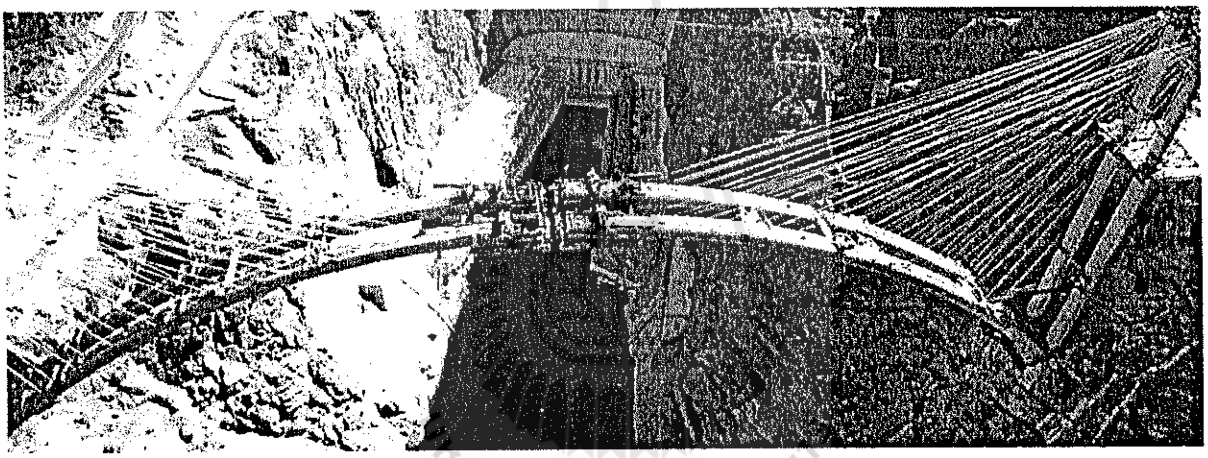
Lecture Note

434636 Foundations on Rock



prepared by

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Geological Engineering Program
Suranaree University of Technology

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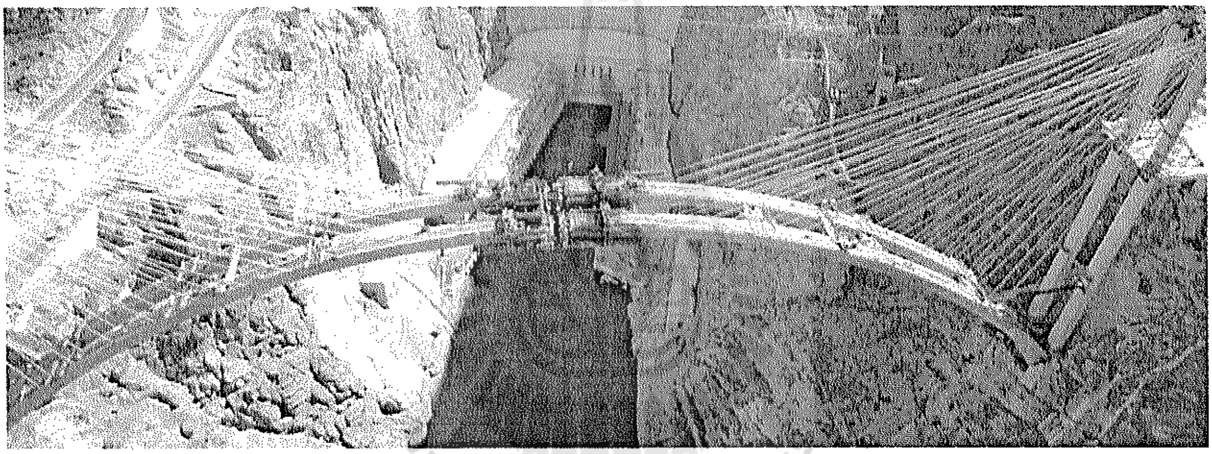

prepared by

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Geological Engineering Program
Suranaree University of Technology

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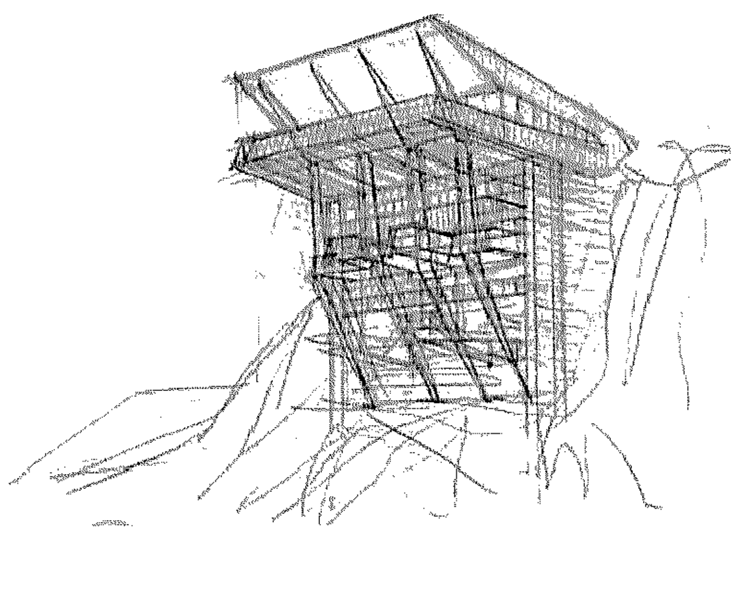
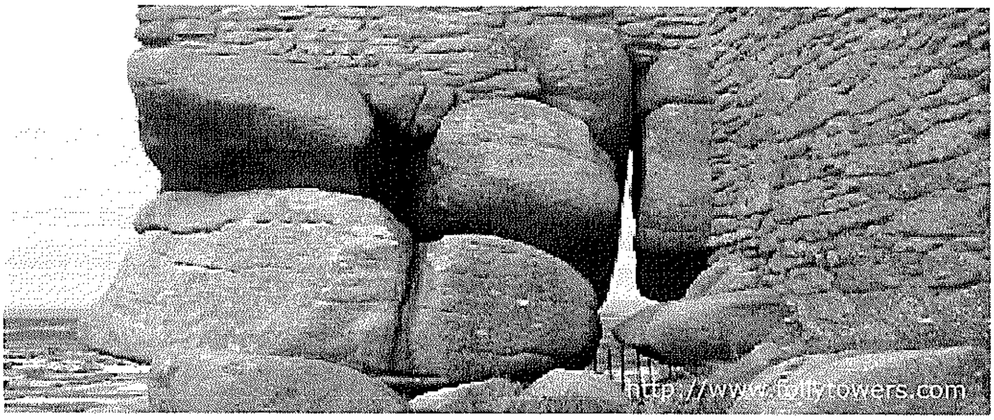




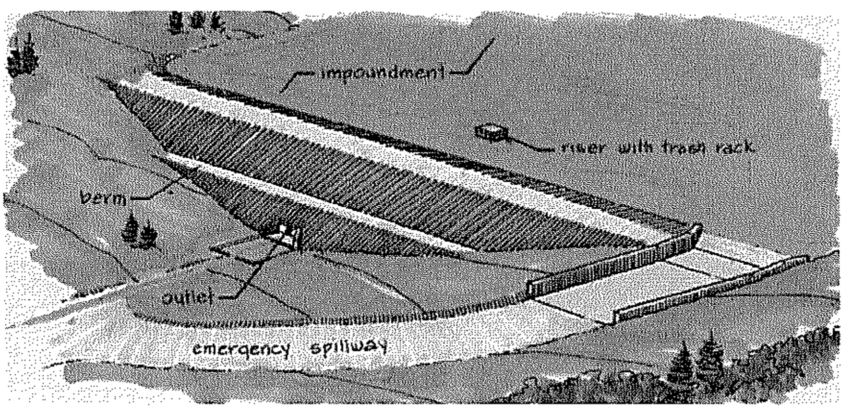
434636 Foundations on Rock
4 credits

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

434636 Foundations on Rock

Prerequisite: 434 370 Rock Mechanics or
or 505 530 Fundamental of Rock Mechanics

Instructor: Prachya Tepnarong, Ph.D.

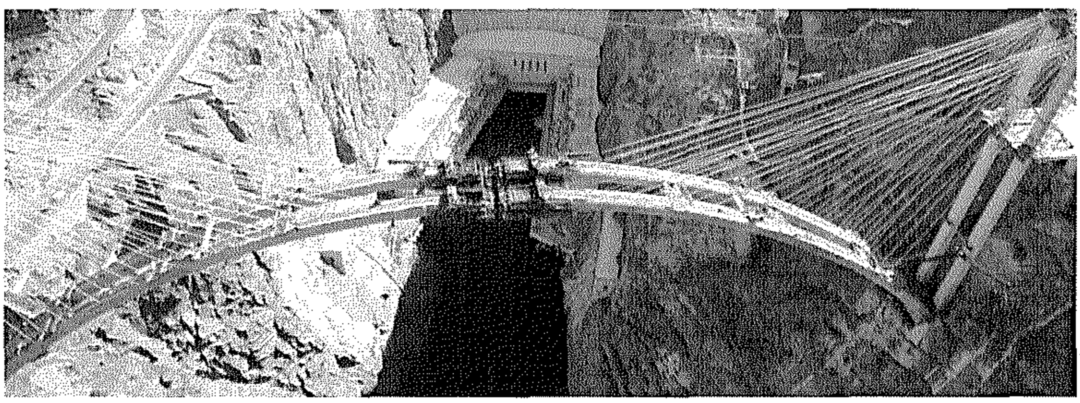


SYLLABUS:

Topic	1:	Introduction to Foundations on Rock
Topic	2:	Characteristics of Rock Foundation
Topic	3:	Rock Strength & Deformability
Topic	4:	Investigation & In-situ Testing
Topic	5:	Bearing Capacity, Settlement & Stress Distribution
	MIDTERM EXAM	
Topic	6:	Stability of Foundations
Topic	7:	Foundation of Gravity & Embankment Dams
Topic	8:	Rock Socket Piers
Topic	9:	Tension Foundation
	FINAL EXAM	

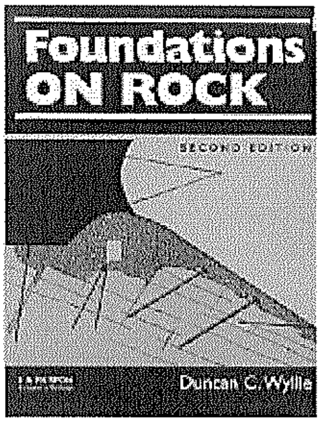
Scoring

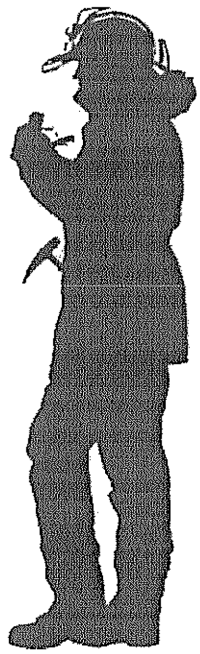
- Homework 20%
- Quiz 10%
- Term Project 20%
- Mid-term Exam 25%
- Final Exam 25%

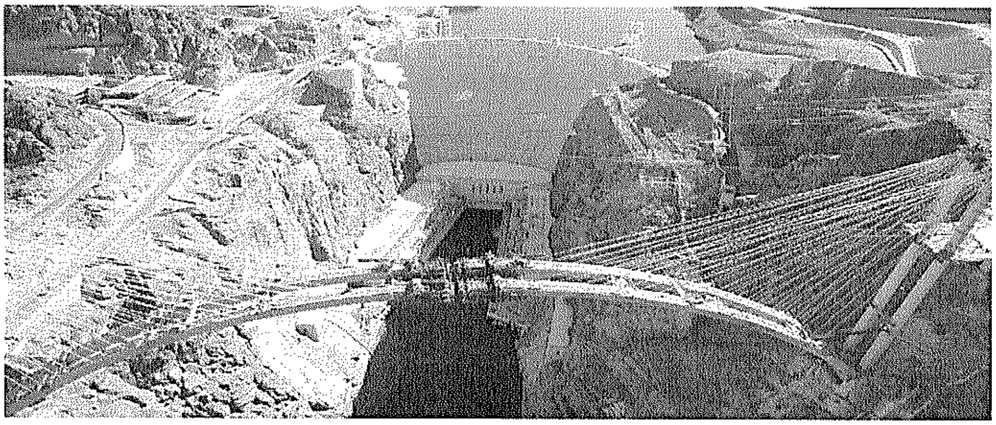


References:

- Wyllie, D.C., 1992, *Foundations on Rock*, 2nd edition Chapman & Hall, London.
- Jaeger, J.C. and N.G.W. Cook, 1979, *Fundamentals of Rock Mechanics*, 3rd edition, Chapman and Hall, London.







434636 Foundations on Rock

Topic 2 Characteristics of Rock Foundation

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Stability of Foundation on Rock

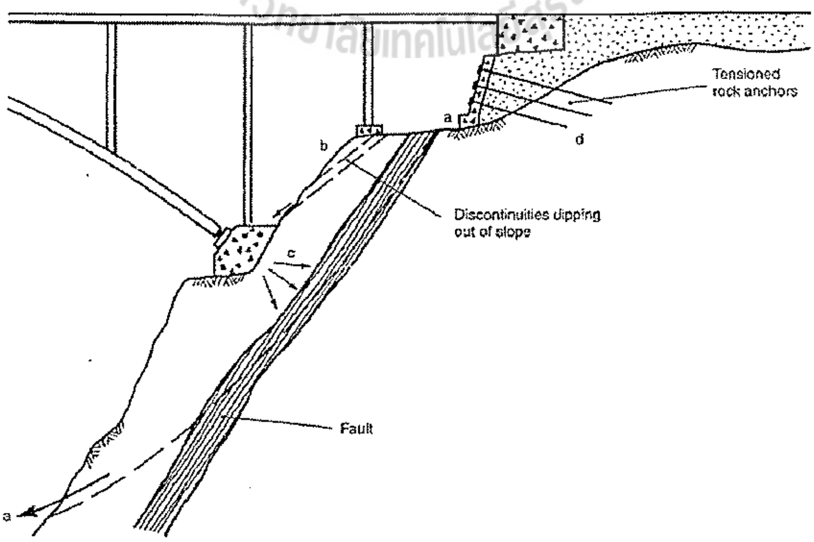


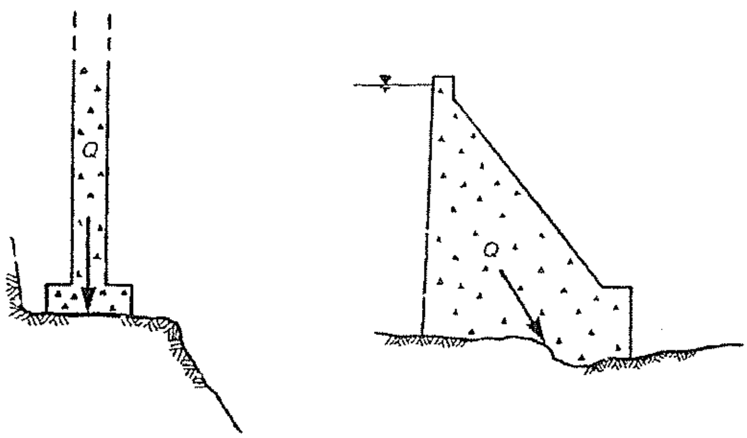
Figure 1.1 Stability of bridge abutment founded on rock: (a–a) overall failure of abutment on steeply dipping fault zone; (b) shear failure of foundation on daylighting joints; (c) movement of arch foundation due to compression of low-modulus rock; and (d) tied-back wall to support weak rock in abutment foundation.

Characteristics of Foundation

Types of Foundation

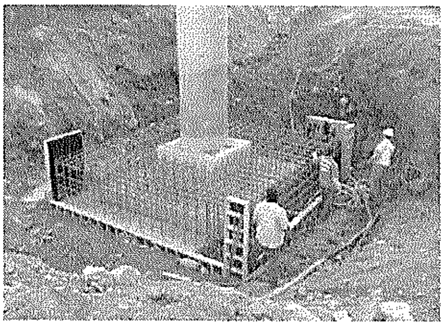
1. Spread Footing / Dam Foundation
2. Socket Piers
3. Tension Foundation

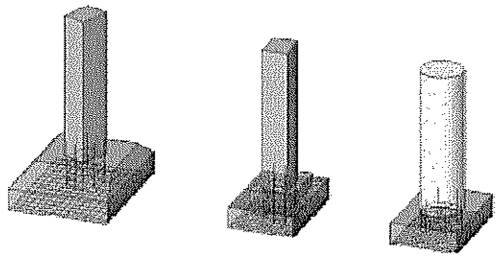
Spread Footing / Dam Foundation

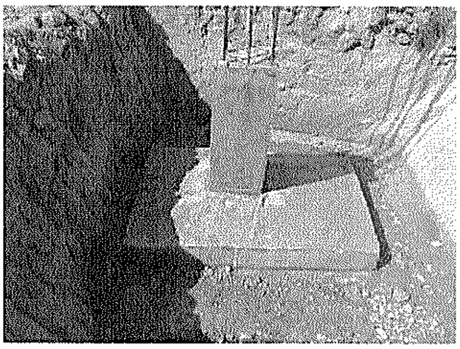


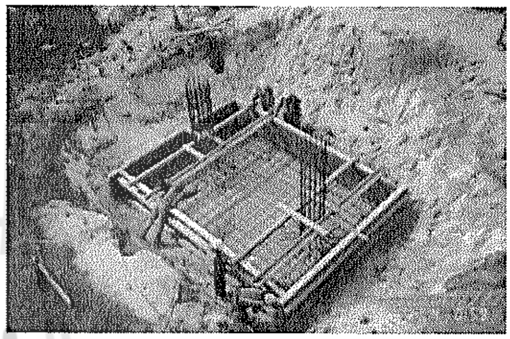
(a) (b)

