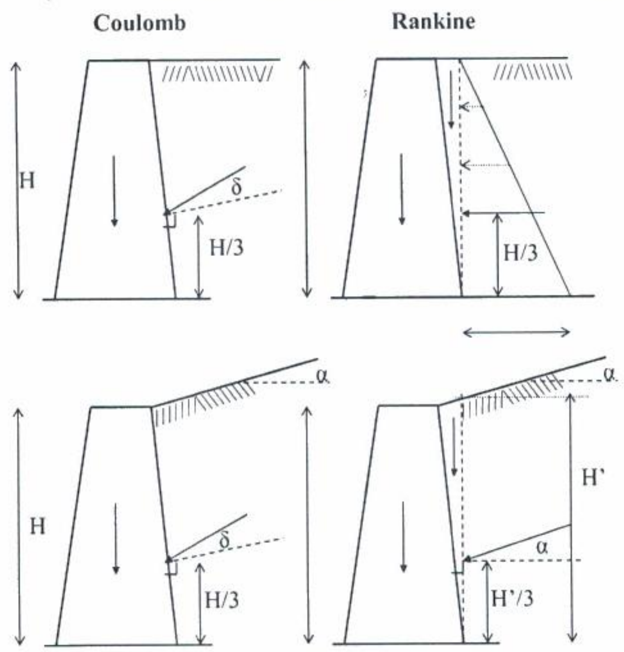


Diagram Tekanan Lateral Aktif dan Pasif Pada Tanah Pasir

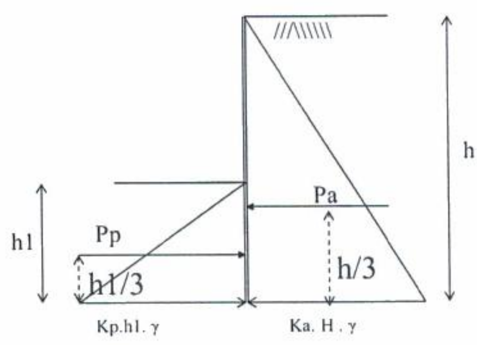
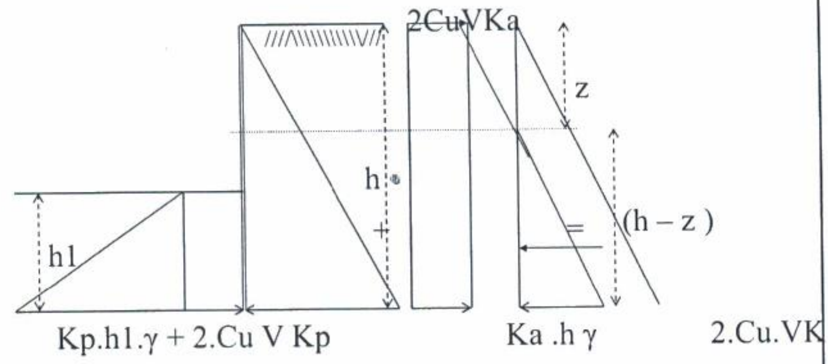
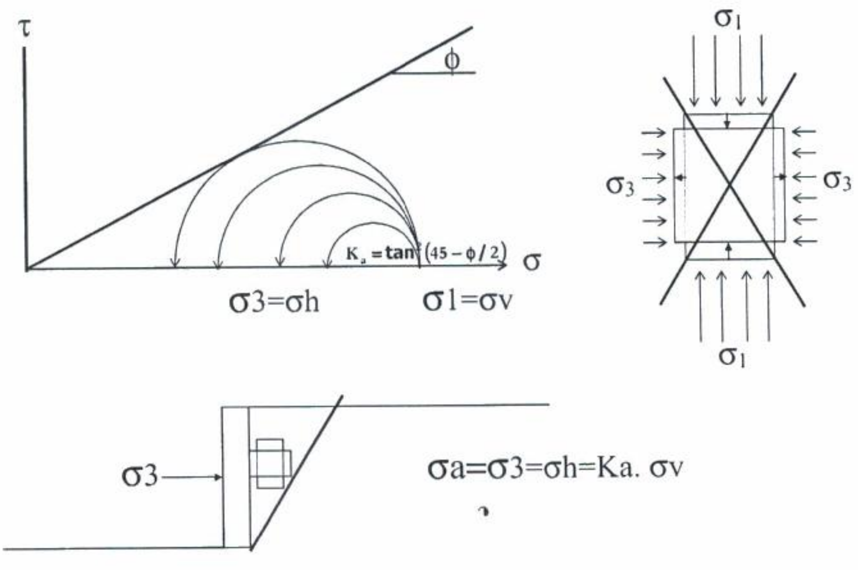


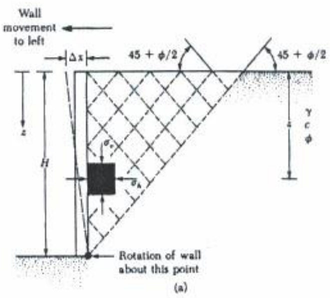
Diagram Tekanan Lateral Aktif dan Pasif Pada Tanah Lempung



Tegangan Aktif



Rankine active earth pressure coefficient



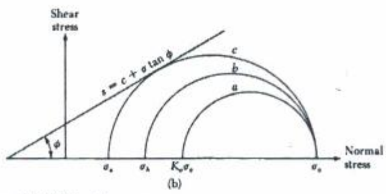


Figure 5.5 Rankine active pressure

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

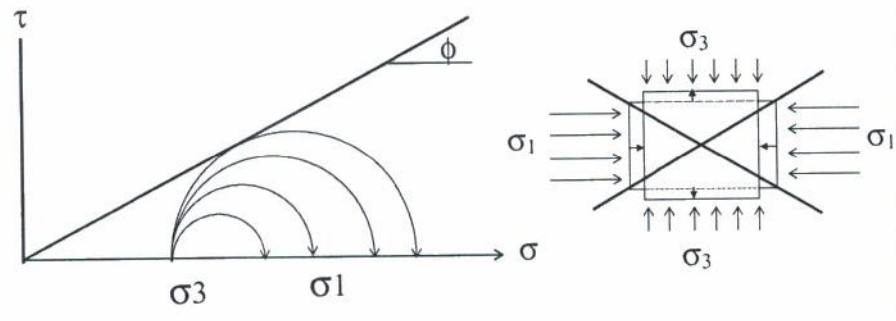
$$\sigma_v = \sigma_a \tan^2\left(45+\frac{\phi}{2}\right) + 2C\tan\left(45+\frac{\phi}{2}\right)$$

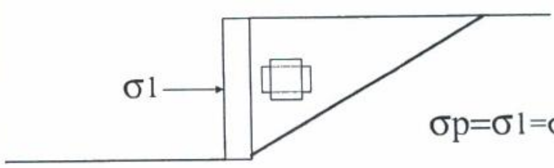
$$\sigma_a = \frac{\sigma_v}{\tan^2\left(45+\frac{\phi}{2}\right)} - \frac{2c}{\tan\left(45+\frac{\phi}{2}\right)}$$

$$\sigma_a = \sigma_v \tan^2\left(45-\frac{\phi}{2}\right) - 2c\tan\left(45-\frac{\phi}{2}\right)$$

$$= \sigma_v K_a - 2c\sqrt{K_a}$$

Tegangan Pasif





$\sigma p = \sigma 1 = \sigma h = Kp.\ \sigma v$

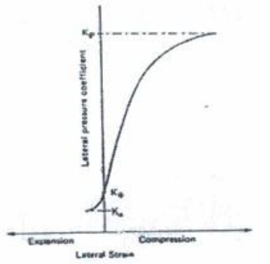
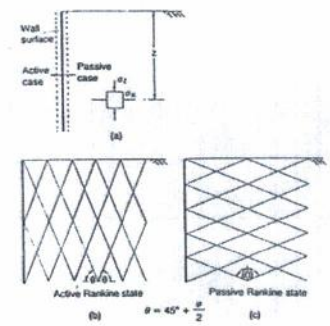


Fig. 6.10 Relationship between lateral strain and lateral pressure coefficient



$$\sigma_p = \sigma_v \tan^2\left(45 + \frac{\phi}{2}\right) + 2c \tan\left(45 + \frac{\phi}{2}\right) \quad (5.22)$$

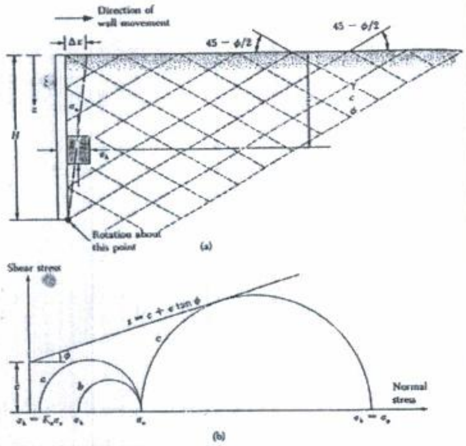


Figure 5.11 Rankine passive pressure

SLOPE STABILITY ANALYSIS
(CONCEPTS AND INPUT SOIL PARAMETERS)

Prepared by:
Dr. I Wayan Sengara

SERTIFIKASI
Himpunan Ahli Teknik Tanah Indonesia
Jakarta, 1 Juni 2005

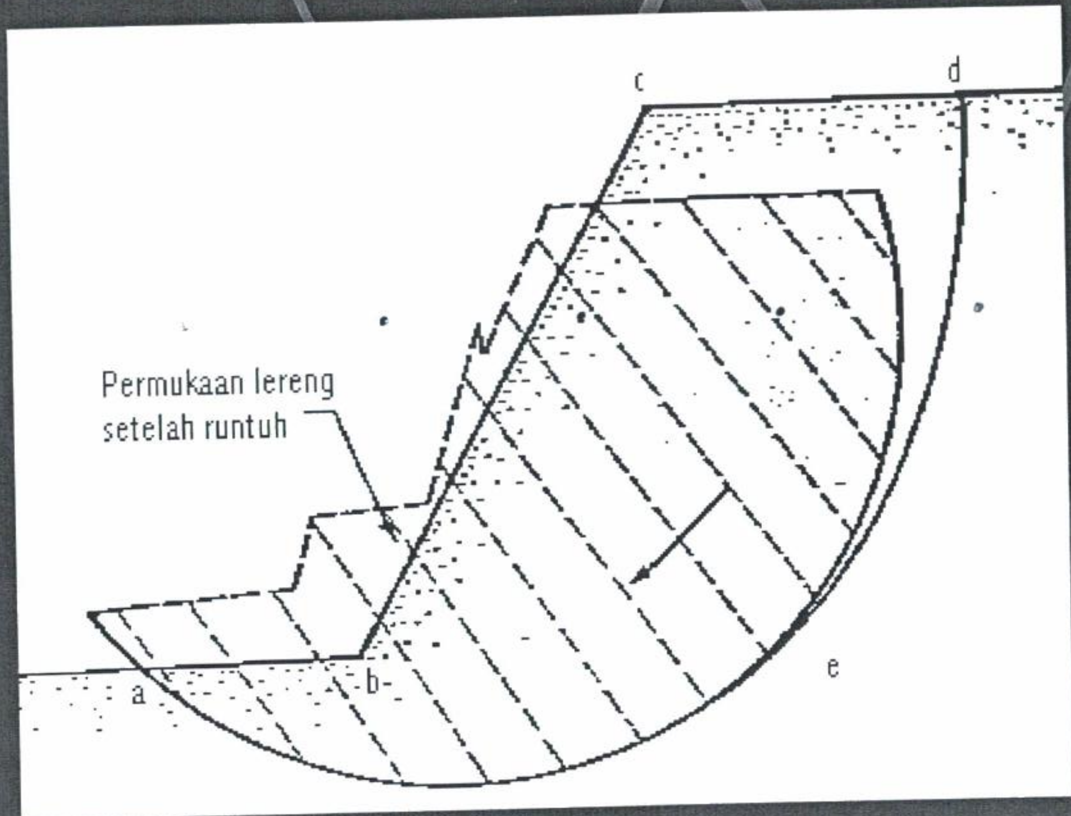
TOPIC MATERIALS

- SOIL MECHANICS (Engineering Soil Properties)
- SOIL INVESTIGATION
- **<u>SLOPE STABILITY ANALYSIS</u>**
  <u>(CONCEPTS AND INPUT SOIL PARAMETERS)</u>
- SLOPE STABILITY ANALYSIS
  (CALCULATION OF SAFETY FACTOR)

Maksud Analisis Stabilitas Lereng

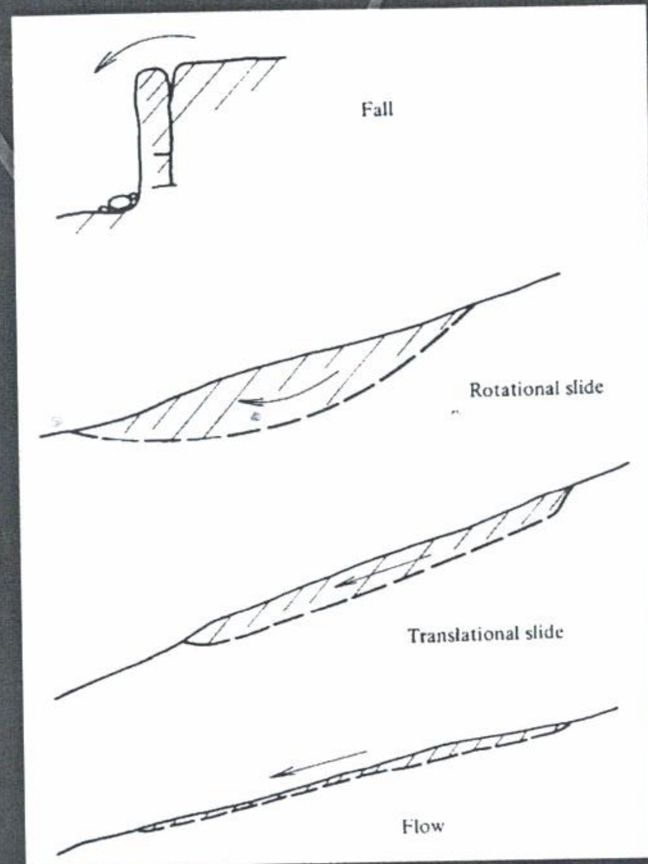
- Melakukan kajian kemungkinan kelongsoran lereng yang melibatkan lereng-lereng baik alami maupun buatan.
- Melakukan analisis kelongsoran dan mengerti mekanisme keruntuhan dan pengaruh faktor-faktor lingkungan untuk design lereng.
- Melakukan analisis terhadap stabilitas lereng untuk kondisi jangka pendek (during construction) dan jangka panjang
- Untuk memungkinkan melakukan redesign terhadap lereng yang telah longsor dan merencanakan dan mendesign langkah-langkah preventif jika diperlukan.
- Untuk mempelajari efek dari beban seismic pada lereng atau tanggul.