Spread Footing

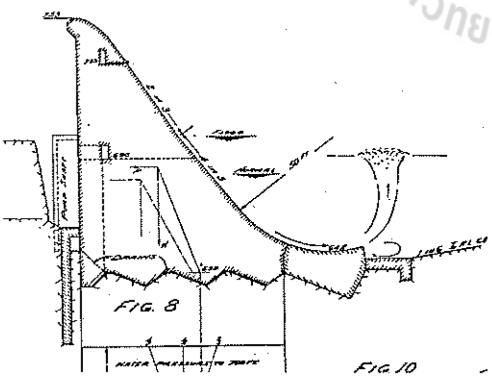


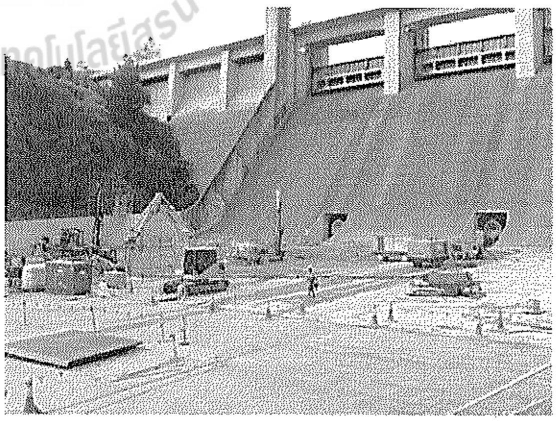


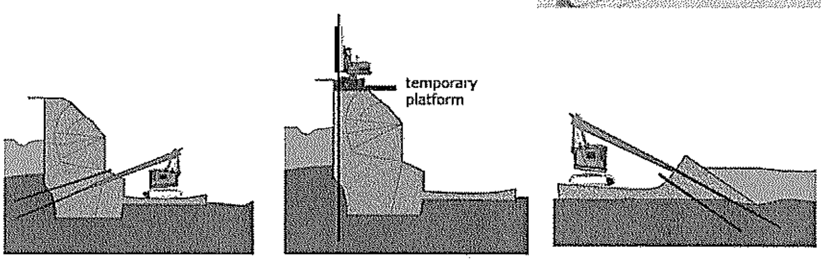




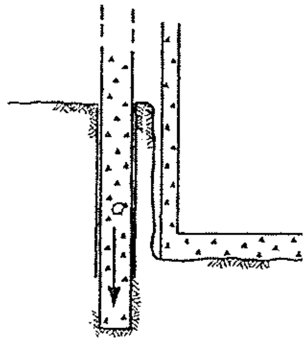
Dam Foundation







Socket Piers



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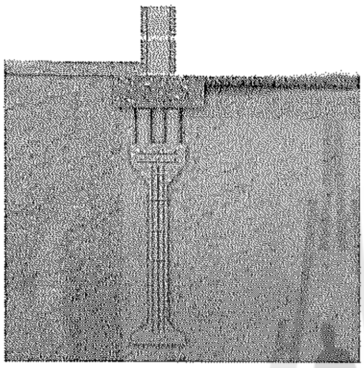
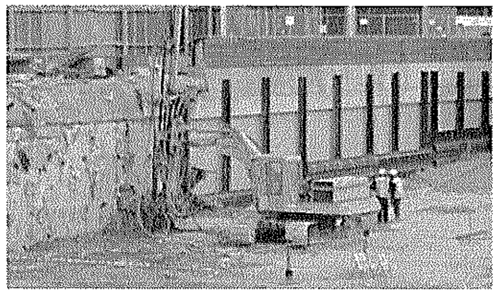
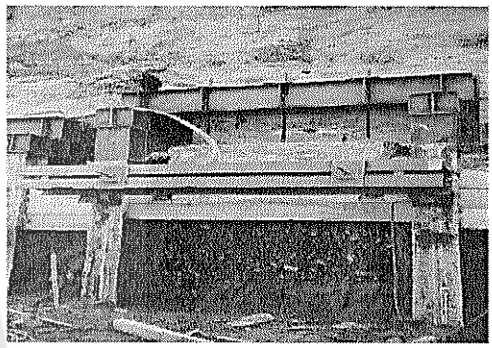
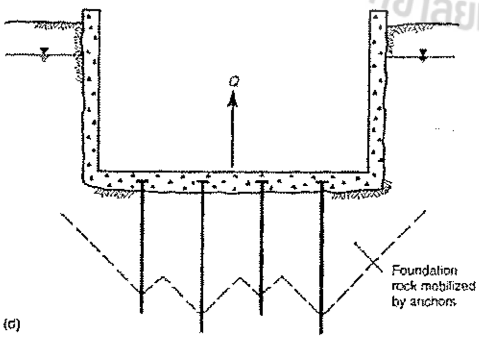






Tension Foundation (Anchors)



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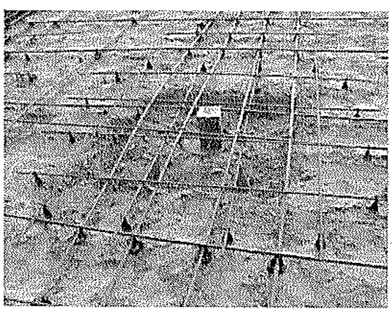
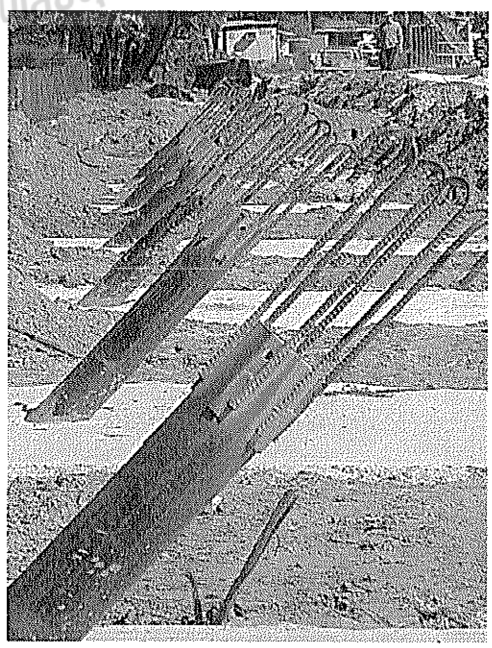




Performance of Foundation on Rock

Factors

1. Settlement and Bearing Capacity Failure
2. Creep (Time Dependent Deformation)
3. Block Failure
4. Failure of Socketed Piers and Tension Anchors
5. Influence of Geological Structure
6. Excavation Methods
7. Reinforcement

Retaining Wall Foundation

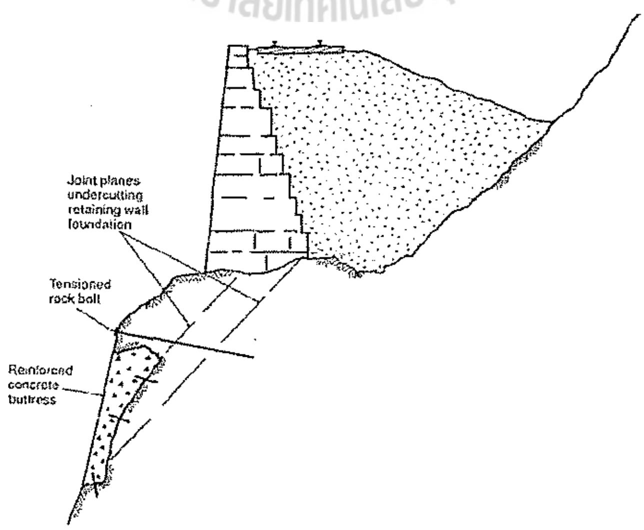


Figure 1.3 Retaining wall foundation stabilized with reinforced concrete buttress and rock bolts.

A typical effect of Geologic Conditions on Foundation Excavation

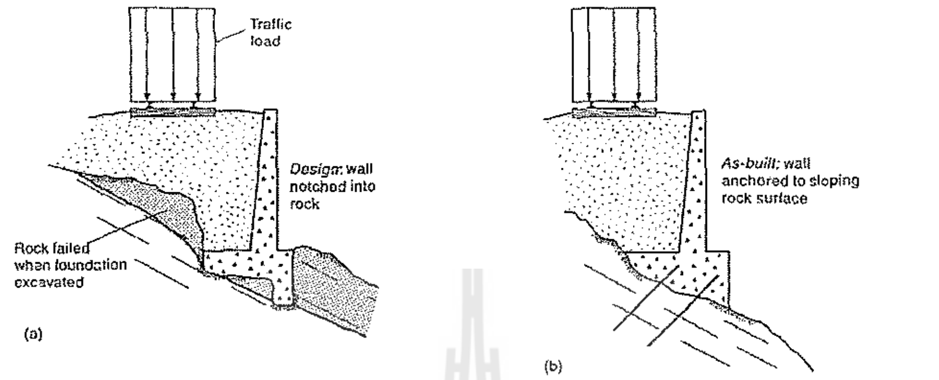


Figure 1.4 Construction of rock foundation: (a) attempted 'sculpting' of rock foundation to form shear key; and (b) 'as-built' condition with footing located on surface formed by joints.

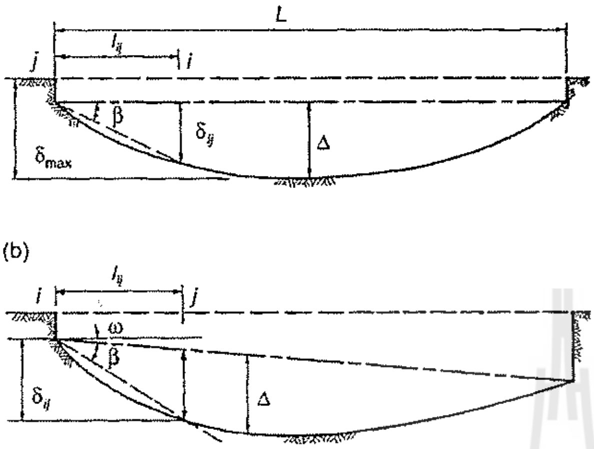
Structural Loads

- Building
- Bridges
- Dam
- Tension Foundation

Dead Load
Live Load
Additional Load

Allowable Settlement

- Building



β > 1/150 – structural damage probable;
β > 1/300 – cracking of load bearing or panel walls likely;
β < 1/500 – safe level of distortion at which cracking will not occur.

Allowable Settlement

- Bridge

3 categories depending on its effect on the structure

1. Tolerable movement
2. Intolerable movement (poor riding)
3. Intolerable movement (structure damage)

Allowable Settlement

- Dam

  Allowable settlement of dam is directly related to type of dam,

  for example:

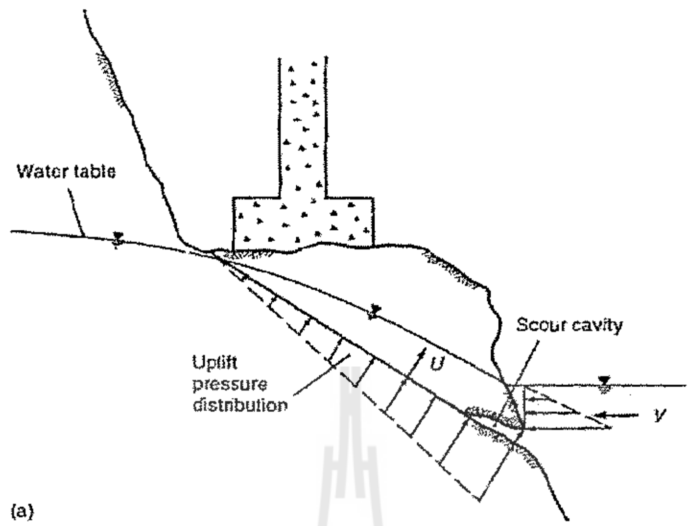
  concrete dam are much less tolerant of movement and deformation than embankment dam.

Influence of GW of Foundation Performance

- Foundation Stability
- Dams
- Tension Foundation

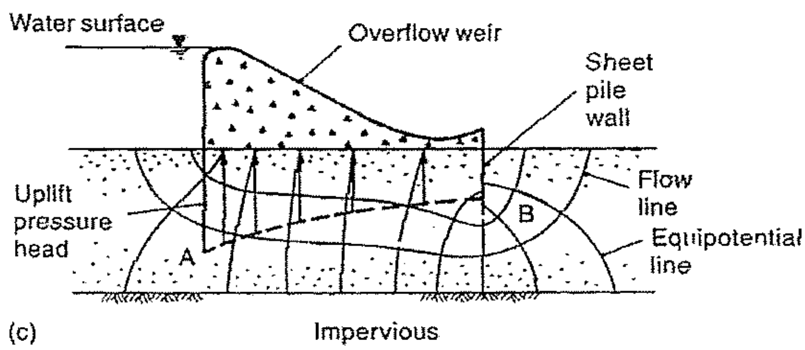
Typical Effects of GW Flow on Rock Foundation

- Foundation Stability



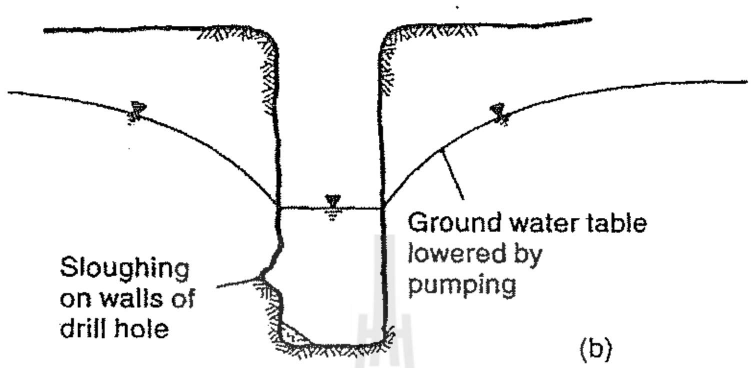
Typical Effects of GW Flow on Rock Foundation

- Dam



Typical Effects of GW Flow on Rock Foundation

- Tension Foundation



Factor of Safety & Reliability

- Factor of safety analysis

$$\text{Factor of safety}, FS = \frac{\Sigma(Resisting\ forces)}{\Sigma(Displacing\ forces)}$$

Table 1.1 Values of minimum total safety factors

Failure type	*Category*	*Safety factor*
Shearing	Earthworks	1.3 – 1.5
	Earth retaining structures, excavations	1.5 – 2.0
	Foundations	2 – 3

Example of condition that require the use of F.S.

- A limited drilling program that not adequately sample condition at the site.
- Absence of rock outcrops
- Inability to obtain undisturbed samples for strength testing.
- Absence of information on GW condition.
- Uncertainty in failure mechanism

Example of condition that require the use of F.S. (cont.)

- Uncertainty in load values (particularly environment factors)
- Concern regarding the quality of construction
- Lack of experience of local foundation
- Usage of structure (i.e. hospital, police station, fire hall, bridge on major transportation route)

Limit states design

Limit states design use partial F.S. (resistance factor)

$$\tau = f_c c + (\sigma - f_U U) f_\phi \tan \phi$$

Table 1.2 Values of minimum partial factors (Meyerhof, 1984)

Category	*Item*	*Load factor*	*Resistance factor*
Loads	Dead loads	(f_{DL}) 1.25 (0.8)	
	Live loads, wind, earthquake	(f_{LL}) 1.5	
	Water pressure (U)	(f_U) 1.25 (0.8)	
Shear strength	Cohesion (c) – stability, earth pressure		(f_c) 0.65
	Cohesion (c) – foundations		(f_c) 0.5
	Friction angle (ϕ)		(f_ϕ) 0.8

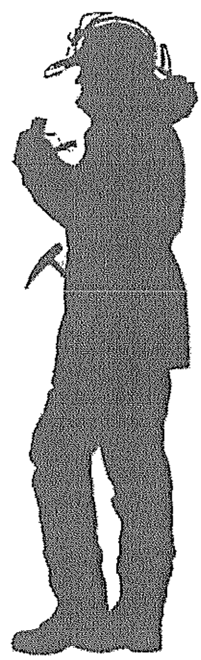
Sensitivity Analysis

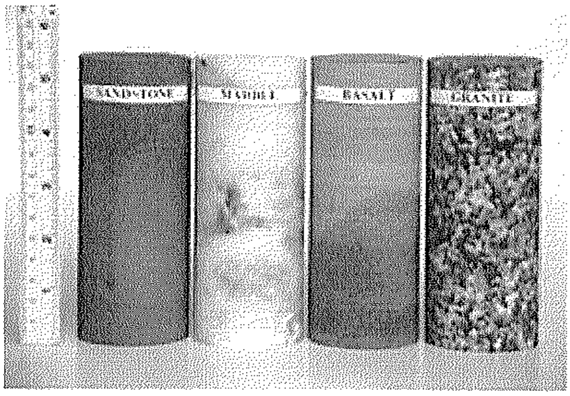
- Sensitivity ↑ → F.S. ↓
- Sensitivity ↓ → F.S. ↑

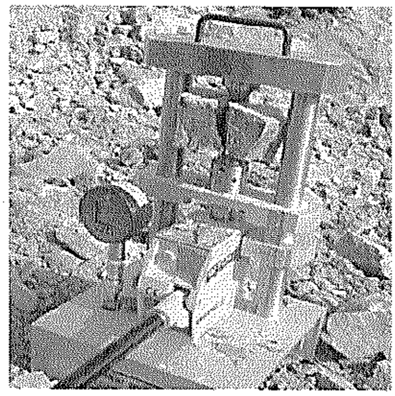
Coefficient of Reliability

- CR = (1-PF)

PF = Probability of Failure







434636 Foundations on Rock

Topic 3 Rock Strength and Deformability

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Range of Rock Strength Conditions

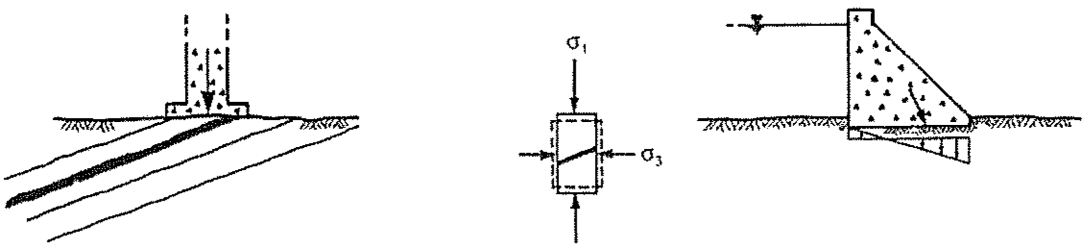
- Determination of appropriate strength parameters to use in design of foundations depend on;
  - Type of foundation
  - Load condition
  - Characteristic of rock in bearing area

Basic Rock Strength Parameters

1. Deformation Modulus
2. Compressive Strength (rock mass)
3. Compressive Strength (intact rock)
4. Shear Strength
5. Tensile Strength
6. Time Dependent

Basic Rock Strength Parameters

1. Deformation Modulus
   - calculate of settlement

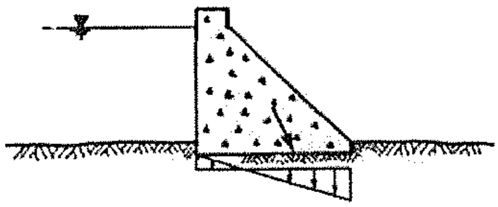


(a) (b)

Basic Rock Strength Parameters

2. Compressive Strength of Rock Mass

   ➲ bearing capacity of spread footing

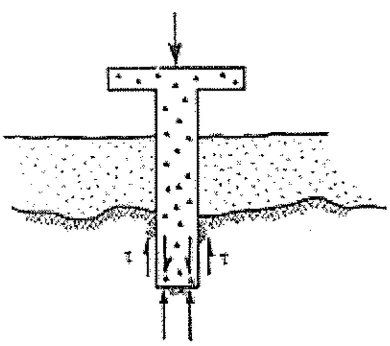


(b)

Basic Rock Strength Parameters

3. Compressive Strength of Intact Rock

   ➲ bond stress of socketed and tensioned anchors is correlated with intact rock strength on the basis of empirical tests.



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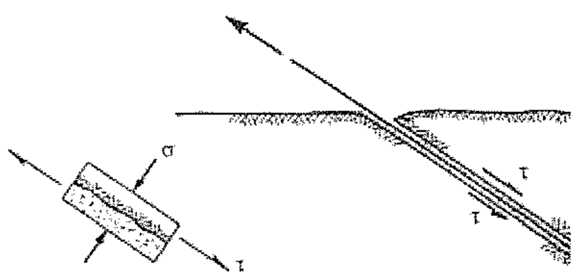
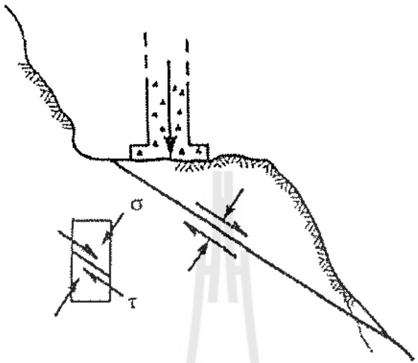


(d)

Basic Rock Strength Parameters

4. Shear Strength

   ➩ shear resistance at interface b/w structure and foundation, and stability of sliding block.

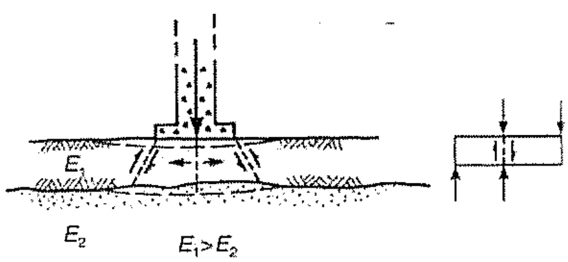


(e)

Basic Rock Strength Parameters

5. Tensile Strength

   ➩ punching of flexural failures where a week bed underlies a layer of stiffer rock.



(f)

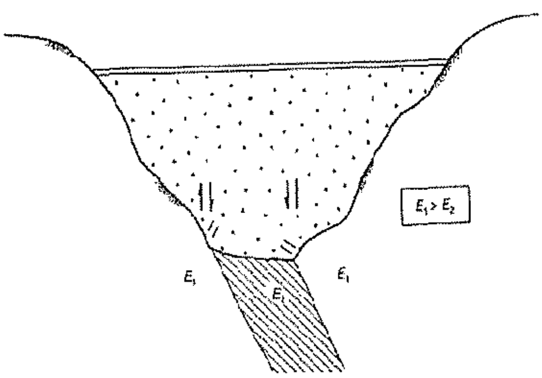
Basic Rock Strength Parameters

6. Time Dependent Properties

   ➲ settlement may occur with time as a result of creep, or degradation of the rock due to weathering.

Deformation Modulus

- Differential settlement can induce stresses in the concrete sufficient to develop cracking.



Deformation Modulus

In-situ Testing

- Borehole Pressuremeter
- Plate Load
- Flat Jacks
- Pressure Chamber
- Geophysical Testing

Deformation Modulus

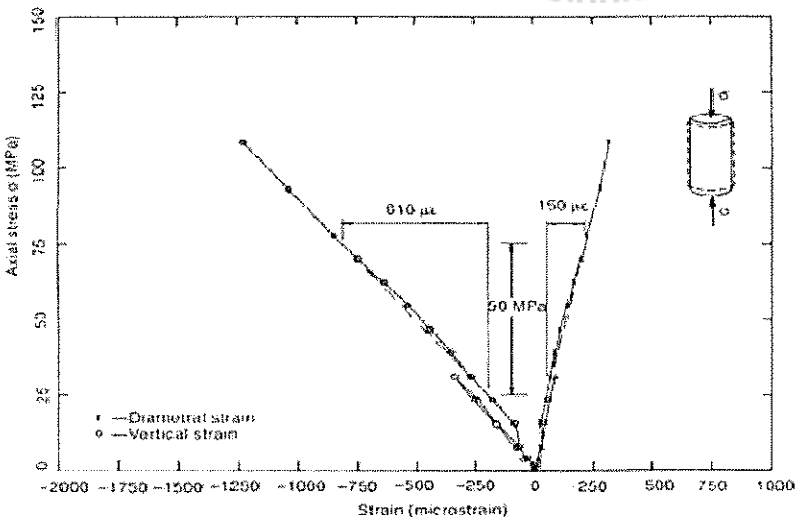
Definitions (ISRM, 1975)

- Deformation Modulus – the ratio of stress to corresponding strain during loading of a rock mass including elastic and inelastic behavior.
- Elastic Modulus – the ratio of stress to corresponding strain below the proportional limit of a material.

Deformation Modulus

1. Intact Rock Modulus
2. Stress-strain Behavior
3. Size Effect on Deformation Modulus
4. Discontinuity Spacing and Modulus
5. Modulus of Anisotropic rock

Intact Rock Modulus



$$\text{Young's modulus} = \text{Vertical stress/Strain} = 50.0\ \text{MPa}/610E-6 = 82\ \text{GPa}(11.9 \times 10^6\ \text{p.s.i.})$$

$$\text{Poisson's ratio} = \text{Diametral strain/Vertical strain} = 150E-6/610E-6 = 0.25$$

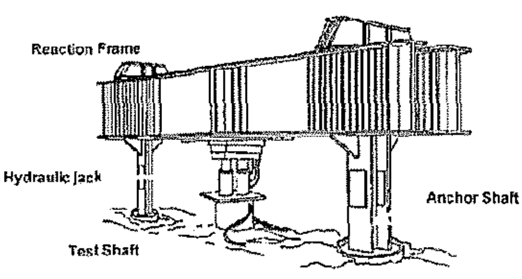
Intact Rock Modulus

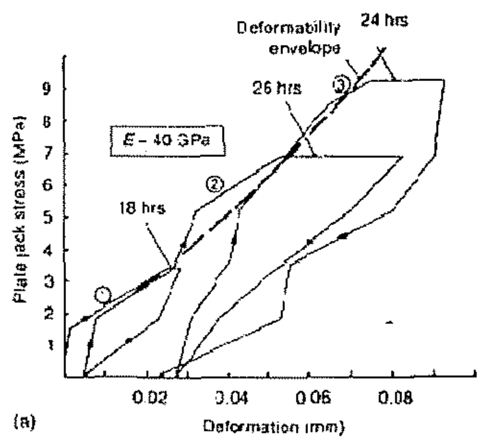
Table 3.1 Typical elastic constants for intact rock

Rock type	*Young's modulus* GPa (p.s.i. × 10^6)	*Poisson's ratio*	*Reference*
Andesite, Nevada	37.0 (5.5)	0.23	Brandon (1974)
Argillite, Alaska	68.0 (9.9)	0.22	Brandon (1974)
Basalt, Brazil	61.0 (8.8)	0.19	Ruiz (1966)
Chalk, USA	2.8 (0.4)	–	Underwood (1961)
Chert, Canada	95.2 (13.8)	0.22	Herget (1973)
Claystone, Canada	0.26 (0.04)	–	Brandon (1974)
Coal, USA	3.45 (0.5)	0.42	Ko and Gerstle (1976)
Diabase, Michigan	68.9 (10)	0.25	Wuerker (1956)
Dolomite, USA	51.7 (7.5)	0.29	Haimson and Fairhurst (1970)
Dolomite, Canada	64.0 (9.3)	0.29	Lo and Hori (1979)
Gneiss, Brazil	79.9 (11.6)	0.24	Ruiz (1966)
Granite, California	58.6 (8.5)	0.26	Michalopoulos and Triandafilidis (1976)
Limestone, USSR	53.9 (8.5)	0.32	Belikov (1967)
Salt, Ohio	28.5 (4.1)	0.22	Sellers (1970)
Sandstone, Germany	29.9 (4.3)	0.31	van der Vlis (1970)
Shale, Japan	21.9 (3.2)	0.38	Kitahara *et al.* (1974)
Siltstone, Michigan	53.0 (7.7)	0.09	Parker and Scott (1964)
Tuff, Nevada	3.45 (0.5)	0.24	Cording (1967)

Stress-strain Behavior of Fracture Rock

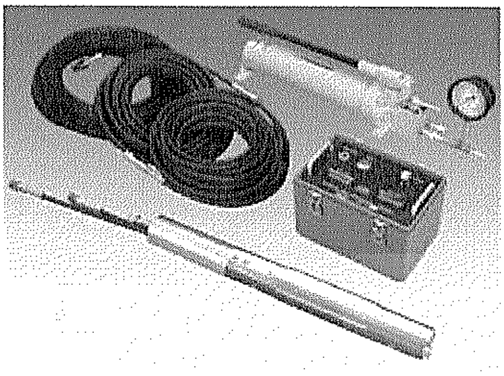
Plate Load Test

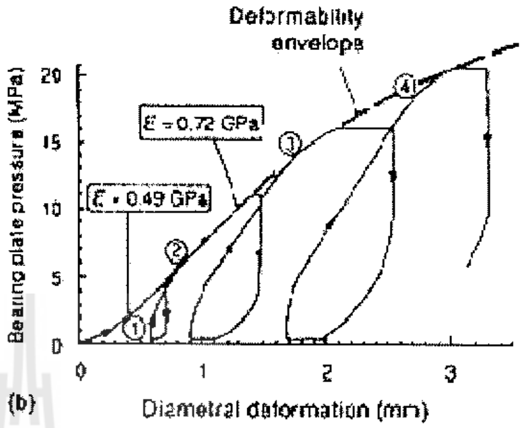




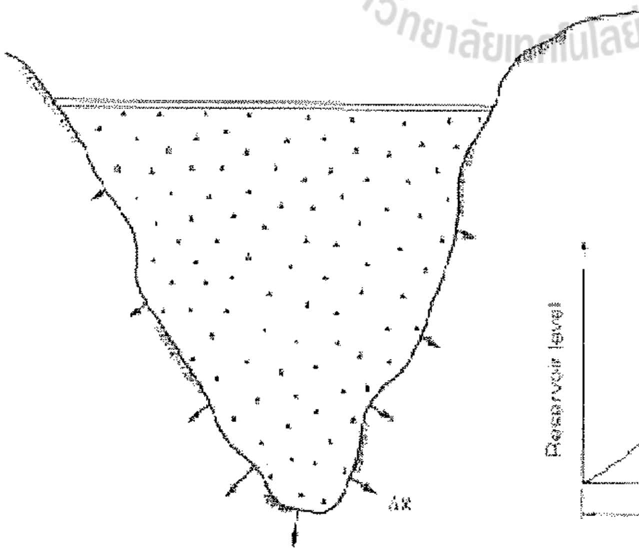
Stress-strain Behavior of Fracture Rock

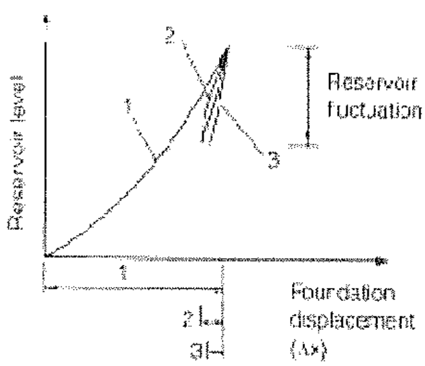
Goodman Jack Test



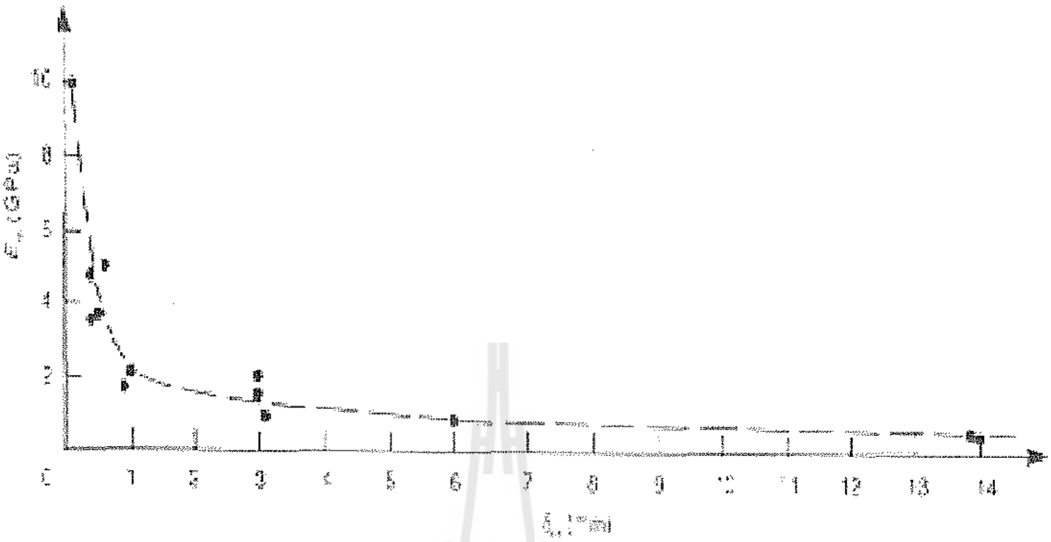


Stress-strain Behavior of Fracture Rock





Stress-strain Behavior of Fracture Rock



Size Effect on Deformation Modulus

$$E_{static} < E_{earthquake} < E_{seismic} < E_{intact\ rock}$$

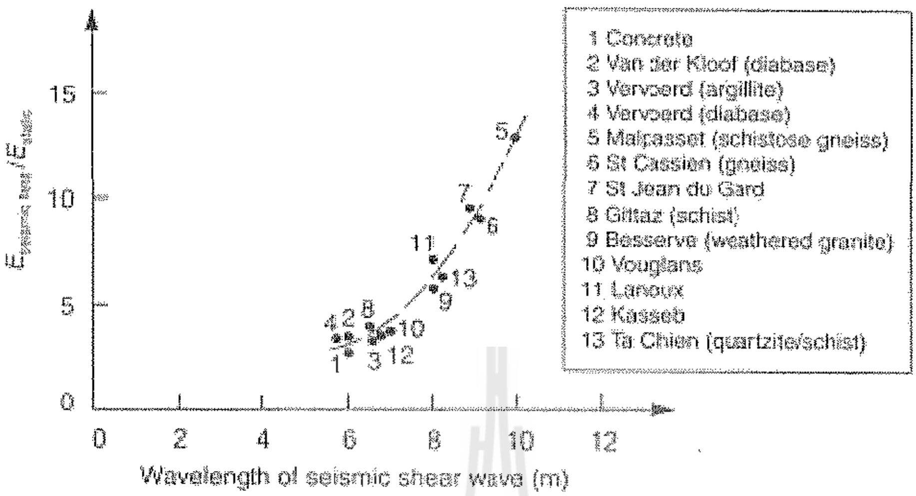
E_{static} - Modulus for rock load by plate bearing, borehole jack or dilatometer

$E_{earthquake}$ -Modulus for rock mass subjected to shaking at 1-10 Hz

$E_{seismic}$ -Modulus for rock mass subjected shock wave with > 100 Hz

$E_{intact\ rock}$ -Modulus for intact rock specimen

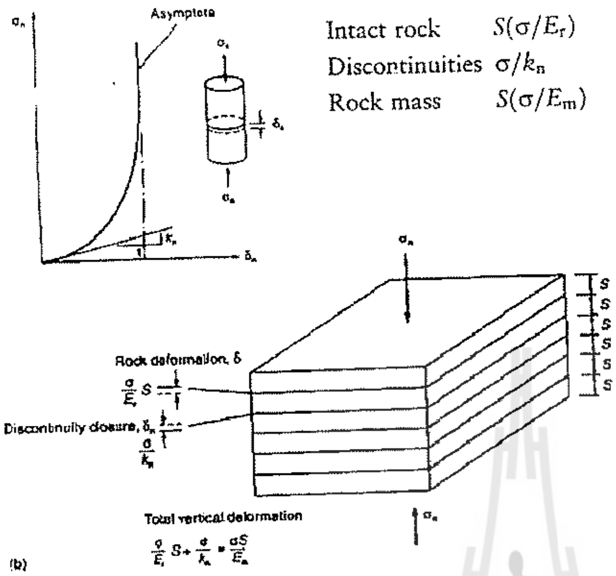
Size Effect on Deformation Modulus



Size Effect on Deformation Modulus

Type of test	*Number of tests*	*Mean ratio*
Platebearing	27	3.1
Full scale deformation	14	2.4
Flat jacks	10	1.9
Borehole jack or dilatometer	9	3.0
Pressure chamber	8	2.2
Petit seismique	5	2.9
Others	5	2.4

Discontinuity Spacing and Modulus



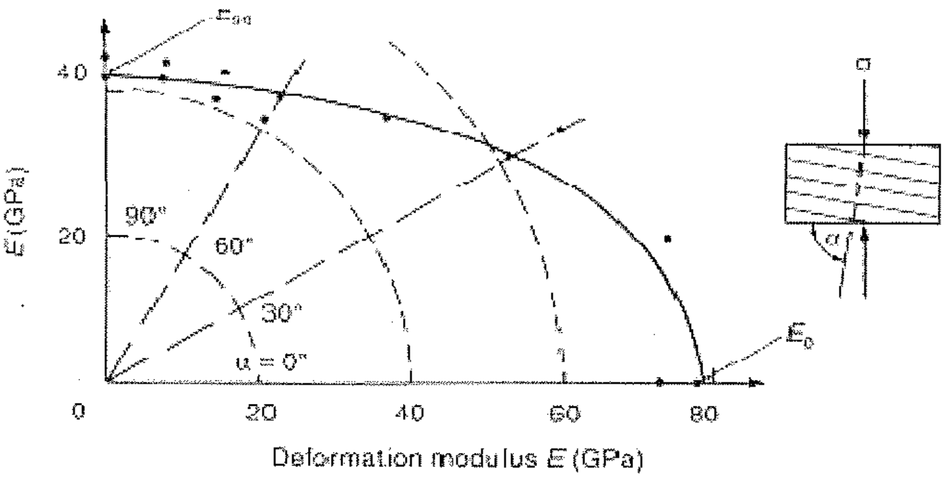
$$k_n = \frac{\sigma}{\delta_n}$$

$$\frac{1}{E_m} = \frac{1}{E_r} + \frac{1}{k_n S}$$

$$\frac{1}{G_m} = \frac{1}{G_r} + \frac{1}{k_s S}$$

S = Fracture Spacing
k_n = Normal Stiffness
k_s = Shear Stiffness
E = Elastic Modulus
G = Shear Modulus