Terjadinya Kelongsoran

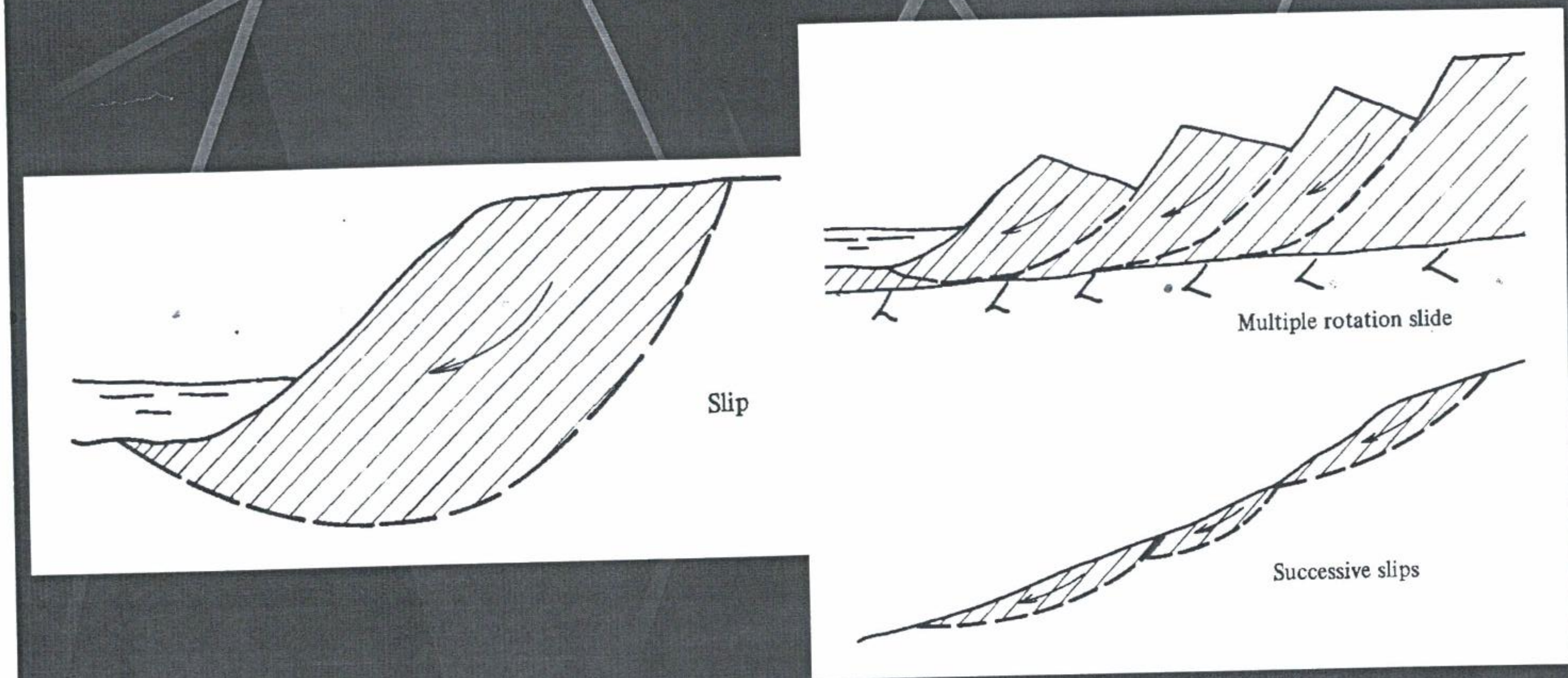


Jenis *Slope* Failure:

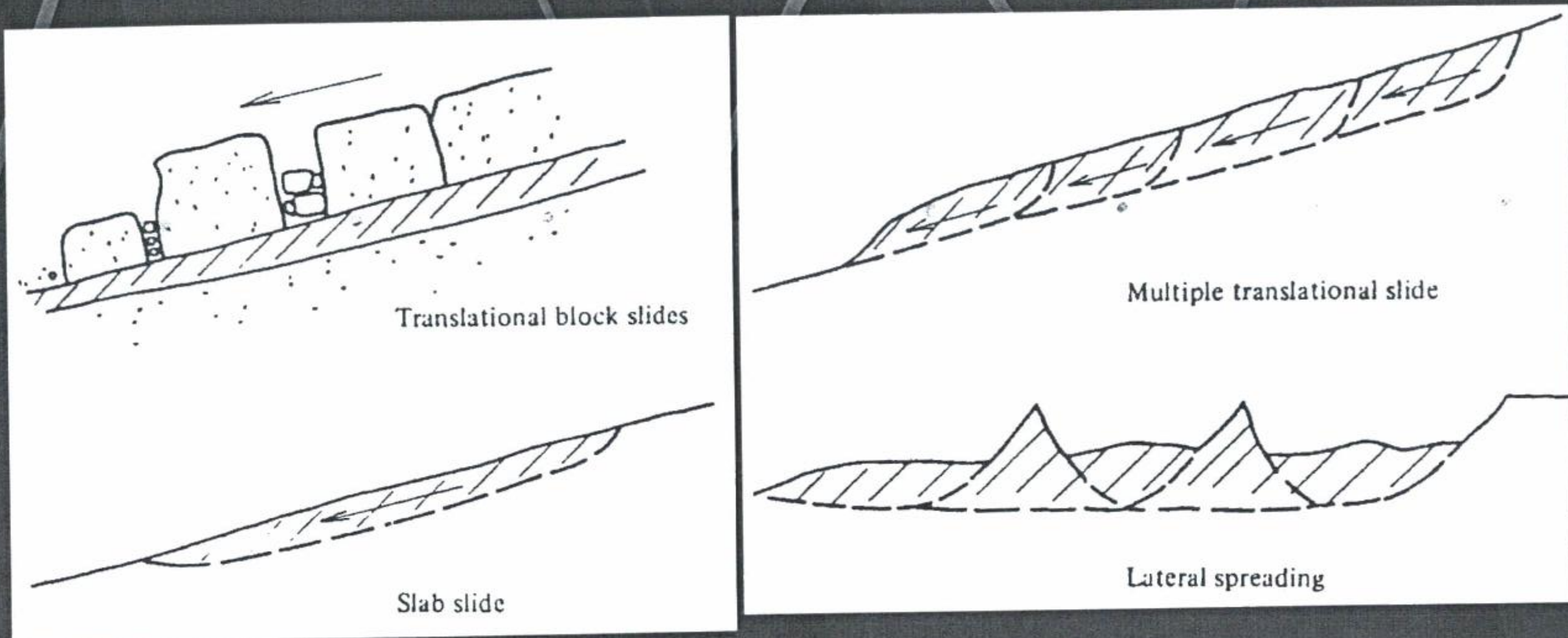
- Falls
  Tidak terdapat bidang gelincir
- Rotational slide
  Umumnya circular
- Translation slides
  Terjadi sepanjang *bedding planes*, *fissures* yang sejajar dengan permukaan
- Flows
  Material terurai kemudian bergerak "mengalir"



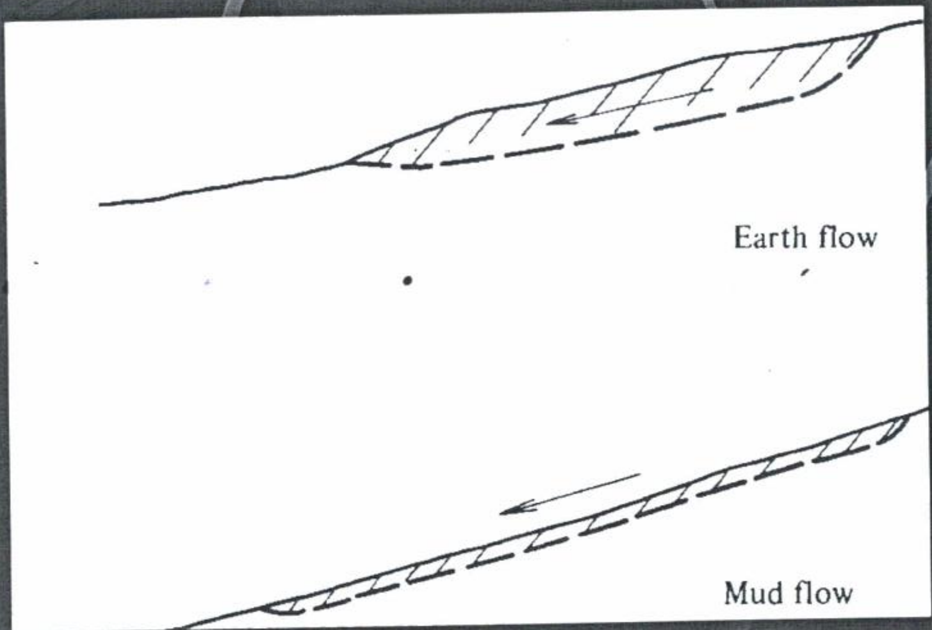
Rotational Slides:



Translational Slides:



Flows:



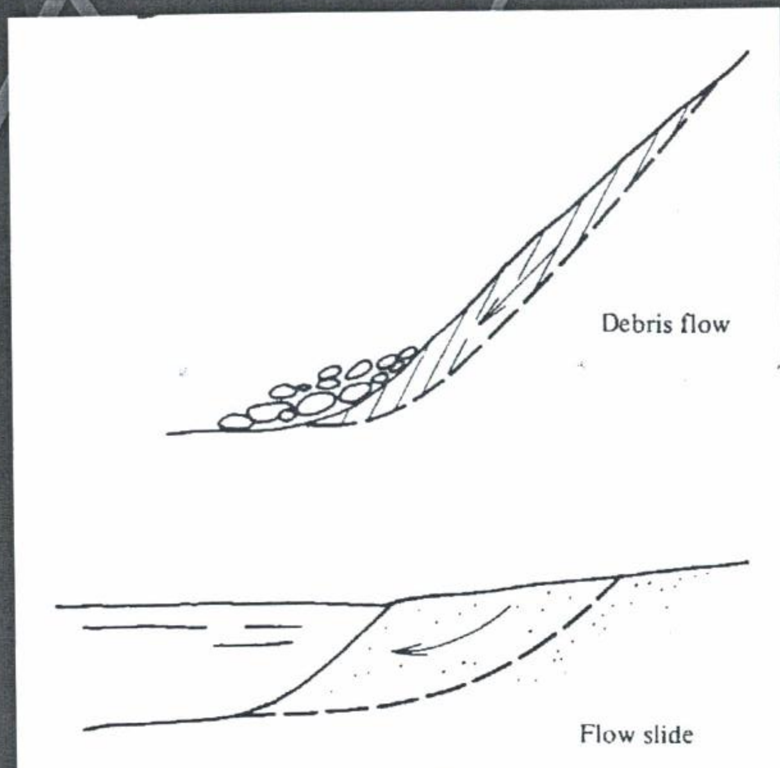


Fig. 11.14 Flows.