Modulus of Anisotropic Rock

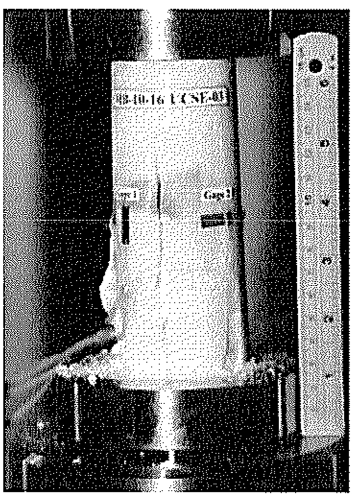


Modulus of Anisotropic Rock

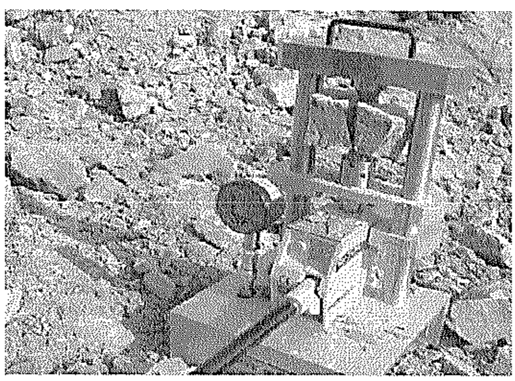
Table 3.4 Modulus ratios of anisotropic rock

Rock type	E_0/E_{90}	*Reference*
Clay shale	1.36–2.86	Stepanov and Batugin (1967)
Slate	1.7	Bamford (1969)
Phyllite	1.28–1.33	Lekhnitskii (1966)
Schist	1.3–3.2	Pinto (1970)

Compressive Strength of Intact Rock



UCS – ASTM 7012-04
(ASTM-2938 ,be replaced)



Point Load Test – ASTM 5731-05

Point Load Strength Test

Designation: D 5731 – 05

Standard Test Method for
Determination of the Point Load Strength Index of Rock[1]

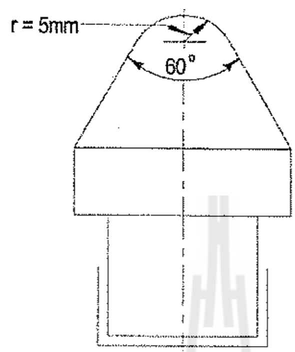
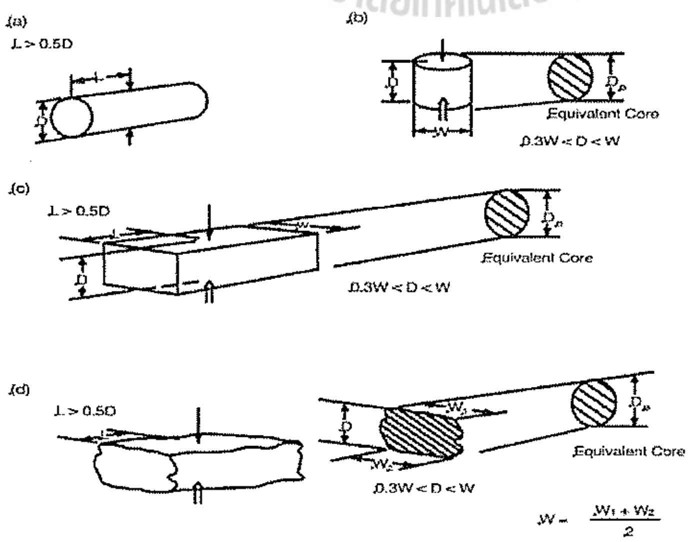


FIG. 2 Platen Dimensions

Point Load Index



NOTE 1—Legend: L = length, W = width, D = depth or diameter, and D_e = equivalent core diameter (see 9.1).
FIG. 3 Load Configurations and Specimen Shape Requirement for (a) the Diametral Test, (b) the Axial Test, (c) the Block Test, and (d) the Irregular Lump Test[3]

Point Load Index

$$I_s = P/D_e^2, \text{MPa}$$

where:

P = failure load, N,

D_e = equivalent core diameter = D for diametral tests (see Fig. 3), m, and is given by:

D_e^2 = D^2 for cores, mm^2, or

D_e^2 = $4A/\pi$ for axial, block, and lump tests, mm^2;

where:

A = WD = minimum cross-sectional area of a plane through the platen contact points (see Fig. 3).

$D_e^2 = D \times D'$ for cores $= 4/\pi\ W \times D'$ for other shapes

Size Correction Factor

$$I_{s(50)} = F \times I_s$$

The "Size Correction Factor F" can be obtained from the chart in Fig. 6, or from the expression:

$$F = (D_e/50)^{0.45} \tag{4}$$

For tests near the standard 50-mm size, only slight error is introduced by using the approximate expression:

$$F = \sqrt{(D_e/50)} \tag{5}$$

Size Correction Factor

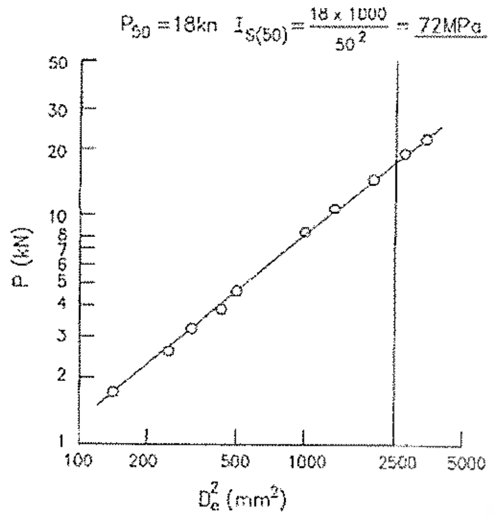


FIG. 5 Procedure for Graphical Determination of $I_{s(50)}$ from a Set of Results at D_e Values Other Than 50 mm [3]

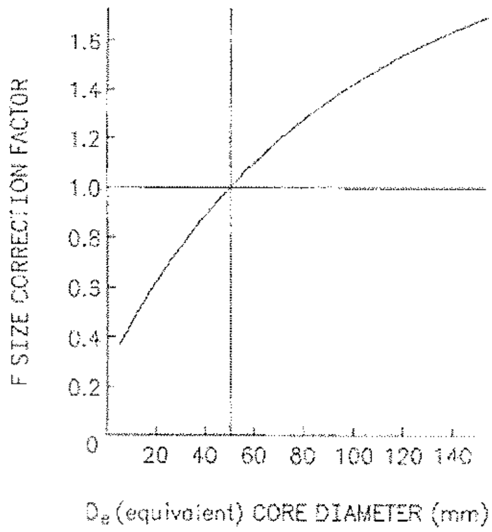


FIG. 6 Size Correction Factor Chart [3]

Estimate of Compressive Strength

$$\delta_{uc} = C\, I_{s(50)}$$

where:

δ_{uc} = uniaxial compressive strength,

C = factor that depends on site-specific correlation between δ_{uc} and $I_{s(50)}$, and

$I_{s(50)}$ = corrected point load strength index.

Estimate of Compressive Strength

TABLE 1 Generalized Value of "C"[A]

Core Size, mm	Value of "C" (Generalized)
20	17.5
30	19
40	21
50	23
54	24
60	24.5

[A] From ISRM Suggested Methods.[3]

Compressive Strength of Fracture Rock

- Hoek and Brown Criterion

$$\sigma_1' = \sigma_3' + (m\sigma_{u(r)}\sigma_3' + s\sigma_{u(r)}^2)^{1/2}$$

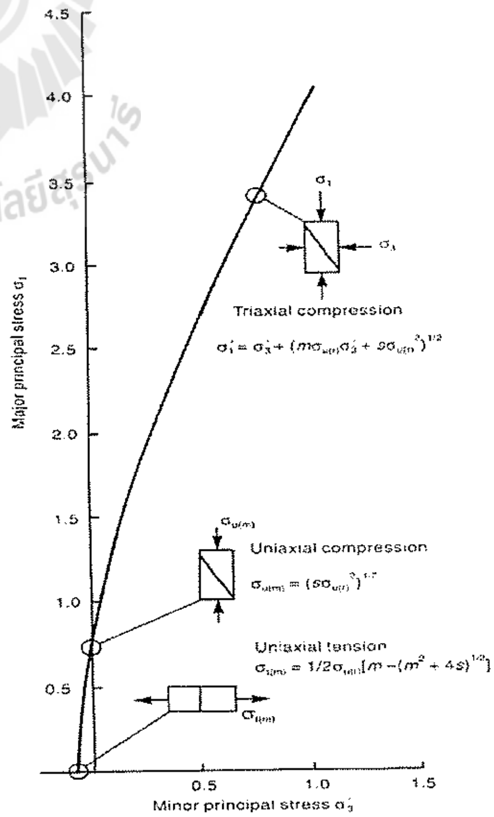
σ'$_1$ = maximum principal effective stress

σ'$_3$ = minimum principal effective stress (confining stress)

σ'$_{u(r)}$ = UCS of intact rock

m,s = dimensionless constant

$$\sigma_{u(m)} = (s\sigma_{u(r)}^2)^{1/2} \quad \text{or} \quad \frac{\sigma_{u(m)}}{\sigma_{u(r)}} = s^{1/2}$$



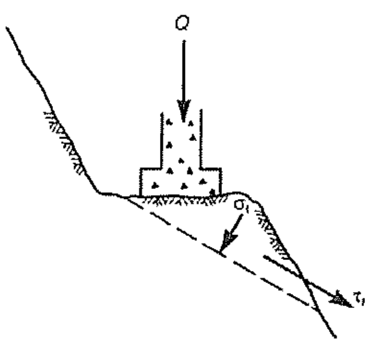
Shear Strength

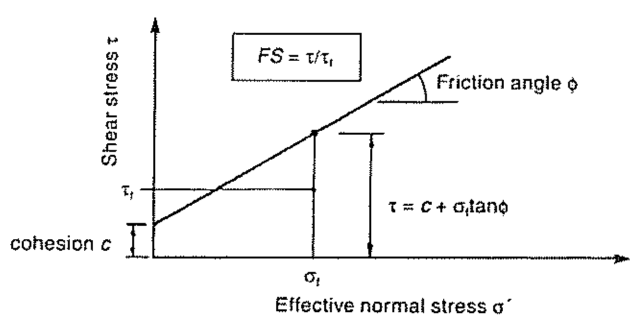
- Mohr-Coulomb Material
- Shear Strength of Discontinuities
- Shear Strength Testing
- Shear Strength of Fracture Rock

Shear Strength

- Mohr-Coulomb Material

$$\tau = c + \sigma' \tan \phi$$





Shear Strength

- Shear Strength of Discontinuities
  - Friction Angle
  - Surface Roughness
  - Cohesion
  - Infillings

Friction Angle

- Low (20°-27°)
  - Schist (high mica content)
  - Shale
  - marl
- Medium (27°-34°)
  - Sandstone / siltstone
  - Chalk
  - Gneiss / slate
- High (34°-40°)
  - Basalt
  - Granite
  - Limestone
  - conglomerate

Surface Roughness

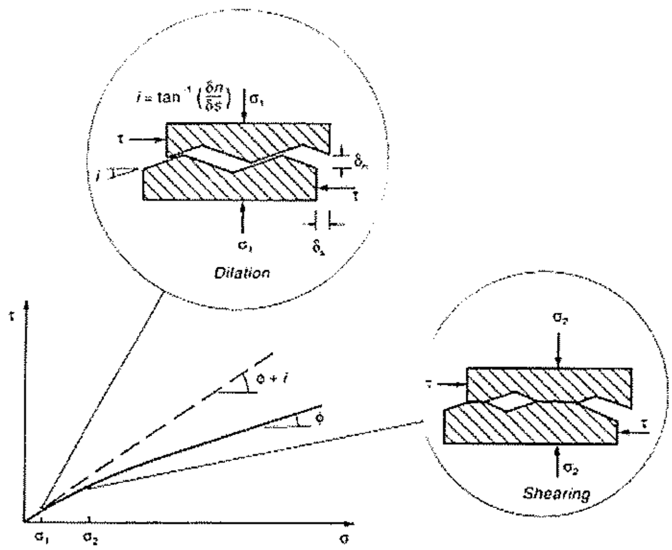
- Patton, 1966

$$\tau = c + \sigma' \tan(\phi + i)$$

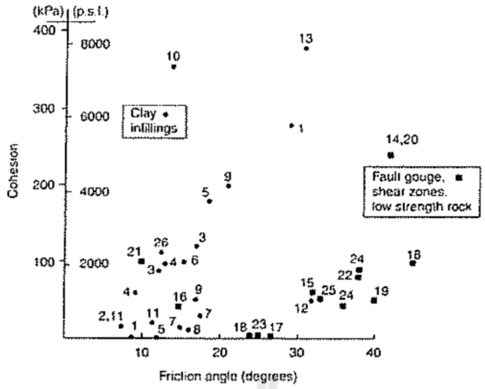
- Barton, 1973

$$\tau = \sigma' \tan\left(\phi + JRC \log_{10} \frac{JCS}{\sigma'}\right)$$

Effect of Surface Roughness



Cohesion

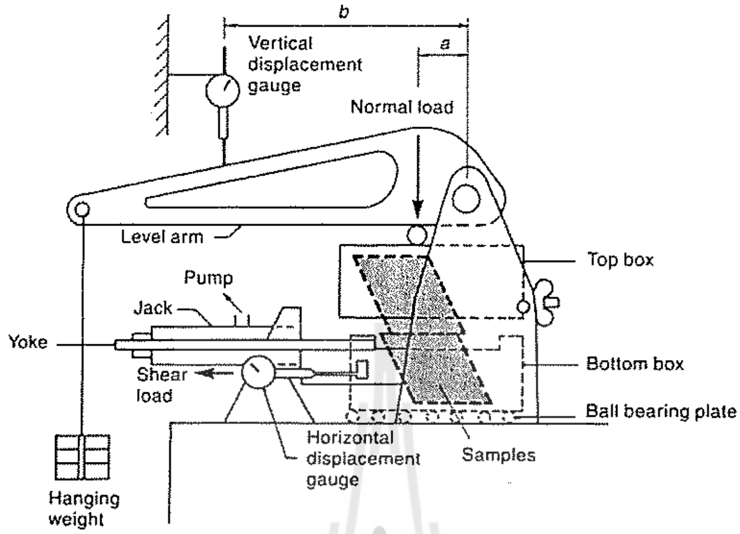


1. Bentonitic shale. 2. Bentonite seams in chalk. 3. Bentonite; thin layers. 4. Bentonite triaxial tests. 5. Clay, over-consolidated. 6. Limestone, 10–20 mm clay infillings. 7. Lignite and underlying clay contact. 8. Coal measures, clay mylonite seams. 9. Limestone; <1 mm clay infillings. 10. Montmorillonite clay. 11. Montmorillonite; 80 mm clay seam in chalk. 12. Schists/quartzites; 100–150 mm thick infilling. 13. Schists/quartzites; stratification, thick clay. 14. Basalt; clayey, basaltic breccia. 15. Clay shale; triaxial tests. 16. Dolomite, altered shale bed. 17. Diorite/granodiorite; clay gouge. 18. Granite; clay-filled faults. 19. Granite; sandy-loam fault filling. 20. Granite, shear zone, rock and gouge. 21. Lignite/marl contact. 22. Limestone/marl/lignites; lignite layers. 23. Limestone; marlaceous joints. 24. Quartz/kaolin/pyrolusite; remolded triaxial. 25. Slates; finely laminated and altered. 26. Limestone; 10–20 mm clay infillings.

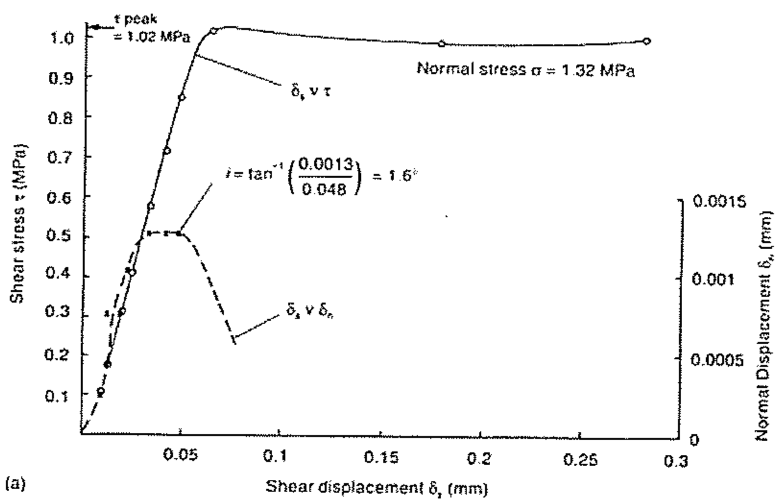
Infillings

- Clay
- Fault, shear and breccias

Shear Strength Testing



Shear Strength Testing



Shear Strength of Fracture Rock

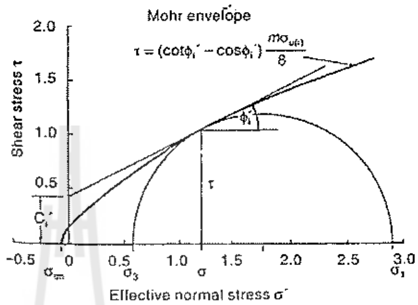
- Back Analysis of Failure
- Curve Shear Strength Envelopes
  (Hoek-Brown Strength Criterion)

$$\phi_i' = \arctan\left[\frac{1}{(4h\cos^2\theta - 1)^{1/2}}\right]$$

where

$$h = 1 + \frac{16(m\sigma' + s\sigma_{u(r)})}{3m^2\sigma_{u(r)}}$$

$$\theta = \frac{1}{3}\left\{90 + \arctan\left[\frac{1}{(h^3 - 1)^{1/2}}\right]\right\}$$



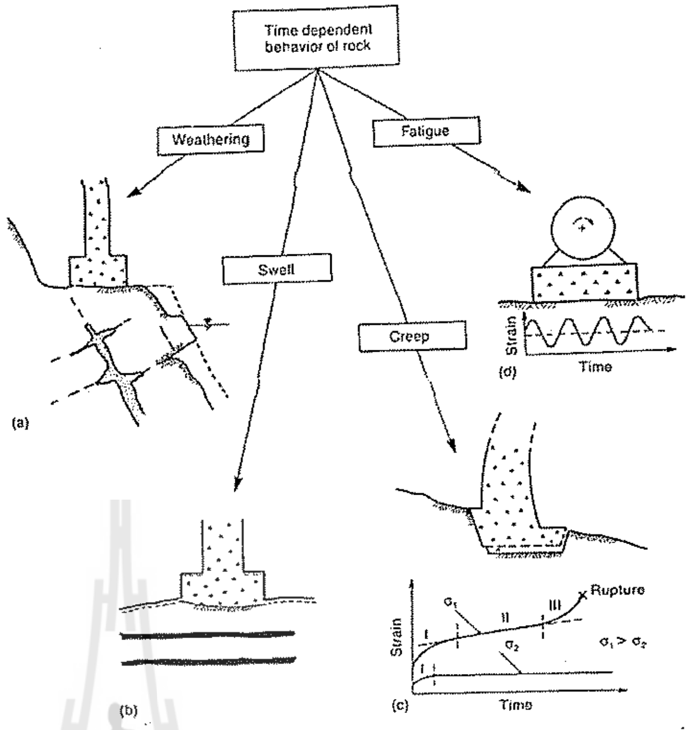
Tensile Strength of Rock mass

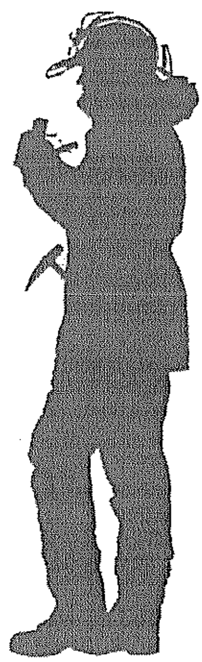
- Non-linear strength envelop
  (Hoek and Brown, 1988)

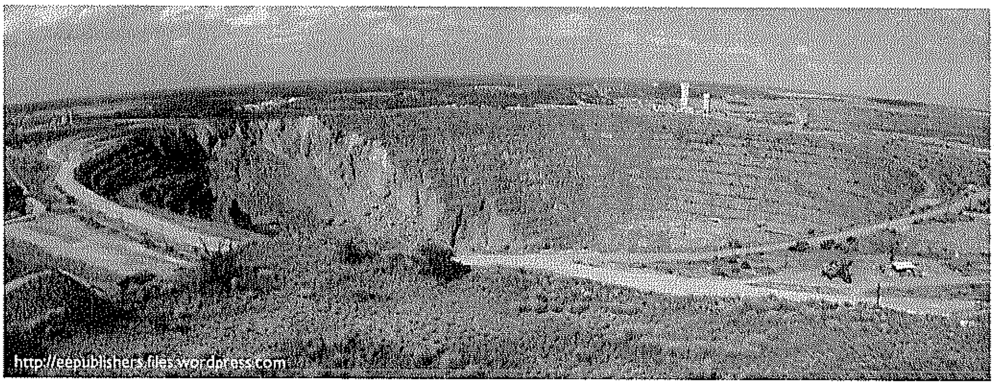
$$\sigma_t = 0.5\sigma_{u(r)}[m - (m^2 + 4s)^{1/2}]$$

Time-dependent Properties

1. Weathering
2. Swelling
3. Creep
4. Fatigue







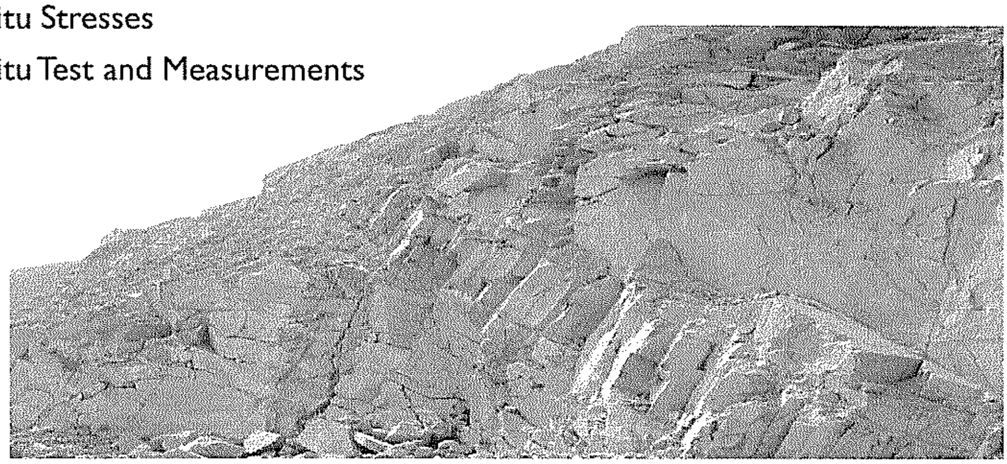
434636 Foundations on Rock

Topic 4 Rock Mass Investigation and In-situ Testing Methods

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Outline

- Investigation
- Rock Mass Characterization
- Rock Mass Classifications
- Strength of Rock Mass
- In-situ Stresses
- In-situ Test and Measurements



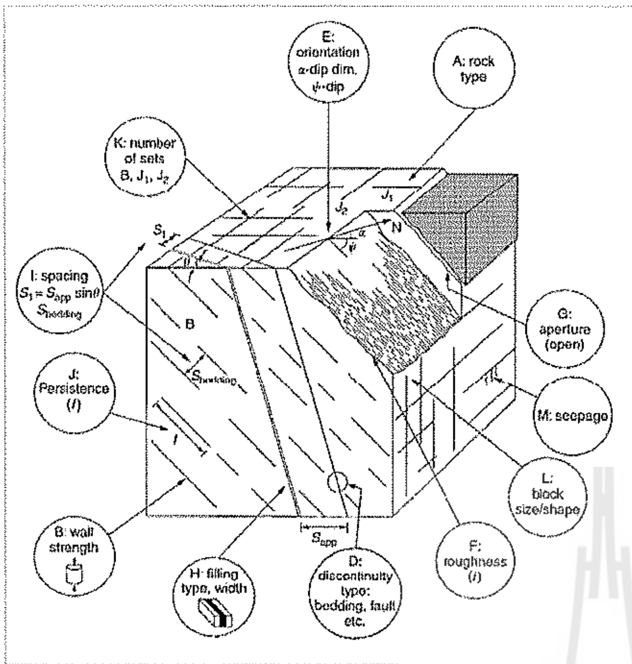
Investigation

1. Site selection
2. Geologic mapping
3. Drilling
4. Groundwater measurements
5. In-situ modulus and shear strength testing

4 stages of complete investigation

1. Reconnaissance
   - Geologic map / report / air photograph / field visit
2. Site Selection
   - Test pit / outcrops mapping / geophysics / index test / limited diamond drilling at alternative sites
3. Preliminary Site Investigation
   - Diamond drilling of selected site / detailed mapping of outcrops & exploration adits / lab testing
4. Detailed Investigation
   - Drilling of selected geological feature / in-situ testing / lab testing

Quantitative Description of Discontinuities in Rock Masses (ISRM)



A - Rock type
B - Rock strength
C - Weathering
D - Discontinuity description
E - Discontinuity orientation
F - Roughness
G - Aperture
H - Infilling type and width
I - Spacing
J - Persistence
K - Number of sets
L - Block size and shape
M - Seepage

A-Rock type

Three primary characteristics of rock

1. **Color**, as well as whether light or dark minerals predominate
2. **Texture** or fabric ranging from crystalline, granular or glassy
3. **Grain size** that can range from clay particles to gravel

A-Rock type

Table II.1 Rock type classification

Genetic Group			Detrital Sedimentary			Pyroclastic	Chemical Organic	Metamorphic		Igneous			
Usual Structure			BEDDED			BEDDED		FOLIATED	MASSIVE	MASSIVE			
COMPOSITION										Light coloured minerals are quartz, feldspar, mica and feldspar-like minerals			Dark minerals
	Grain size (mm)		Grains of rock, quartz, feldspar and minerals	At least 50% of grains are of carbonate		At least 50% of grains are of fine-grained volcanic rock		Quartz, feldspars, micas, acicular dark minerals		Acid rocks	Intermediate rocks	Basic rocks	Ultra basic rocks
Very coarse grained	60	RUDACEOUS	Grains are of rock fragments. Rounded grains: CONGLOMERATE	LIMESTONE (undifferentiated)	CALCIRUDITE	Rounded grains AGGLOMERATE	SALINE ROCKS Halite Anhydrite Gypsum	MIGMATITE	HORNFELS	PEGMATITE			PYROXENITE and PERIDOTITE
Coarse grained	2		Angular grains: BRECCIA			Angular grains VOLCANIC BRECCIA		GNEISS Alternate layers of granular and flakey minerals	MARBLE GRANULITE	GRANITE	DIORITE	GABBRO	
Medium grained		ARENACEOUS	SANDSTONE. Grains are mainly mineral fragments QUARTZ SANDSTONE: 95% quartz, voids empty or cemented ARKOSE: 75% quartz, up to 25% feldspar: voids empty of cemented ARGILLACEOUS SANDSTONE: 75% quartz, 15% + fine detrital material		CALCARENITE	TUFF	CHERT	SCHIST PHYLLITE SLATE	QUARTZITE AMPHIBOLITE	MICRO-GRANITE	MICRO-DIORITE	DOLERITE	
	0.06						FLINT	MYLONITE					SERPENTINE
Fine grained	0.002	ARGILLACEOUS or LUTACEOUS	MUDSTONE SHALE: fissile mudstone SILTSONE: 50% fine grained particles CLAYSTONE 50% very fine grained particles CALCAREOUS MUDSTONE		CALCISILTITE	Fine-grained TUFF	COAL OTHERS			RHYOLITE	ANDESITE	BASALT	
Very fine grained					CALCILUTITE	Very fine-grained TUFF							
GLASSY										OBSIDIAN and PITCHSTONE		TACHYLYTE	

Note: Numbers can be used to identify rock types on data sheets (see Appendix III)
Reference: *Geological Society Engineering Group Working Party (1977)*

A-Rock type

Table II.2 Grain size scale

Description	*Grain size*
Boulders	200–600 mm (7.9–23.6 in)
Cobbles	60–200 mm (2.4–7.9 in)
Coarse gravel	20–60 mm (0.8–0.24 in)
Medium gravel	6–20 mm (0.2–0.8 in)
Fine gravel	2–6 mm (0.1–0.2 in)
Coarse sand	0.6–2 mm (0.02–0.1 in)
Medium sand	0.2–0.6 mm (0.008–0.02 in)
Fine sand	0.06–0.2 mm (0.002–0.008 in)
Silt, clay	<0.06 mm (<0.002 in)

B-Rock Strength

Table II.3 Classification of rock material strengths

Grade	*Description*	*Field identification*	*Approximate compressive (MPa)*	*Range of strength (psi)*
R6	Extremely strong rock	Specimen can only be chipped with geological hammer.	>250	>36,000
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100–250	15,000–36,000
R4	Strong rock	Specimen requires more than one blow with a geological hammer to fracture it.	50–100	7000–15,000
R3	Medium weak rock	Cannot be scraped or peeled with a pocket knife; specimen can be fractured with single firm blow of geological hammer.	25–50	3500–7000
R2	Weak rock	Can be peeled with a pocket knife; shallow indentations made by firm blow with point of geological hammer.	5–25	725–3500
R1	Very weak rock	Crumbles under firm blows with point of geological hammer; can be peeled by a pocket knife.	1–5	150–725
R0	Extremely weak rock	Indented by thumbnail.	0.25–1	35–150
S6	Hard clay	Indented with difficulty by thumbnail.	>0.5	>70
S5	Very stiff clay	Readily indented by thumbnail.	0.25–0.5	35–70
S4	Stiff clay	Readily indented by thumb but penetrated only with great difficulty.	0.1–0.25	15–35
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort.	0.05–0.1	7–15
S2	Soft clay	Easily penetrated several inches by thumb.	0.025–0.05	4–7
S1	Very soft clay	Easily penetrated several inches by fist.	<0.025	<4

C-Weathering

Table II.4 Weathering and alteration grades

Grade	*Term*	*Description*
I	Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.
II	Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
III	Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
IV	Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
V	Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
VI	Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

D-Discontinuity description

Type of Discontinuity

Fault – discontinuity along which there has been and observable amount of displacement

Bedding – surface parallel to the surface of deposition

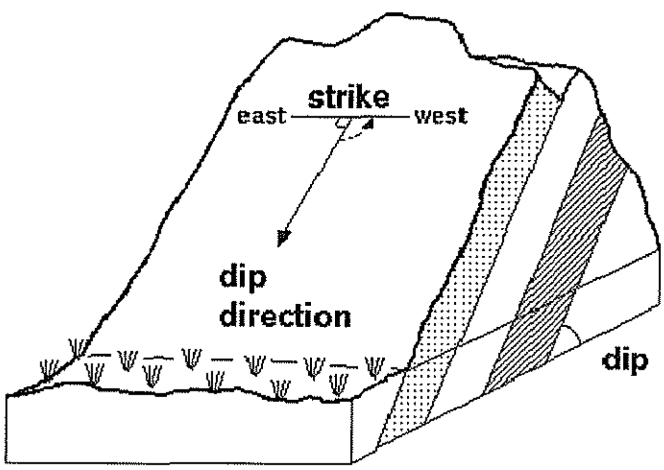
Foliation – parallel orientation of platy minerals, or mineral banding in metamorphic rocks

Joint – discontinuity in which there has been no observable relative moment

Cleavage – parallel discontinuities formed incompetent layers in a series of beds of varying degrees of competency

Schistosity – foliation in schist or other coarse grained crystalline rock

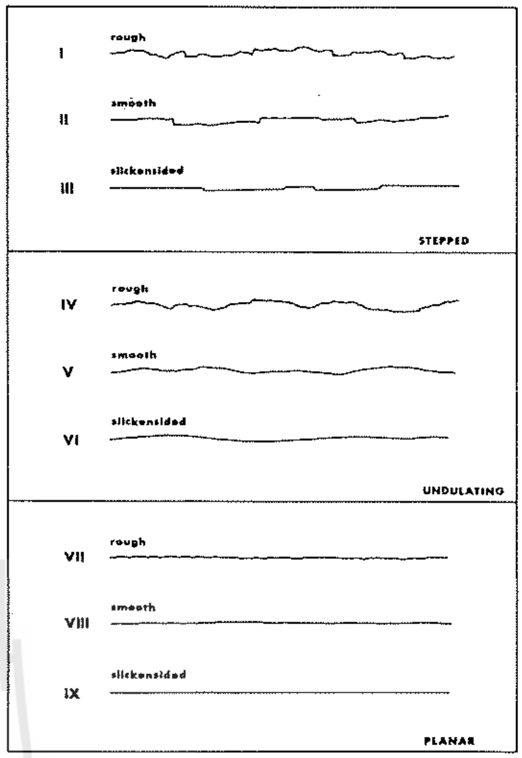
E-Discontinuity orientation



F-Roughness

Table II.5 Descriptive terms for roughness

I	Rough, stepped
II	Smooth, stepped
III	Slickensided, stepped
IV	Rough, undulating
V	Smooth, undulating
VI	Slickensided, undulating
VII	Rough, planar
VIII	Smooth, planar
IX	Slickensided, planar



F-Roughness

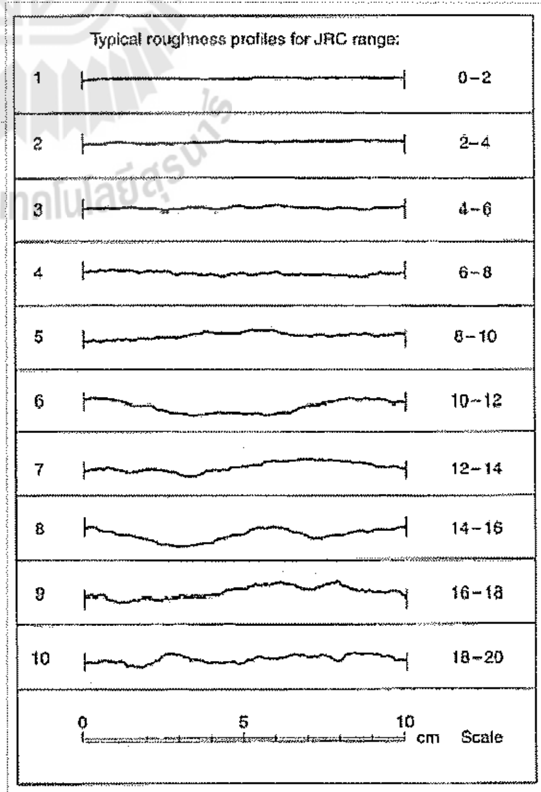
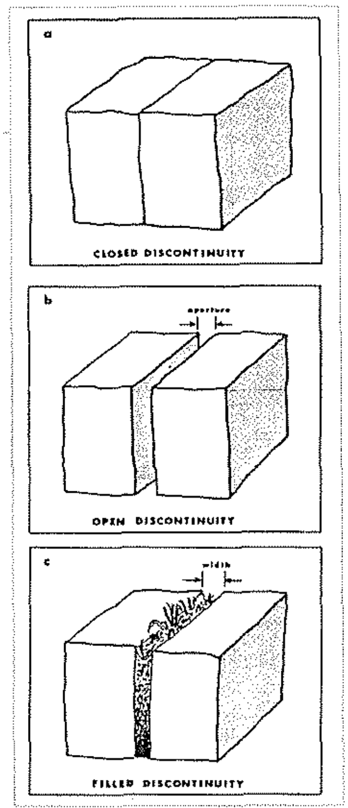


Figure II.3 Roughness profiles and corresponding range of JRC (joint roughness coefficient) values (ISRM, 1981a).