Macam-macam Lereng

- Lereng alam
- Lereng buatan:
  - timbunan
  - galian
- Landfills

Proses Kelongsoran Slope

Slope movement terjadi akibat gaya yang bekerja melebihi batas tahanan tanah.

- Sumber gaya(Driving force):
  - gravity (berat sendiri)
  - Climate
  - Beban luar (static, dynamic)
- Tahanan (Resistance): tahanan geser tanah: c, ϕ

Penyebab Sliding

- Faktor-faktor yang menyebabkan naiknya tegangan geser:
  1. Hilangnya lateral support (erosi, galian, tidak adanya dinding penahan tanah, dsb.)
  2. Beban/surcharge (berat bangunan, hujan, dsb.)
  3. Gempa, ledakan, getaran mesin
  4. Tekanan lateral (swelling clays)

- Faktor-faktor yang mengurangi tahanan geser tanah:
  1. Initial state: - komposisi, texture, geometri lereng
  2. Pelapukan tanah
  3. Perubahan intergranular forces: tekanan air tanah, fractures

Failure akibat seismic Load
(Gempa Alaska, 1964)

I. KRITERIA KERUNTUHAN MOHR-COULOMB

Kriteria keruntuhan Mohr-Coulomb dapat dituliskan konsisten dalam keadaan efektif sebagai:

$$\tau_{ff} = \sigma'_{ff} \tan\phi' + c'$$

di mana :

τ_{ff} = tegangan geser pada bidang runtuh pada saat keruntuhan

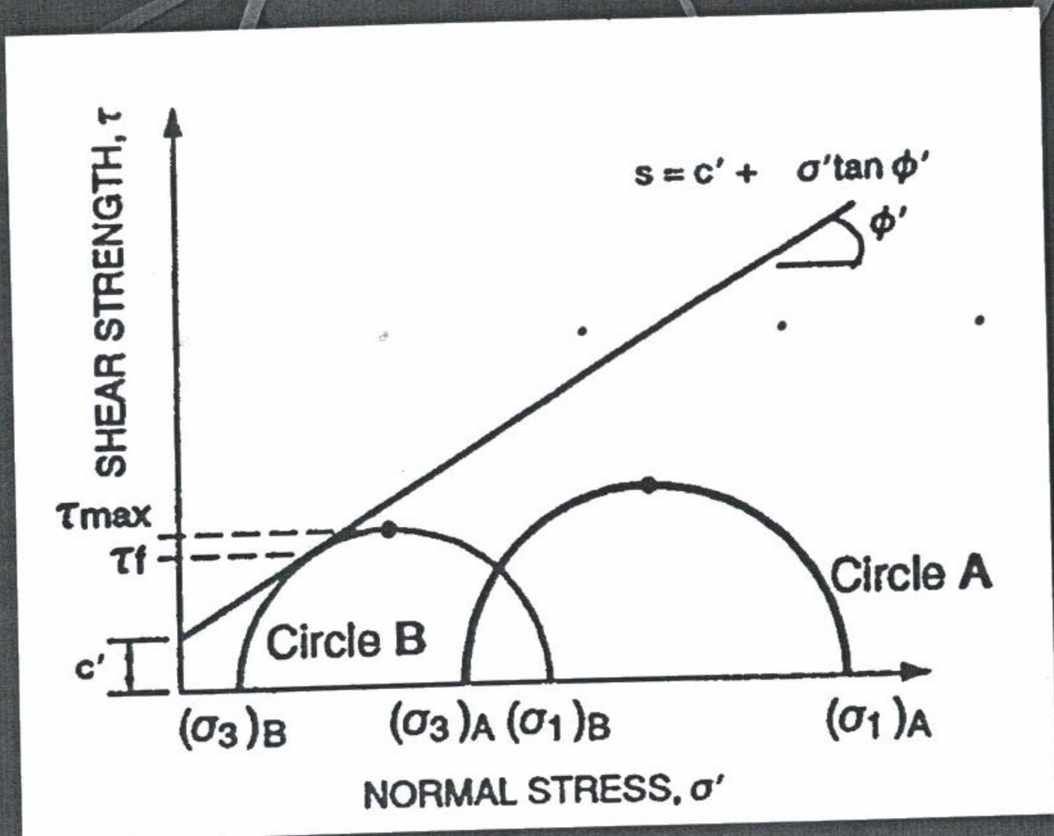
σ'_{ff} = tegangan normal efektif pada bidang runtuh pada saat keruntuhan

ϕ' = sudut geser dalam dalam keadaan efektif

c' = cohesi dari tanah dalam keadaan efektif

Shear strength parameters:

- *Stress independent component* (c)
- *Stress dependent component* (ϕ)



Kriteria keruntuhan Mohr-Coulomb ditunjukkan oleh garis lurus yang dikenal dengan nama Mohr-Coulomb failure envelope. Garis ini menunjukkan batas kondisi stabil dan keruntuhan.

Setiap tegangan yang berada di bawah garis adalah keadaan stabil. Sedangkan keruntuhan terjadi kalau tegangan menyentuh atau melewati garis keruntuhan Mohr-Coulomb.

Besaran-besaran c' dan f' merupakan parameter-parameter tahanan geser tanah efektif yang merepresentasikan sifat-sifat atau besarnya tahanan geser dari tanah tersebut.

Dengan kriteria keruntuhan Mohr-Coulomb kita dapat meramalkan tegangan-tegangan pada bidang runtuh pada saat keruntuhan terjadi.

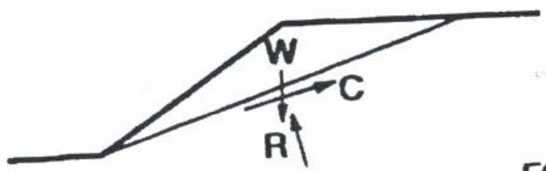
Faktor Keamanan = τ_{ff} (yang ada) / τ_f (yang bekerja)

Faktor Keamanan (1)

LIMIT EQUILIBRIUM

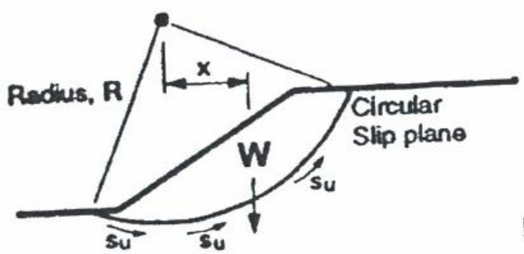
$$FOS = \frac{s_u}{\tau_{required}} \quad \text{(Total Stress)}$$

$$FOS = \frac{c' + \sigma' \tan\phi'}{\tau_{required}} \quad \text{(Effective Stress)}$$



FORCES

$$FOS = \frac{\text{Summation of resisting force}}{\text{Summation of mobilized force}}$$



MOMENTS

$$FOS = \frac{\text{Resisting moment}}{\text{Overturning moment}} = \frac{R \int s_u \, ds}{W x}$$

Tabel 6. Recommended Minimum Values of Static Safety Factor

(J.M. Duncan and A.L. Buchignani, UC Berkeley, 1975)

Cost and Consequences of Slope Failure	Uncertainties of	Strength Measurement
	Small [1]	Large [2]
Cost of repair comparable to cost of construction. No danger to human life or other property if slope fails.	1.25	1.5
Cost of repair much greater than cost of construction, or danger to human life or other valuable properties if slope fails.	1.5	2.0

[1] The uncertainty of the strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete and logical picture of the strength characteristics.

[2] The uncertainty of the strength measurements is greatest when the soil conditions are complex and when the available strength test data do not provide a consistent, complete and logical picture of the strength characteristics.

IWS

Tabel 7. Recommended Minimum Values of Seismic Safety Factor

Cost and Consequences of Slope Failure	Uncertainties of Strength Measurement	
	Small [1]	Large [2]
Cost of repair comparable to cost of construction. No danger to human life or other property if slope fails.	1.1	1.2
Cost of repair much greater than cost of construction, or danger to human life or other valuable properties if slope fails.	1.2	1.3

[1] The uncertainty of the strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete and logical picture of the strength characteristics.

[2] The uncertainty of the strength measurements is greatest when the soil conditions are complex and when the available strength test data do not provide a consistent, complete and logical picture of the strength characteristics.