G-Aperture

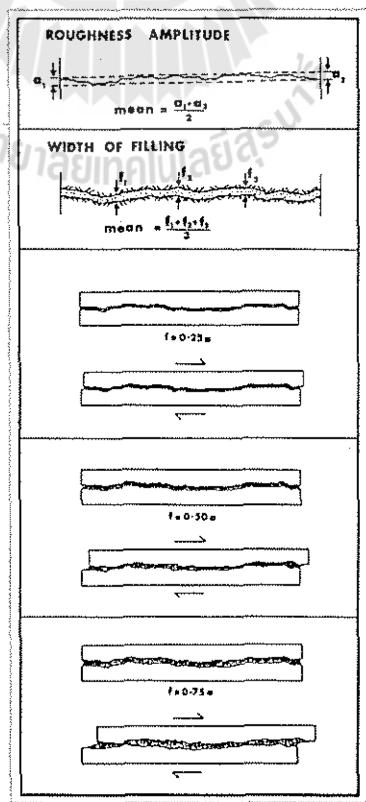
Table II.6 Aperture dimensions

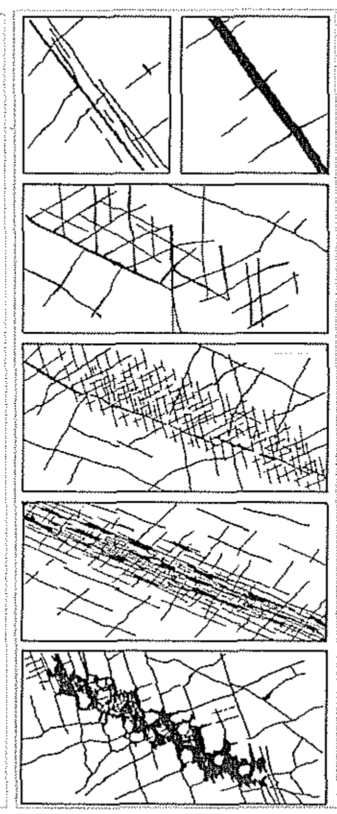
Aperture	*Description*	
<0.1 mm	Very tight	
0.1–0.25 mm	Tight	"Closed" features
0.25–0.5 mm	Partly open	
0.5–2.5 mm	Open	
2.5–10 mm	Moderately wide	"Gapped" features
>10 mm	Wide	
1–10 cm	Very wide	
10–100 cm	Extremely wide	"Open" features
>1 m	Cavernous	



H-Infilling type and width

- •Width
- •Weathering Grade
- •Mineralogy
- •Particle Size
- •Filling Strength
- •Previous Displacement
- •Water Content and Permeability





I-Spacing

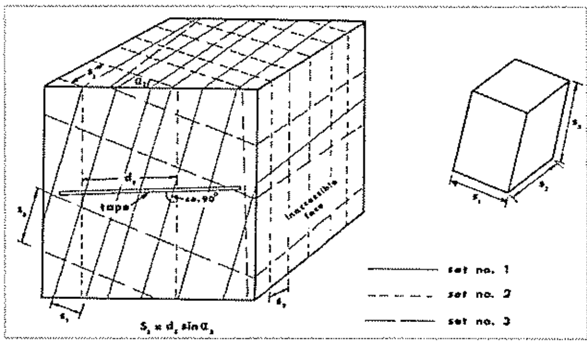


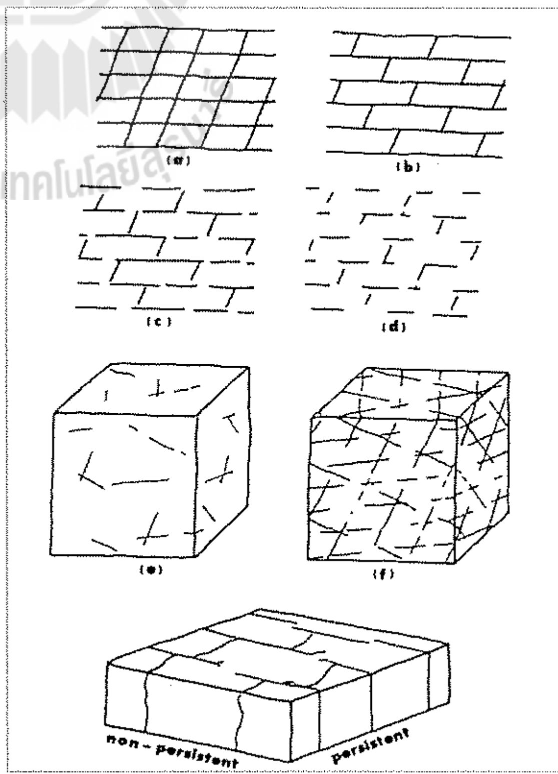
Table II.7 Spacing dimensions

Description	*Spacing (mm)*
Extremely close spacing	<20
Very close spacing	20–60
Close spacing	60–200
Moderate spacing	200–600
Wide spacing	600–2000
Very wide spacing	2000–6000
Extremely wide spacing	>6000

J-Persistence

Table II.8 Persistence dimensions

Very low persistence	<1 m
Low persistence	1–3 m
Medium persistence	3–10 m
High persistence	10–20 m
Very high persistence	>20 m



K-Number of sets

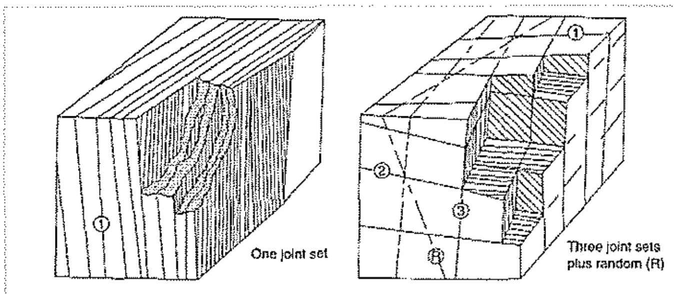


Figure II.4 Examples illustrating the effect of the number of joint sets on the mechanical behavior and appearance of rock masses (ISRM, 1981a).

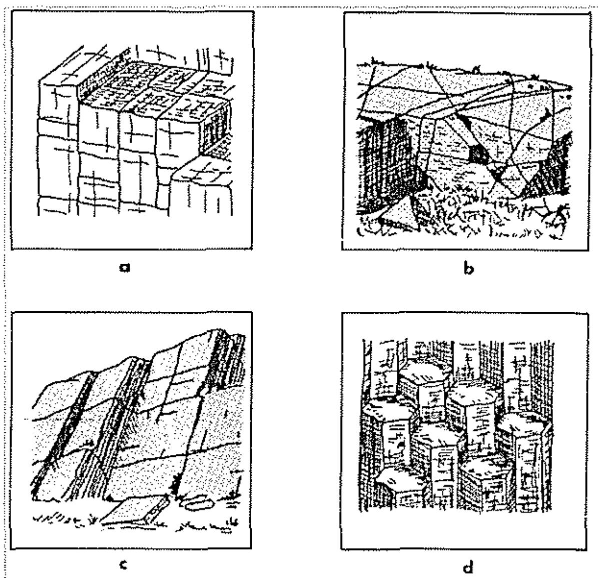
I	massive, occasional random joints
II	one joint set
III	one joint set plus random
IV	two joint sets
V	two joint sets plus random
VI	three joint sets
VII	three joint sets plus random
VIII	four or more joint sets
IX	crushed rock, earth-like

L-Block size and shape

Table II.9 Block dimensions

Description	J_v *(joints/m^3)*
Very large blocks	<1.0
Large blocks	1–3
Medium-sized blocks	3–10
Small blocks	10–30
Very small blocks	>30

(i)	*massive*	= few joints or very wide spacing
(ii)	*blocky*	= approximately equidimensional
(iii)	*tabular*	= one dimension considerably smaller than the other two
(iv)	*columnar*	= one dimension considerably larger than the other two
(v)	*irregular*	= wide variations of block size and shape
(vi)	*crushed*	= heavily jointed to "sugar cube"



M- Seepage

Table II.10 Seepage quantities in unfilled discontinuities

Seepage rating	*Description*
I	The discontinuity is very tight and dry, water flow along it does not appear possible.
II	The discontinuity is dry with no evidence of water flow.
III	The discontinuity flow is dry but shows evidence of water flow, that is, rust staining.
IV	The discontinuity is damp but no free water is present.
V	The discontinuity shows seepage, occasional drops of water, but no continuous flow.
VI	The discontinuity shows a continuous flow of water—estimate l/ min and describe pressure, that is, low, medium, high.

M- Seepage

Table II.11 Seepage quantities in filled discontinuities

Seepage rating	*Description*
I	The filling materials are heavily consolidated and dry, significant flow appears unlikely due to very low permeability.
II	The filling materials are damp, but no free water is present.
III	The filling materials are wet, occasional drops of water.
IV	The filling materials show signs of outwash, continuous flow of water—estimate l/ min.
V	The filling materials are washed out locally, considerable water flow along out-wash channels—estimate l/ min and describe pressure that is low, medium, high.
VI	The filling materials are washed out completely, very high water pressures experienced, especially on first exposure—estimate l/ min and describe pressure.

Rock Mass Classifications

Classification system	Form and type*	Main applications	Reference
Terzaghi rock load classification system	Descriptive and behaviouristic form Functional type.	Design of steel support in tunnels	Terzaghi, 1946
Lauffer's stand-up time classification	Descriptive form General type	Tunnelling design	Laufer H., 1958
New Australian tunneling method (NATM)	Descriptive and behaviouristic form Tunneling concept	Excavation and design in incompetent (overstressed) ground	Rabcewicz, Müller and Pacher, 1958–1964
Rock classification for rock mechanical purposes	Descriptive form General type	Input in rock mechanics	Patching and Coates, 1968
Unified classification of soils and rocks	Descriptive form General type	Based on particles and blocks for communication	Deer et al., 1969
Rock quality designation (RQD)	Numerical form General type	Based on core logging; used in other classification systems	Deer et al., 1967
Size-strength classification	Numerical form Functional type	Based on rock strength and block diameter, used mainly in mining	Franklin, 1975
Rock structure rating classification (RSR)	Numerical form Functional type	Design of (steel) support in tunnels	Wickham et al., 1972
Rock mass rating classification (RMR)	Numerical form Functional type	Design of tunnels, mines, and foundations	Bieniawski, 1973
Q-classification system	Numerical form Functional type	Design of support in underground excavation	Barton et al., 1974
Typological classification	Descriptive form General type	Use in communication	Matula and Holzer, 1978
Unified rock classification system	Descriptive form General type	Use in communication	Williamson, 1980
Basic geotechnical classification (BGD)	Descriptive form General type	General applications	ISRM, 1981
Geological strength index (GSI)	Numerical form Functional type	Design of support in underground excavation	Hoek, 1994
Rock mass index system (RMi)	Numerical form Functional type	General characterization, design of support, TMB progress	Palmström, 1995

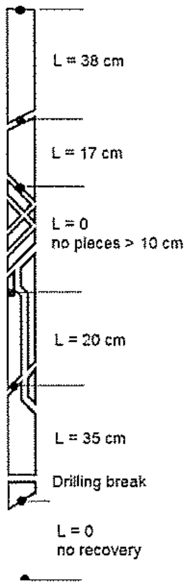
Rock Mass Classifications

Deer's Rock Quality Destination (RQD)

- Deere (1964) proposed a quantitative index of rock mass quality based upon core recovery by diamond drilling.
- RQD has come to be very widely used and has been shown to be particularly useful in classifying rock masses for the selection of tunnel support systems.
- RQD is defined as the percentage of intact core pieces longer than 100 mm (4 inches) in the total length of core.

Rock Mass Classifications

Deer's Rock Quality Destination (RQD)



Total length of core run = 200 cms

$$RQD = \frac{\Sigma \text{ Length of core pieces} > 10 \text{ cm length}}{\text{Total length of core run}} \times 100$$

$$RQD = \frac{38 + 17 + 20 +}{200} \times 100 = 55\ \%$$

Rock Mass Classifications

RQD Estimation from outcrop

- Palmström (1982) suggested that, when no core is available but discontinuity traces are visible in surface exposures or exploration adits, the *RQD may be estimated from the* number of discontinuities per unit volume. The suggested relationship for clay-free rock masses is:

$$RQD = 115 - 3.3\,J_v \qquad (J_v < 4.5)$$

$$RQD = 100 \exp(-0.1/S)\,(1 + 0.1/S)$$

- where J_v *is the sum of the number of joints per unit length for all joint (discontinuity)* sets known as the volumetric joint count and S is average spacing of joint.

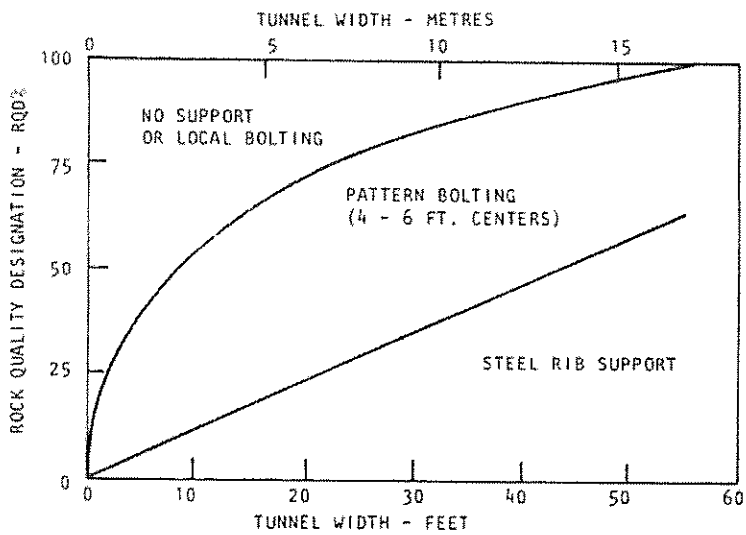
Rock Mass Classifications

Deer's Rock Quality Destination (RQD)

RQD	Rock Quality
< 25%	Very poor
25 –50 %	poor
50 – 75%	Fair
75 – 90%	Good
90 – 100%	Very good

Rock Mass Classifications

Deer's Rock Quality Destination (RQD)



Merritt, 1972

Rock Mass Classifications

Geomechanics Classification (RMR)

- Bieniawski (1976) published the details of a rock mass classification called the Geomechanics Classification or the Rock Mass Rating (*RMR) system.*
- The following six parameters are used to classify a rock mass using the *RMR system:*
  1. Uniaxial compressive strength of rock material.
  2. Rock Quality Designation (*RQD).*
  3. Spacing of discontinuities.
  4. Condition of discontinuities.
  5. Groundwater conditions.
  6. Orientation of discontinuities.

Geomechanics Classification (RMR)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Groundwater	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0 1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

(After Bieniawski 1989).

Geomechanics Classification (RMR)

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS						
Rating		100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number		I	II	III	IV	V
Description		Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
D. MEANING OF ROCK CLASSES						
Class number		I	II	III	IV	V
Average stand-up time		20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)		> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)		> 45	35 - 45	25 - 35	15 - 25	< 15

Geomechanics Classification (RMR)

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions					
Discontinuity length (persistence) Rating	< 1 m 6	1 - 3 m 4	3 - 10 m 2	10 - 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 - 1.0 mm 4	1 - 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**			
Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

Geomechanics Classification (RMR)

- The *RMR value for the example under consideration is determined as follows:*

Table	*Item*	*Value*	*Rating*
A.1	Point load index	8 MPa	12
A.2	*RQD*	70%	13
A.3	Spacing of discontinuities	300 mm	10
E.4	Condition of discontinuities	Note 1	22
A.5	Groundwater	Wet	7
B	Adjustment for joint orientation	Note 2	-5
		Total	59

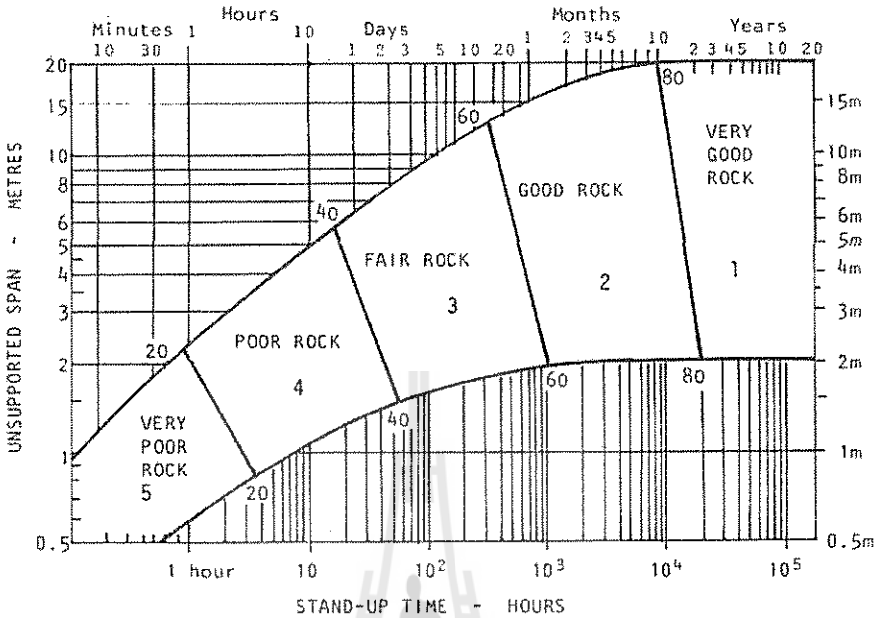
Geomechanics Classification (RMR)

- Guidelines for excavation and support of 10 m span rock tunnels in accordance with the *RMR system (After Bieniawski 1989).*

Rock mass class	Excavation	Rock bolts (20 mm diameter, fully grouted)	Shotcrete	Steel sets
I - Very good rock *RMR*: 81-100	Full face. 3 m advance.	Generally no support required except spot bolting.		
II - Good rock *RMR*: 61-80	Full face , 1-1.5 m advance. Complete support 20 m from face.	Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh.	50 mm in crown where required.	None.
III - Fair rock *RMR*: 41-60	Top heading and bench 1.5-3 m advance in top heading. Commence support after each blast. Complete support 10 m from face.	Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown.	50-100 mm in crown and 30 mm in sides.	None.
IV - Poor rock *RMR*: 21-40	Top heading and bench 1.0-1.5 m advance in top heading. Install support concurrently with excavation, 10 m from face.	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh.	100-150 mm in crown and 100 mm in sides.	Light to medium ribs spaced 1.5 m where required.
V – Very poor rock *RMR*: < 20	Multiple drifts 0.5-1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting.	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert.	150-200 mm in crown, 150 mm in sides, and 50 mm on face.	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert.

Geomechanics Classification (RMR)

- **By Bieniawski (1976)**



Rock Mass Classifications

Rock Tunnelling Quality Index, Q (NGI)

- On the basis of an evaluation of a large number of case histories of underground excavations, Barton et al (1974) of the Norwegian Geotechnical Institute proposed a Tunnelling Quality Index (*Q*) *for the determination of rock mass characteristics and* tunnel support requirements.
- The numerical value of the index *Q varies on a logarithmic* scale from 0.001 to a maximum of 1,000 and is defined by:

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

Rock Tunnelling Quality Index, Q (NGI)

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF}$$

where *RQD* *is the Rock Quality Designation*
J_n *is the joint set number*
J_r *is the joint roughness number*
J_a *is the joint alteration number*
J_w *is the joint water reduction factor*
SRF *is the stress reduction factor*

Rock Tunnelling Quality Index, Q (NGI)

- It appears that the rock tunnelling quality *Q can now be considered to be a function of* only three parameters which are crude measures of:
  1. Block size (RQD/J_n)
  2. Inter-block shear strength (J_r/J_a)
  3. Active stress (J_w/SRF)

Rock Tunnelling Quality Index, Q (NGI)

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	*RQD*	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as ≤ 10 (including 0),
B. Poor	25 - 50	a nominal value of 10 is used to evaluate Q.
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently
E. Excellent	90 - 100	accurate.