Tabel 4. Angka keamanan minimum untuk Lereng Galian Terbuka (Tanpa Gempa) (Konsensus TPKB DKI-Jakarta, 1999; Djayaputra, 1999)

Kondisi Lingkungan	Keandalan Parameter Tanah			
	Kurang		Cukup	
	Temporer	Permanen	Temporer	Permanen
Tidak ada hunian manusia atau bangunan sekitar	1.3	1.5	1.25	1.3
Banyak bangunan sekitar	1.5	2.0	1.3	1.5

Tabel 5. Angka Keamanan Minimum untuk Galian dengan Sistem Dinding Penahan (Konsensus TPKB DKI-Jakarta, 1999; Djayaputra, 1999)

Kondisi	Angka keamanan kondisi temporer	Angka keamanan kondisi Permanen	Keterangan
Stabilitas (umum)	1.3	1.5	Parameter tanah yang ditentukan oleh ahli geoteknik
Bottom heave pada level fondasi	1.3	1.5	Idem
Bottom heave di atas level fondasi	1.5		Idem
Piping	1.5	2.0	Idem

IWS

Pendekatan Analisis:

- Pendekatan Tegangan Total
  - *Short term condition (Undrained)*
  - *(End of construction)*
- Pendekatan Tegangan Efektif
  - *Long-term (Drained)*

Parameter-parameter tanah Test triaxial

- *Consolidated Drained (CD)*
- *Consolidated Undrained (CU)*
- *Unconsolidated Undrained (UU)*

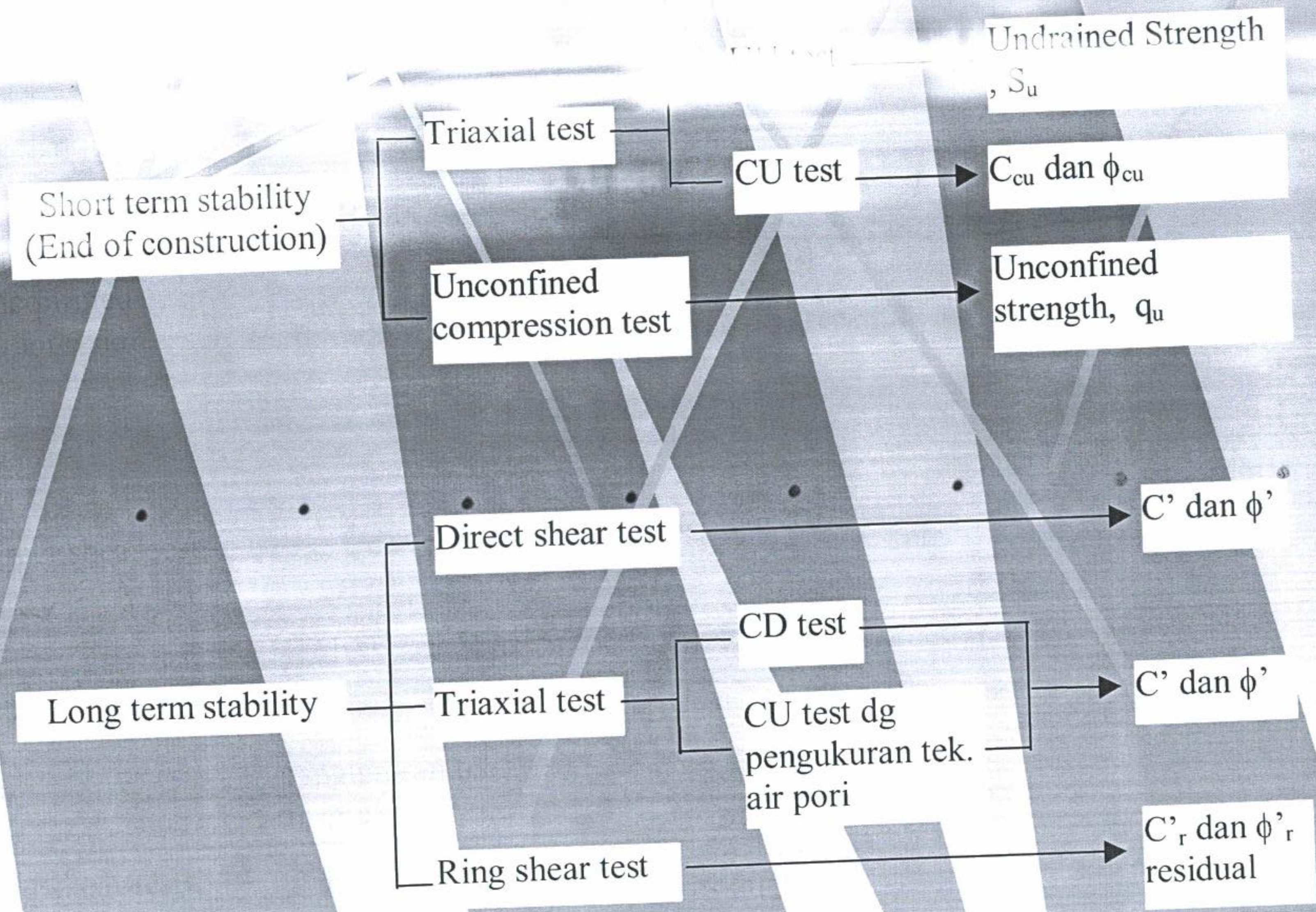
... Lempung Jenuh. (Lee, 1995)

	Soft (NC) Clay	Stiff (Highly OC) Clay
Timbunan		
Kondisi kritis	Kasus Unconsolidated Undrained (UU) tanpa drainase	Kemungkinan kasus UU tapi cek juga kasus Consolidated Drained (CD) Stabilitas biasanya bukan problem utama.
Catatan:	Gunakan $\phi = 0$, $c = \tau_{ff}$ dengan koreksi yang sesuai.	
Galian atau *Natural Slope*		
Kondisi kritikal	Bisa keduanya, kasus UU atau CD.	Kasus CD (drainase penuh). Gunakan analisis tegangan efektif *dg equilibrium pore presssure*; jika clay agak fissured, c' dan juga mungkin ϕ' dapat menurun sbg fungsi waktu.
Catatan:	Jika tanah sangat sensitif, dapat beralih dari kondisi drained ke undrained.	

Triaksial (Lee, 1996)

		Tes dan Kekuatan Geser
Kohesif	Jangka pendek (short term/end of construction)	Test UU atau CU untuk *undrained strength* dengan level tegangan insitu yang sesuai.
	Konstruksi bertahap (staged construction)	Test CU untuk *undrained strength* dengan level tegangan yang sesuai.
	Jangka panjang (long term)	Test CU dengan pengukuran *pore pressure*, atau tes CD untuk parameter kuat geser efektif.
Granular	Semua jenis	Parameter *strength* ϕ' didapatkan dari tes lapangan atau tes *direct shear*.
c-ϕ material	Jangka panjang (long term)	Tes CU dengan pengukuran *pore pressure* atau tes CD untuk parameter kuat geser efektif.