DESCRIPTION	VALUE	NOTES
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed, 'sugar cube', etc.	15	2. For portals use ($2.0 \times J_n$)
J. Crushed rock. earthlike	20	

Rock Tunnelling Quality Index, Q (NGI)

DESCRIPTION	VALUE	NOTES
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is
E. Rough or irregular, planar	1.5	greater than 3 m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for
H. Zones containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	minimum strength.
J. Sandy, gravely or crushed zone thick enough to prevent rock wall contact	1.0 (nominal)	

Rock Tunnelling Quality Index, Q (NGI)

4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
B. Unaltered joint walls, surface staining only	1.0	25 - 35	
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25	
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16	

Rock Tunnelling Quality Index, Q (NGI)

4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)
b. Rock wall contact before 10 cm shear		
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12
c. No rock wall contact when sheared		
K. Zones or bands of disintegrated or crushed	6.0	
L. rock and clay (see G, H and J for clay	8.0	
M. conditions)	8.0 - 12.0	6 - 24
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0	
O. Thick continuous zones or bands of clay	10.0 - 13.0	
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0	

Rock Tunnelling Quality Index, Q (NGI)

5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	

Rock Tunnelling Quality Index, Q (NGI)

6. STRESS REDUCTION FACTOR *a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated*	*SRF*	
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)	10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)	5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)	2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)	5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)	2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)	5.0	

Rock Tunnelling Quality Index, Q (NGI)

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	σ_t/σ_1		2. For strongly anisotropic virgin stress field (if measured): when $5 \le \sigma_1/\sigma_3 \le 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

Rock Tunnelling Quality Index, Q (NGI)

- Barton et al (1974) defined an additional parameter which they called the Equivalent Dimension, De, of the excavation.
- This dimension is obtained by dividing the span, diameter or wall height of the excavation by a quantity called the Excavation Support Ratio, ESR. Hence:

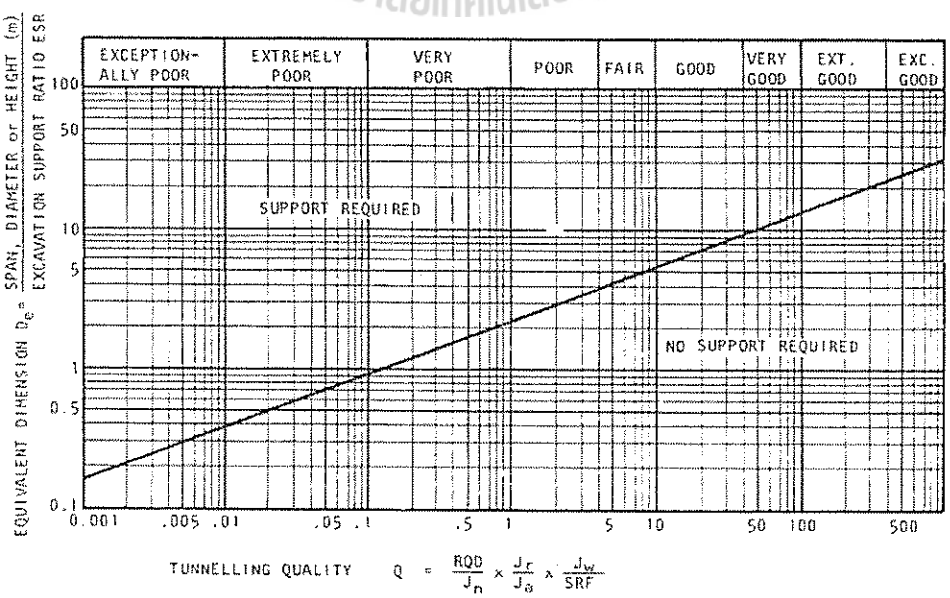
$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio } ESR}$$

Rock Tunnelling Quality Index, Q (NGI)

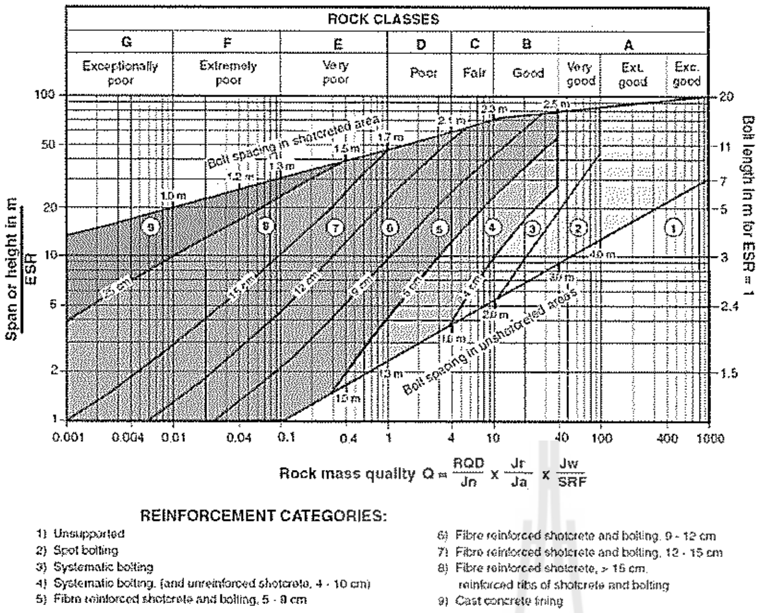
- The value of *ESR is related to the intended use of the excavation and to the degree of* security which is demanded of the support system installed to maintain the stability of the excavation.
- Barton et al (1974) suggest the following values:

Excavation category		*ESR*
A	Temporary mine openings.	3-5
B	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), pilot tunnels, drifts and headings for large excavations.	1.6
C	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D	Power stations, major road and railway tunnels, civil defence chambers, portal intersections.	1.0
E	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

Rock Tunnelling Quality Index, Q (NGI)



Estimated support categories



Estimated support categories based on the tunnelling quality index *Q (After* Grimstad and Barton, 1993, reproduced from Palmstrom and Broch, 2006).

Example

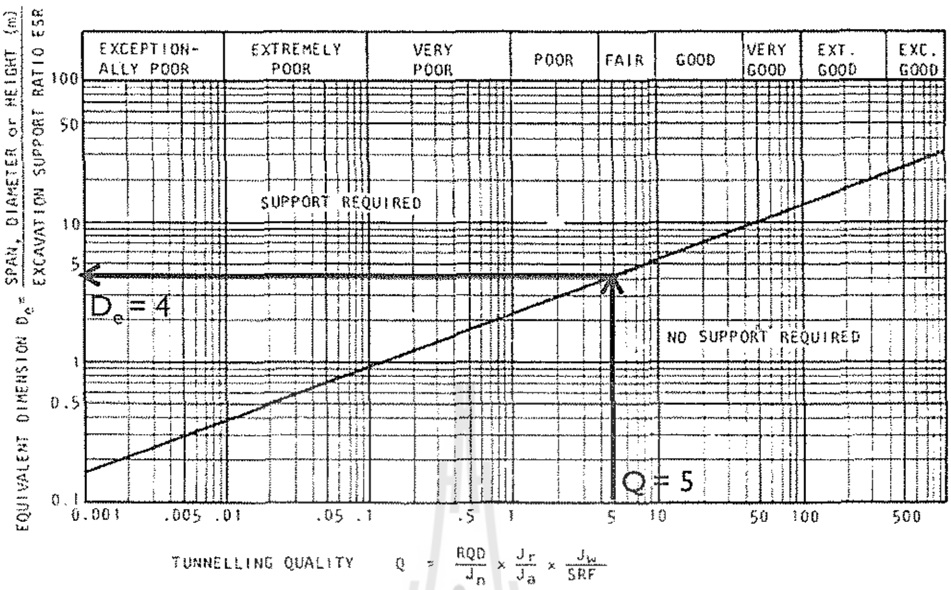
Item	Description	Value
1. Rock Quality	**Good**	**RQD = 80%**
2. Joint sets	**Two sets**	**Jn = 4**
3. Joint roughness	**Rough**	**Jr = 3**
4. Joint alteration	**Clay gouge**	**Ja = 4**
5. Joint water	**Large inflow**	**Jw = 0.33**
6. Stress reduction	**Medium stress**	**SRF = 1.0**

$$Q = \frac{80}{4} \times \frac{3}{4} \times \frac{0.33}{1} = 5$$

From the Figure 3.7, the maximum equivalent dimension D_e = 4 meters.

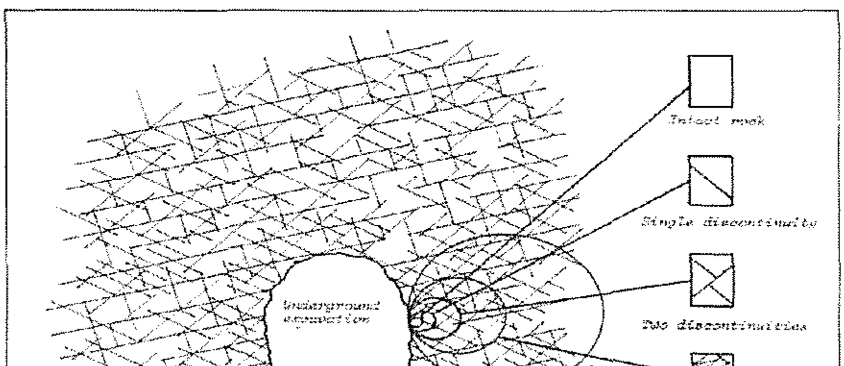
A permanent underground mine opening has an excavation support ratio ESR of 1.6 and, hence the maximum unsupported span which can be considered for this crusher station is ESR × De = 1.6 × 4 = 6.4 meters.

Rock Tunnelling Quality Index, Q (NGI)

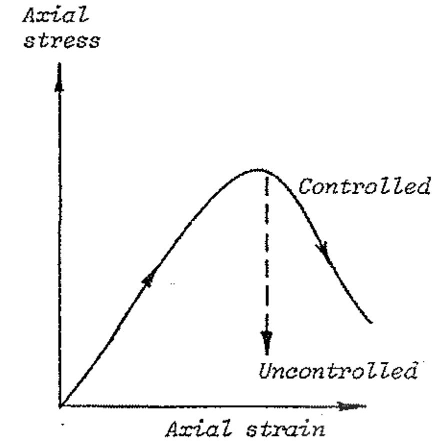


Strength of Rock and Rock Mass

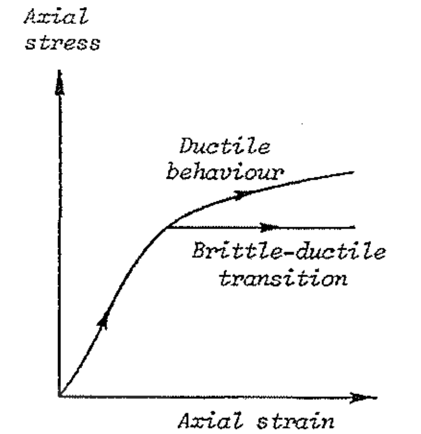
The transition from intact rock material to a heavily jointed rock mass



Brittle and Ductile Behavior

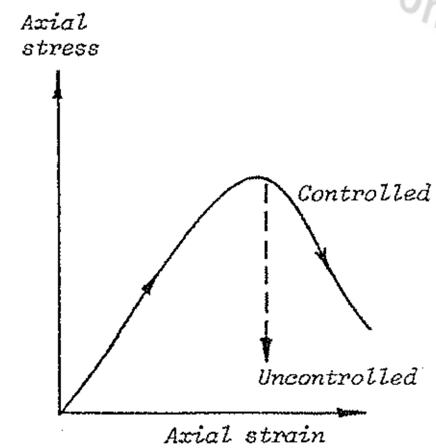


Stress-strain curves for brittle fracture in uniaxial compression



Stress-strain curves for ductile behaviour in compression

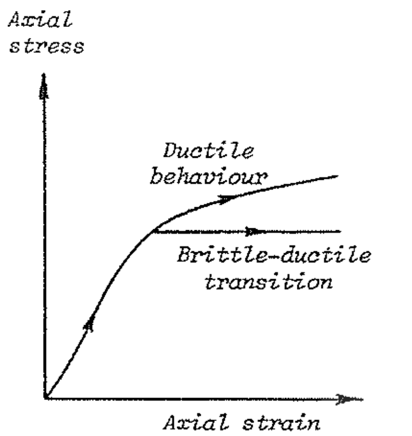
Brittle and Ductile Behavior



Stress-strain curves for brittle fracture in uniaxial compression

Brittle failure occurs when the ability of the rock to resist load decreases with increasing deformation. Brittle failure is often associated with little or no permanent deformation before failure and, depending upon the test conditions, may occur suddenly and catastrophically. Rock bursts in deep hard rock mines provide graphic illustrations of the phenomenon of explosive brittle fracture.

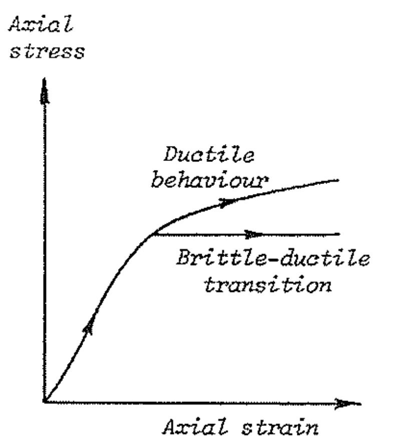
Brittle and Ductile Behavior



Stress-strain curves for ductile behaviour in compression

A material is said to be ***ductile*** when it can sustain permanent deformation without losing its ability to resist load. Most rocks will behave in a brittle rather than a ductile manner at the confining pressures and temperatures encountered in civil and mining engineering applications. *Ductility increases with increased confining pressure and temperature, but can also occur in weathered rocks,* heavily jointed rock masses and some weak rocks such as evaporites under normal engineering conditions.

Brittle and Ductile Behavior

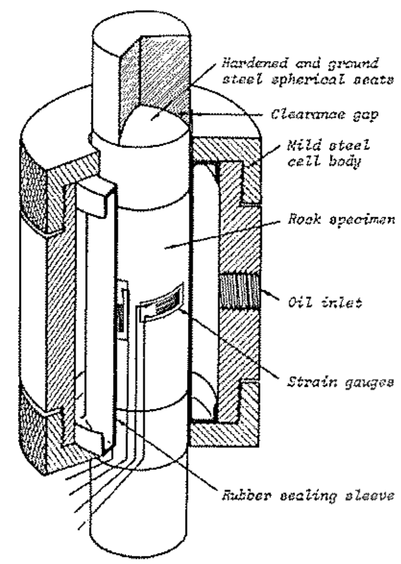


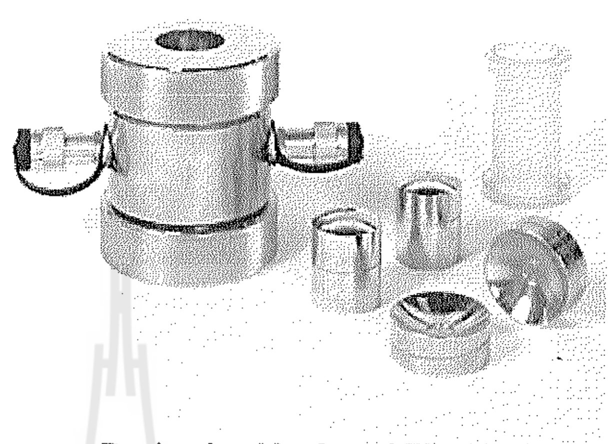
Stress-strain curves for ductile behaviour in compression

As the confining pressure is increased it will reach the ***brittle-ductile transition*** value at which there is a transition from typically brittle to fully ductile behaviour. The brittle-ductile transition pressure as the confining pressure is which the stress required to form a failure plane in a rock specimen is equal to the stress required to cause sliding on that plane.

Laboratory Testing of Intact Rock Specimens

- Uniaxial and triaxial compression tests

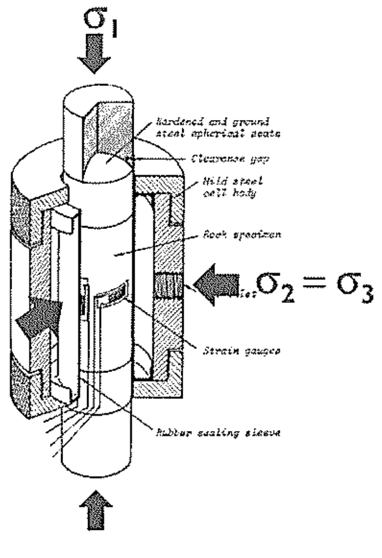




Design by Hoek and Flankin (1964)

Laboratory Testing of Intact Rock Specimens

- Uniaxial and triaxial compression tests



Uniaxial compression

$\sigma_2 = \sigma_3 = 0$

$\sigma_1 = \sigma_c$ (uniaxial compressive strength)

Triaxial compression

$\sigma_2 = \sigma_3 = p$ (confining pressure)

An Empirical Failure Criterion of Rock

- Hoek and Brown (1980) have drawn on their experience in both theoretical and experimental aspects of rock behaviour to develop, by a process of trial and error, the following empirical relationship between the principal stresses associated with the failure of rock :

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_3\sigma_3 + s\sigma_c^2}$$ (The original Hoek and Brown Failure Criterion)

where

m = constant depending on the characteristics of the rock mass,
s = constant depending on the characteristics of the rock mass,
σ_c = uniaxial compressive strength of the intact rock material,
σ_1 = major principal stress at failure, and
σ_3 = minor principal stress at failure.

Hoek and Brown Failure Criterion

- The uniaxial compressive strength of the specimen is given by substitution $\sigma_3 = 0$

$$\sigma_{c,\text{rockmass}} = \sigma_c\sqrt{s}$$ (Compressive Strength of Rock Mass)

- For intact rock, $\sigma_{c,\text{ rockmass}} = \sigma_c$ and s = 1.
- For previously broken rock, s < 1 and the strength at zero confining pressure, where σ_c is the uniaxial compressive strength of the pieces of *intact* rock material making up the specimen.