... Parameter Hasil Tes Laboratorium



IWS

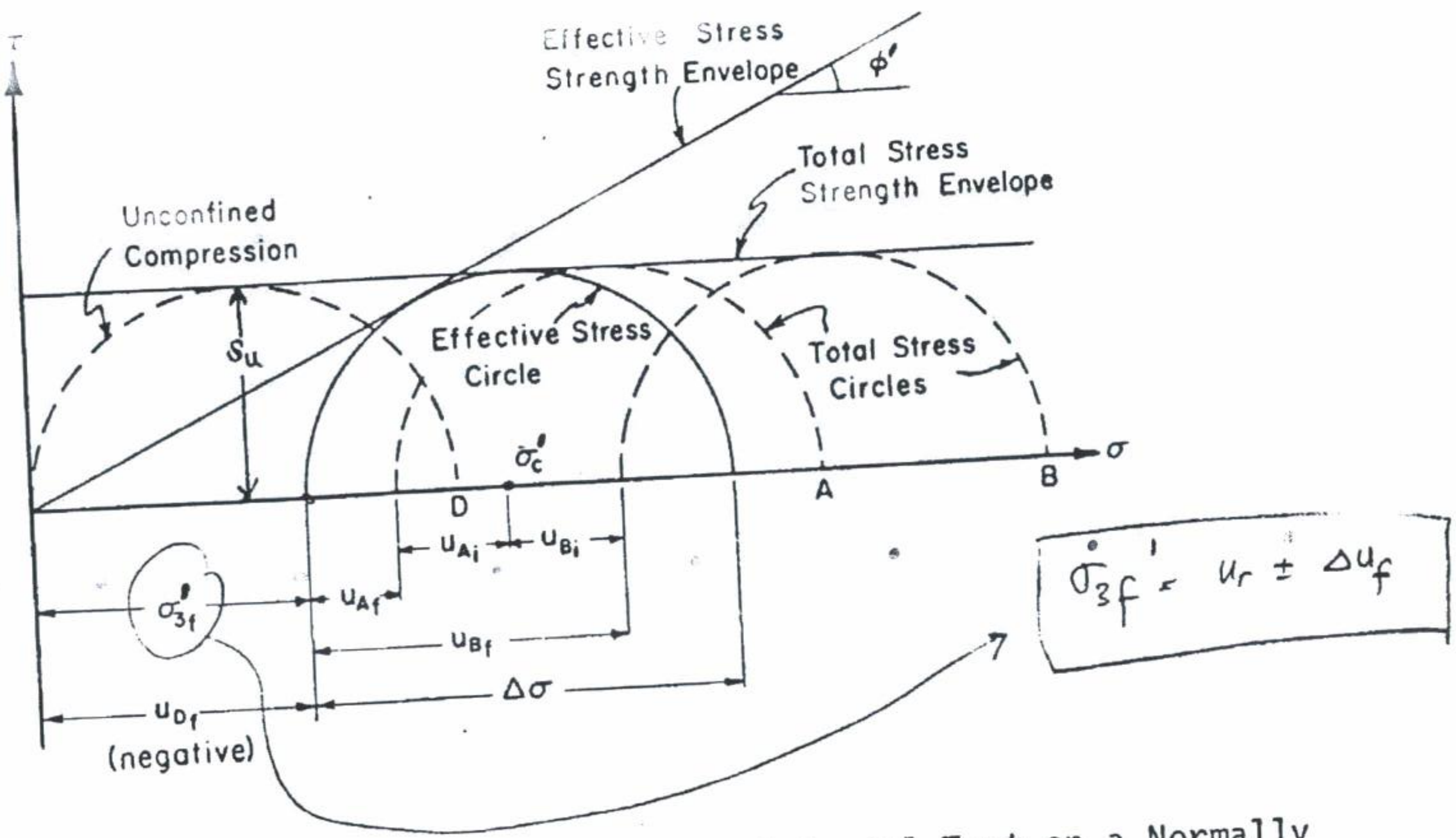
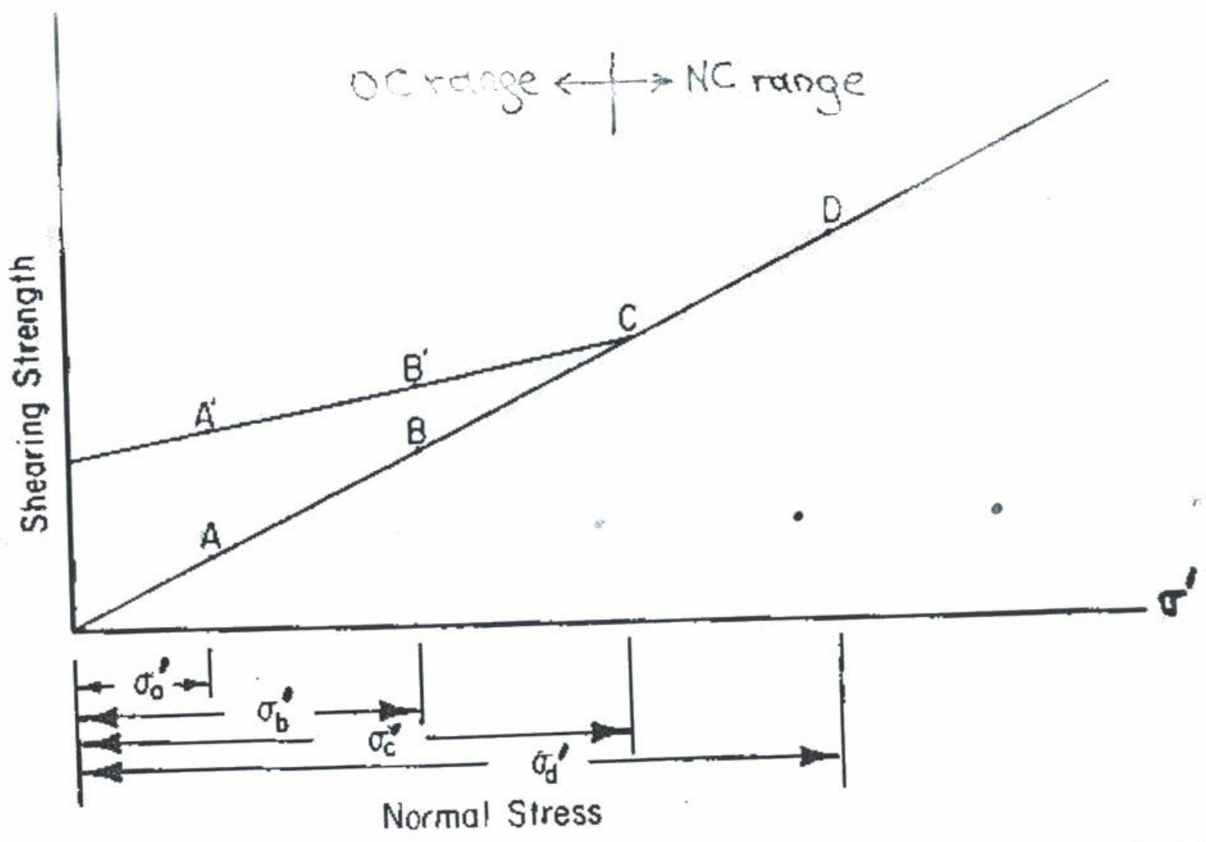
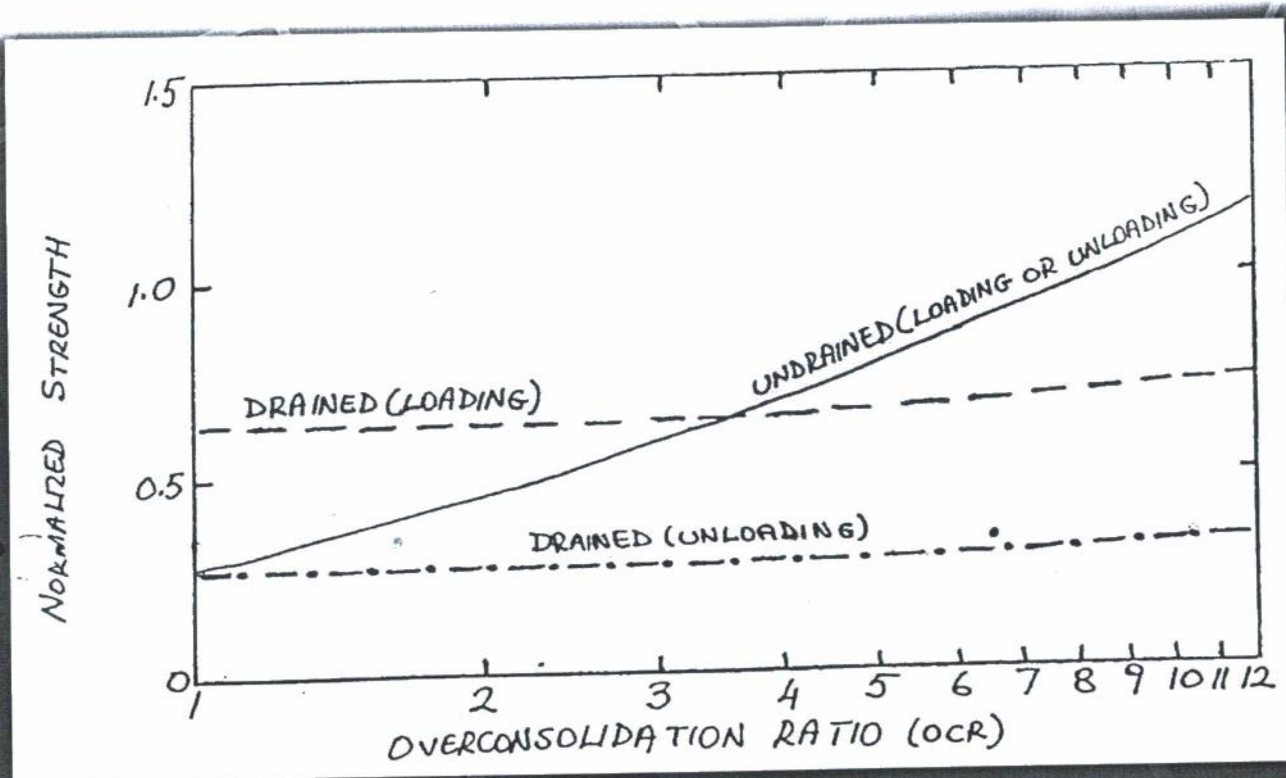


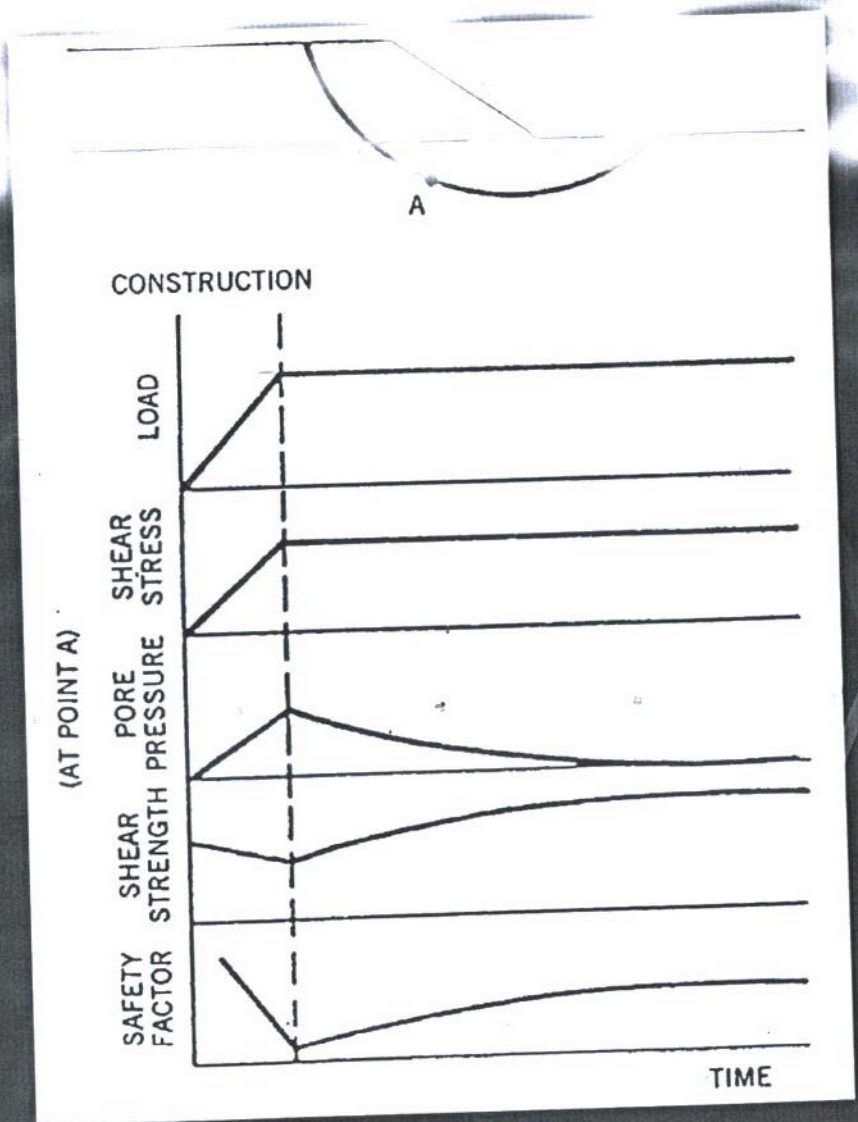
Figure 22. Typical Strength Envelope from a UU Triaxial Test on a Normally Consolidated Clay

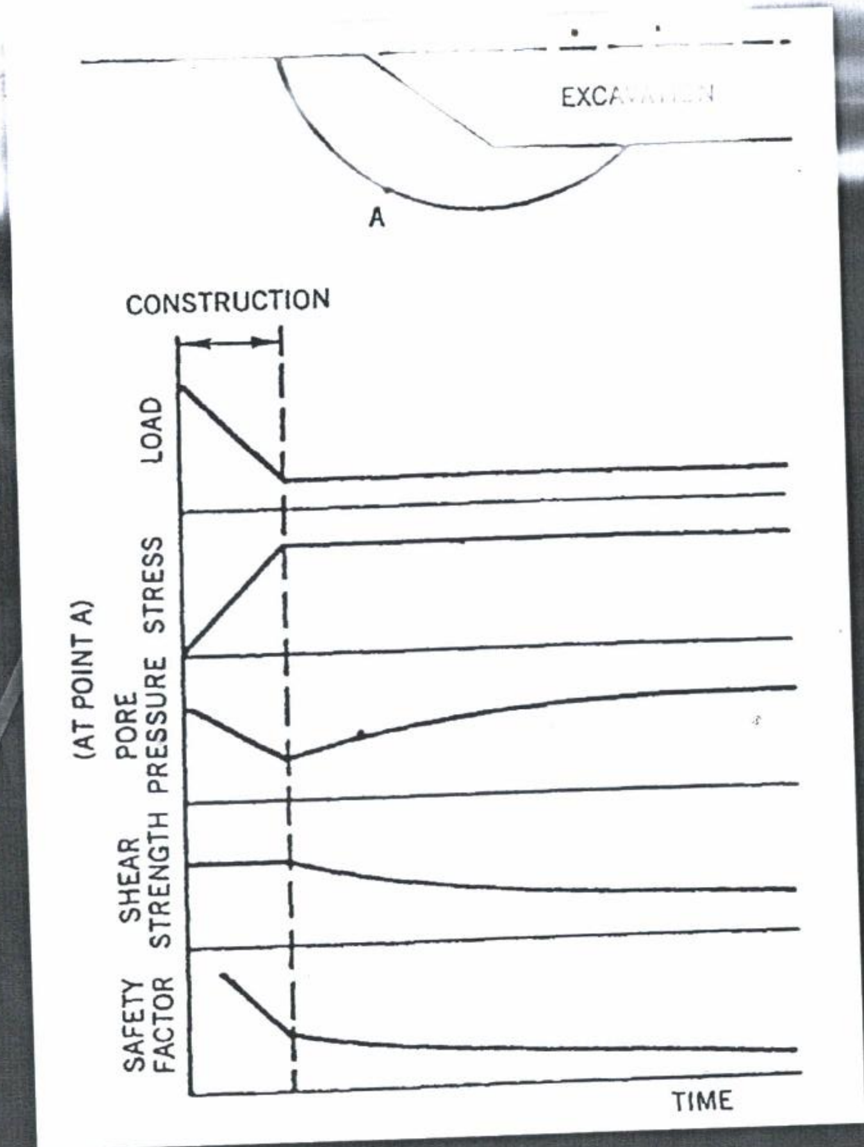


Strength Envelope for Over Consolidated Clay

RAINED

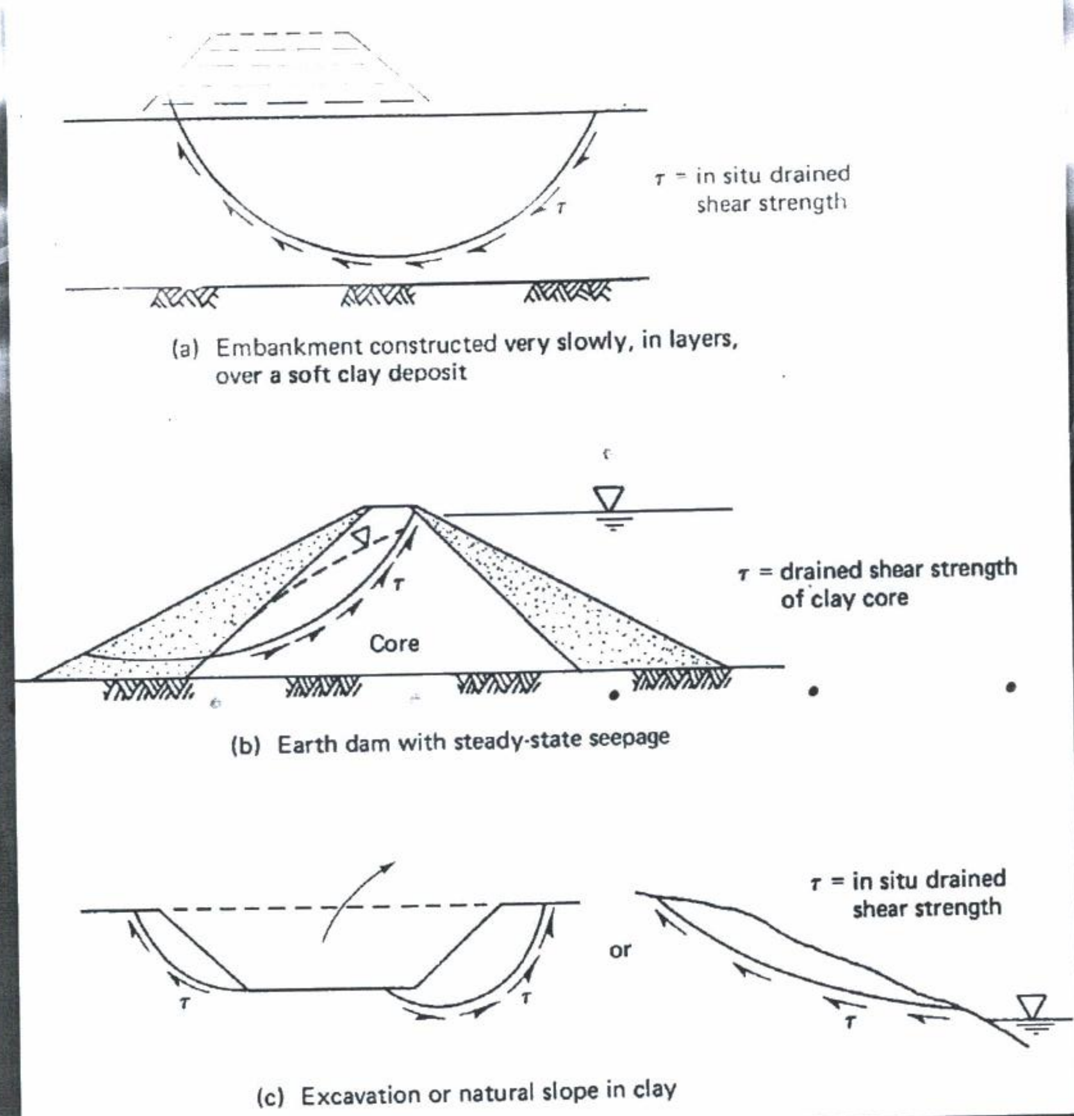

NORMALIZED STRENGTH
1.5
1.0
0.5
0
DRAINED (LOADING)
UNDRAINED (LOADING OR UNLOADING)
DRAINED (UNLOADING)
1
2
3
4
5
6
7
8
9
10
11
12
OVERCONSOLIDATION RATIO (OCR)





Gambar 4. Kondisi stabilitas untuk sebuah timbunan dan galian lereng pada tanah lempung (dari Edil T.B., 1982; Bishop and Bjerrum, 1960)

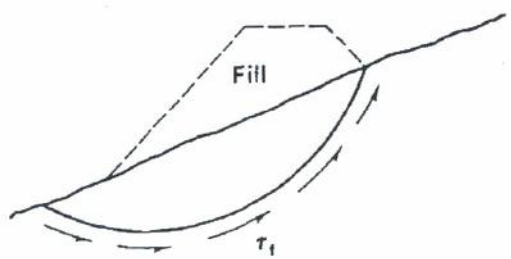
IWS



Gambar 1. Beberapa Contoh Kasus CD (setelah Ladd, 1971)

(a) Embankment raised (2) subsequent to consolidation

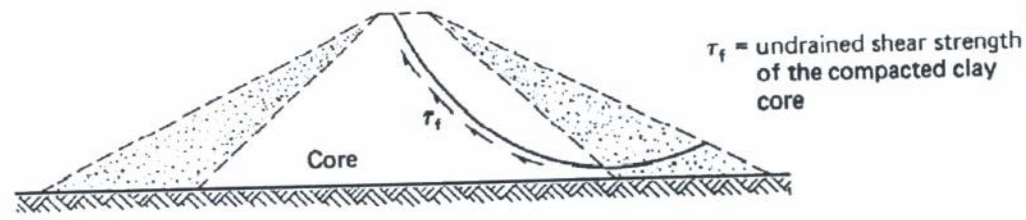
(b) Rapid drawdown behind an earth dam. No drainage of the core. Reservoir level falls from ① → ②.



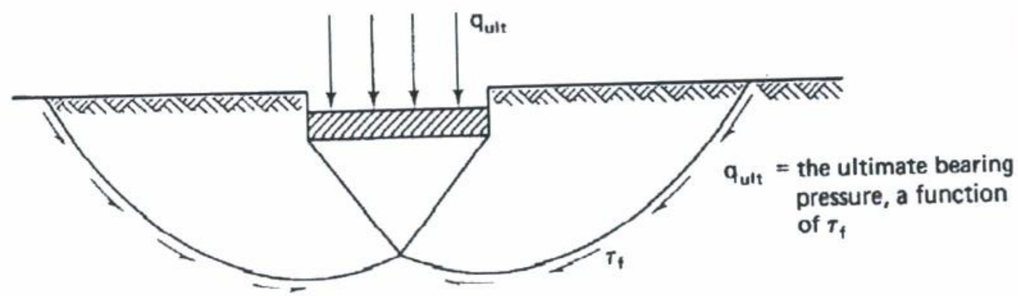
(c) Rapid construction of an embarkment on a natural slope.

Gambar 2. Beberapa Contoh Kasus CU (setelah Ladd, 1971)

IWS



(b) Large earth dam constructed rapidly with no change in water content of clay core



(c) Footing placed rapidly on clay deposit

Gambar 3. Beberapa Contoh Kasus UU (setelah Ladd, 1971)

IWS

Masalah	Informasi yang Dibutuhkan	Parameter yang Dibutuhkan	Uji Coba yang Dianjurkan	Catatan
Stabilitas lerang	Stabilitas jangka pendek	- Berat volume tanah - Undrained shear strength	Uji laboratorium: – Density test – Unconfined Comp. test – Triaxial UU comp. test	
	Stabiltas jangka panjang	- Berat volume tanah - Drained shear strength	Uji laboratorium: – Density test – Triaxial CU comp. test with pore pressure measurement – Triaxial CD comp. test	
		Tinggi muka air tanah permanen	Pengamatan langsung muka air pada water standpipe	Atau informasi dari otoritas yang relevan

Ref: Grandfathering HATTI, Mei 1999

eria

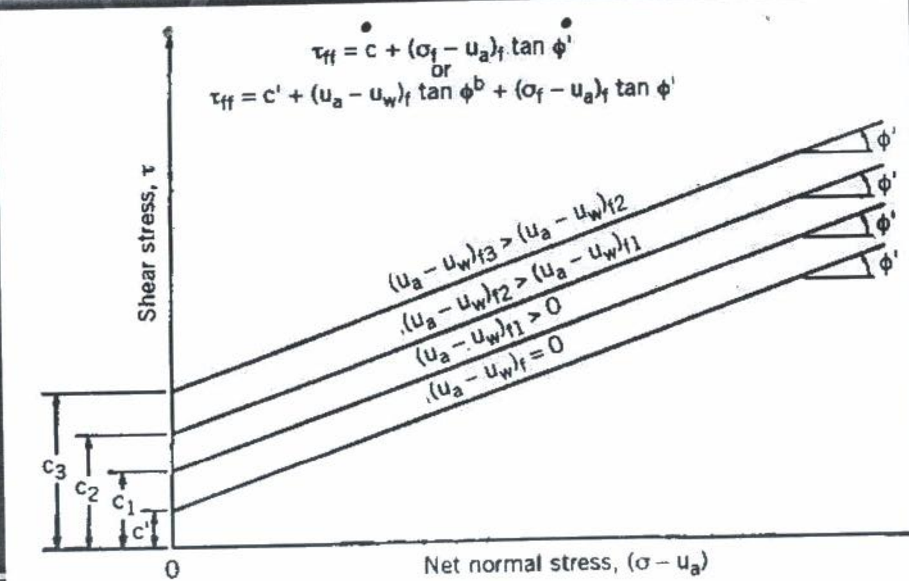
- soil sk
- pore water, and
- air

- Shear strength formulation:

$$s = c' + (\sigma - u_a) \tan \Phi' + (u_a - u_w) \tan \Phi^b$$

$s = c^* + (\sigma - u_a) \tan \Phi'$ where $c^* = [c' + (u_a - u_w) \tan \Phi^b]$

- u_a and u_w: pore air and water pressures,
- $(u_a - u_w)$ = matrix suction,
- Φ^b = slope matrix suction versus shear stress.



τff = c + (σf − ua)f tan φ'
or
τff = c' + (ua − uw)f tan φb + (σf − ua)f tan φ'
Shear stress, τ
(ua − uw)f3 > (ua − uw)f2
(ua − uw)f2 > (ua − uw)f1
(ua − uw)f1 > 0
(ua − uw)f = 0
φ'
c3
c2
c1
c'
0
Net normal stress, (σ − ua)

(*b*)

Wisconsin-Madion USA, 1982.

- Holtz R.D., and Kovacs, W.D., An Introduction to Geotechnical Engineering, Englewood Cliffs, New Jersey, Prentice Hall, 1981.
- Geotechnical Manual for Slopes, Geotechnical Control Office, Engineering Development Department, Hongkong, 1984.