Hoek and Brown Failure Criterion

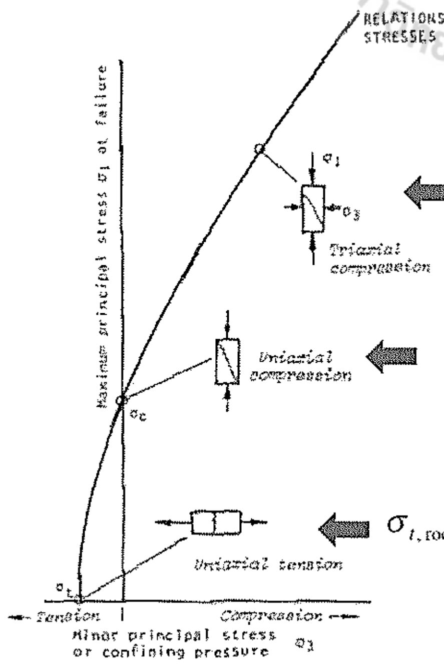
- The uniaxial tensile strength of the specimen is given by substitution of $\sigma_1 = 0$ in equation $\sigma_1 = \sigma_3 + \sqrt{m\sigma_3\sigma_3 + s\sigma_c^2}$ and by solving the resulting quadratic equation for σ_3

$$\sigma_{t,\text{rockmass}} = \frac{\sigma_c}{2}\left(m - \sqrt{m^2 + 4s}\right)$$ (Tensile Strength of Rock Mass)

a **quadratic equation** is a polynomial equation of the second degree.

$$ax^2 + bx + c = 0, \quad x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

Hoek and Brown Failure Criterion

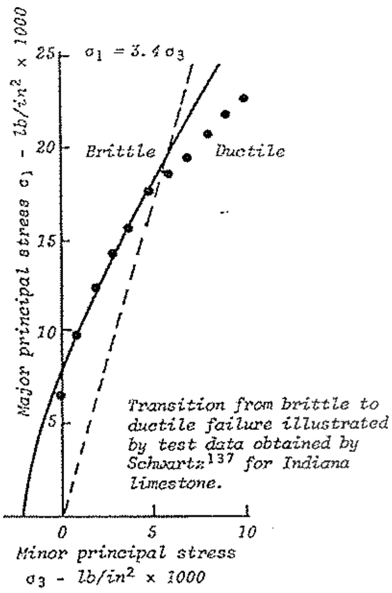


$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_3\sigma_3 + s\sigma_c^2}$$

$$\sigma_{c,\text{rockmass}} = \sigma_c\sqrt{s}$$

$$\sigma_{t,\text{rockmass}} = \frac{\sigma_c}{2}\left(m - \sqrt{m^2 + 4s}\right)$$

Brittle-ductile transition



Transition from brittle to ductile failure illustrated by test data obtained by Schwartz[137] for Indiana limestone.

- Mogi (1966) investigated the behavior of most rocks changes from brittle to ductile at high confining pressure.
- For most rock, the transition pressure is defined by :

$$\sigma_1 = 3.4\,\sigma_3$$

Determination of Strength Parameters

Part I (for Intact Rock) (Data from triaxial lab test)

The empirical criterion given by $\sigma_1 = \sigma_3 + \sqrt{m\sigma_c \cdot \sigma_3 + s\sigma_c^2}$

may be rewritten as: $y = m\sigma_c \cdot x + s\sigma_c^2$

where $y = (\sigma_1 - \sigma_3)^2$ and $x = \sigma_3$

For intact rock, s = 1 and the uniaxial compressive strength σ_c and the material constant m are given by :

$$\sigma_c^2 = \frac{\Sigma y_i}{n} - \left[\frac{\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}}{\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}}\right]\frac{\Sigma x_i}{n}$$

$$m = \frac{1}{\sigma_c}\left[\frac{\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}}{\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}}\right] \rightarrow (m_i)$$

where x_i and y_i are successive data pairs and n is the total number of such data pairs.

Determination of Strength Parameters

The coefficient of determination r^2 is given by :

$$r^2 = \frac{\left[\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}\right]^2}{\left[\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}\right]\left[\Sigma y_i^2 - \frac{(\Sigma y_i)^2}{n}\right]}$$

The closer the value of r^2 is to 1.00, the better the fit of the empirical equation to the triaxial test data.

Determination of Strength Parameters

Part I (for Intact Rock)

1. Enter triaxial data in the form $x = \sigma_3$, $y = (\sigma_1 - \sigma_3)^2$
2. Calculate and accumulate :
   Σx_i, Σx_i^2, Σy_i, Σy_i^2 and $\Sigma x_i y_i$,
3. Calculate σ_c from equation A.2,
4. Calculate m from equation A.3,
5. Calculate r^2 from equation A.4,
6. Note that $s = 1$ for intact rock.

$$\sigma_c^2 = \frac{\Sigma y_i}{n} - \left[\frac{\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}}{\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}}\right]\frac{\Sigma x_i}{n} \qquad \text{(A.2)}$$

$$m = \frac{1}{\sigma_c}\left[\frac{\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}}{\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}}\right] \qquad \text{(A.3)}$$

$$r^2 = \frac{\left[\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}\right]^2}{\left[\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}\right]\left[\Sigma y_i^2 - \frac{(\Sigma y_i)^2}{n}\right]} \qquad \text{(A.4)}$$

Determination of Strength Parameters

Part II (for Broken or heavily jointed rock)

$$m = \frac{1}{\sigma_c}\left[\frac{\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}}{\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}}\right]$$

$$s = \frac{1}{\sigma_c^2}\left[\frac{\Sigma y_i}{n} - m\sigma_c \frac{\Sigma x_i}{n}\right]$$

When the value of the constant s is very close to zero, sometimes give a negative value.
In such a case, put s = 0 and calculate m as follows :

$$m = \frac{\Sigma y_i}{\sigma_c \Sigma x_i}$$

$$r^2 = \frac{\left[\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}\right]^2}{\left[\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}\right]\left[\Sigma y_i^2 - \frac{(\Sigma y_i)^2}{n}\right]}$$

Determination of Strength Parameters

Part II (for Broken or heavily jointed rock) (Data from rock mass test)

1. Enter value of σ_c for intact rock,
2. Enter triaxial data in the form $x = \sigma_3$, $y = (\sigma_1 - \sigma_3)^2$,
3. Calculate and accumulate:
   Σx_i, Σx_i^2, Σy_i, Σy_i^2 and $\Sigma x_i y_i$,
4. Calculate m from equation A.3,
5. Calculate s from equation A.5,
6. Calculate r^2 from equation A.4,
7. When $s < 0$ in step 5, put s = 0 and calculate m from equation A.6,
8. Note that equation A.4 is not valid when $s < 0$.

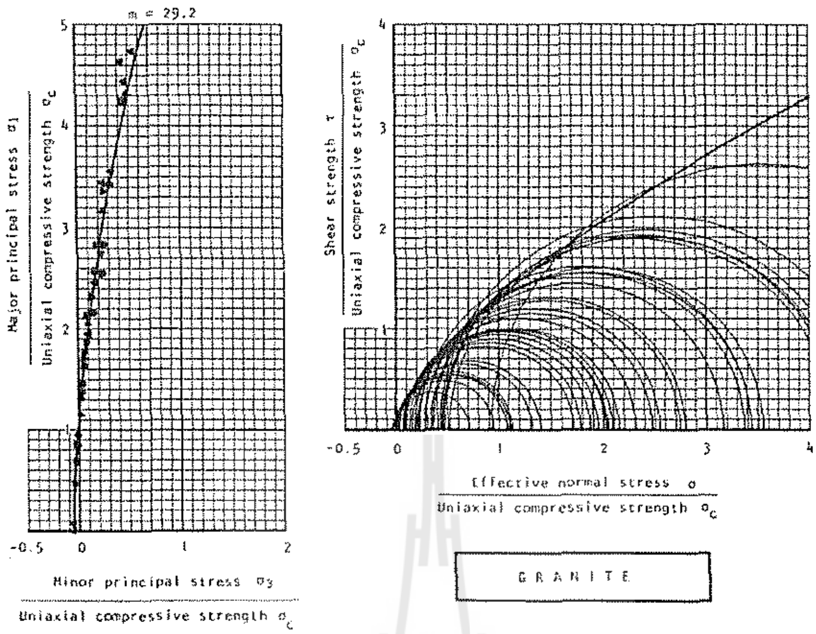
$$m = \frac{1}{\sigma_c}\left[\frac{\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}}{\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}}\right] \quad \text{(A.3)}$$

$$s = \frac{1}{\sigma_c^2}\left[\frac{\Sigma y_i}{n} - m\sigma_c \frac{\Sigma x_i}{n}\right] \quad \text{(A.5)}$$

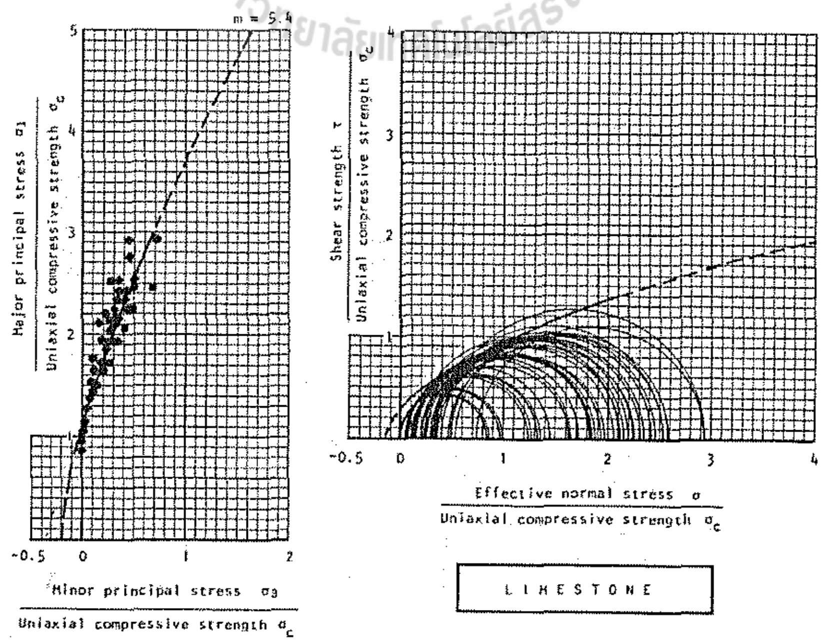
$$r^2 = \frac{\left[\Sigma x_i y_i - \frac{\Sigma x_i \Sigma y_i}{n}\right]^2}{\left[\Sigma x_i^2 - \frac{(\Sigma x_i)^2}{n}\right]\left[\Sigma y_i^2 - \frac{(\Sigma y_i)^2}{n}\right]} \quad \text{(A.4)}$$

$$m = \frac{\Sigma y_i}{\sigma_c \Sigma x_i} \quad \text{(A.6)}$$

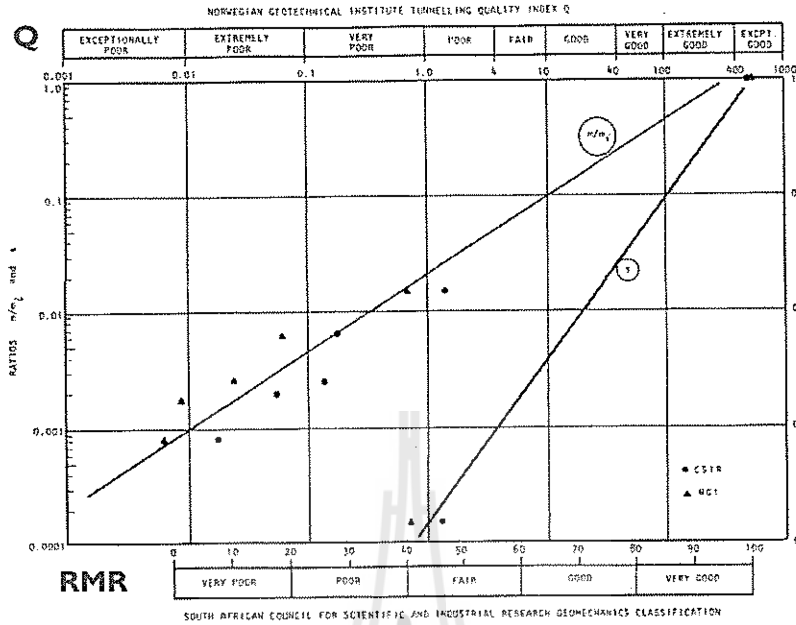
Survey of triaxial test data on intact rock specimens



Survey of triaxial test data on intact rock specimens



Use rock mass classification for strength prediction



Use rock mass classification for strength prediction

Table 1 : Approximate relationship between rock mass quality and material constants

Disturbed rock mass *m* and *s* values — *undisturbed rock mass m and s values*

EMPIRICAL FAILURE CRITERION $\sigma'_1 = \sigma'_3 + \sqrt{m\sigma_c\sigma'_3 + s\sigma_c^2}$ σ'_1 = major principal effective stress σ'_3 = minor principal effective stress σ_c = uniaxial compressive strength of intact rock, and *m* and *s* are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTALLINE ROCKS – *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES						
Laboratory size specimens free	m	7.00	10.00	15.00	17.00	25.00
from discontinuities	s	1.00	1.00	1.00	1.00	1.00
CSIR rating: RMR = 100	*m*	*7.00*	*10.00*	*15.00*	*17.00*	*25.00*
NGI rating: Q = 500	*s*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
VERY GOOD QUALITY ROCK MASS						
Tightly interlocking undisturbed rock	m	2.40	3.43	5.14	5.82	8.56
with unweathered joints at 1 to 3m.	s	0.082	0.082	0.082	0.082	0.082
CSIR rating: RMR = 85	*m*	*4.10*	*5.85*	*8.78*	*9.95*	*14.63*
NGI rating: Q = 100	*s*	*0.189*	*0.189*	*0.189*	*0.189*	*0.189*
GOOD QUALITY ROCK MASS						
Fresh to slightly weathered rock, slightly	m	0.575	0.821	1.231	1.395	2.052
disturbed with joints at 1 to 3m.	s	0.00293	0.00293	0.00293	0.00293	0.00293
CSIR rating: RMR = 65	*m*	*2.006*	*2.865*	*4.298*	*4.871*	*7.163*
NGI rating: Q = 10	*s*	*0.0205*	*0.0205*	*0.0205*	*0.0205*	*0.0205*

Use rock mass classification for strength prediction

Table 1 : Approximate relationship between rock mass quality and material constants

Disturbed rock mass m and s values — *undisturbed rock mass m and s values*

EMPIRICAL FAILURE CRITERION $\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2}$ σ_1' = major principal effective stress σ_3' = minor principal effective stress σ_c = uniaxial compressive strength of intact rock, and m and s are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTALLINE ROCKS - *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
FAIR QUALITY ROCK MASS						
Several sets of moderately weathered	m	0.128	0.183	0.275	0.311	0.458
joints spaced at 0.3 to 1m.	s	0.00009	0.00009	0.00009	0.00009	0.00009
CSIR rating: RMR = 44	*m*	*0.947*	*1.353*	*2.030*	*2.301*	*3.383*
NGI rating: Q = 1	*s*	*0.00198*	*0.00198*	*0.00198*	*0.00198*	*0.00198*
POOR QUALITY ROCK MASS						
Numerous weathered joints at 30-500mm,	m	0.029	0.041	0.061	0.069	0.102
some gouge. Clean compacted waste rock	s	0.000003	0.000003	0.000003	0.000003	0.000003
CSIR rating: RMR = 23	*m*	*0.447*	*0.639*	*0.959*	*1.087*	*1.598*
NGI rating: Q = 0.1	*s*	*0.00019*	*0.00019*	*0.00019*	*0.00019*	*0.00019*
VERY POOR QUALITY ROCK MASS						
Numerous heavily weathered joints spaced	m	0.007	0.010	0.015	0.017	0.025
<50mm with gouge. Waste rock with fines.	s	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001
CSIR rating: RMR = 3	*m*	*0.219*	*0.313*	*0.469*	*0.532*	*0.782*
NGI rating: Q = 0.01	*s*	*0.00002*	*0.00002*	*0.00002*	*0.00002*	*0.00002*

Estimation of m and s (rock mass)

- Based on the attempts by Priest and Brown (1983), the following updated empirical relations to calculate the constants *m* and *s* were presented (Brown and Hoek, 1988, Hoek and Brown, 1988):

Undisturbed (or Interlocking) Rock Masses

$$m = m_i \, e^{\frac{RMR-100}{28}}$$

$$s = e^{\frac{RMR-100}{9}}$$

Disturbed Rock Masses

$$m = m_i \, e^{\frac{RMR-100}{14}}$$

$$s = e^{\frac{RMR-100}{6}}$$

where

m_i = the value of m for the intact rock, and

RMR = Rock Mass Rating (Bieniawski, 1976).

Generalized Hoek and Brown Failure Criterion

- In the book by Hoek, Kaiser and Bawden (1995) a general form of the Hoek-Brown failure criterion was given. With notations as defined earlier, this is written

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + s \right)^a$$

- For intact rock, i.e. $s = 1$ and $m_b = m_i$,

$$\sigma_1' = \sigma_3' + \sigma_c \left(m_i \frac{\sigma_3'}{\sigma_c} + 1 \right)^{1/2}$$

where

σ_1' = major principal effective stress at failure,
σ_3' = minor principal effective stress at failure,
m_b = the value of the constant m for broken rock, and
a = constant for broken rock.

Estimation of m, s and a (rockmass)

- The constant m_i can be determined from triaxial tests on intact rock or, if test results are not available, from the tabulated data provided by Hoek, Kaiser and Bawden (1995),
- To estimate the value of parameters m_b, s and a, the following relations were suggested by Hoek, Kaiser and Bawden (1995).

$$m_b = m_i \, e^{\frac{GSI-100}{28}} ,$$

For $GSI > 25$ *(Undisturbed rock masses)*

$$s = e^{\frac{RMR-100}{9}} ,$$

$$a = 0.5 .$$

Estimation of m, s and a (rock mass)

- To estimate the value of parameters m_b, s and a, the following relations were suggested by Hoek, Kaiser and Bawden (1995).

For $GSI < 25$ *(Undisturbed rock masses)*

$$s = 0\,,$$

$$a = 0.65 - \frac{GSI}{200}\,,$$

where *GSI* is the *Geological Strength Index*.

GSI is similar to *RMR* but incorporates also newer versions of Bieniawski's original system (Bieniawski, 1976, 1989). Hence, the following relations were developed (Hoek, Kaiser and Bawden, 1995).

Estimates of m and s using GSI

(Hoek et al, 2002)

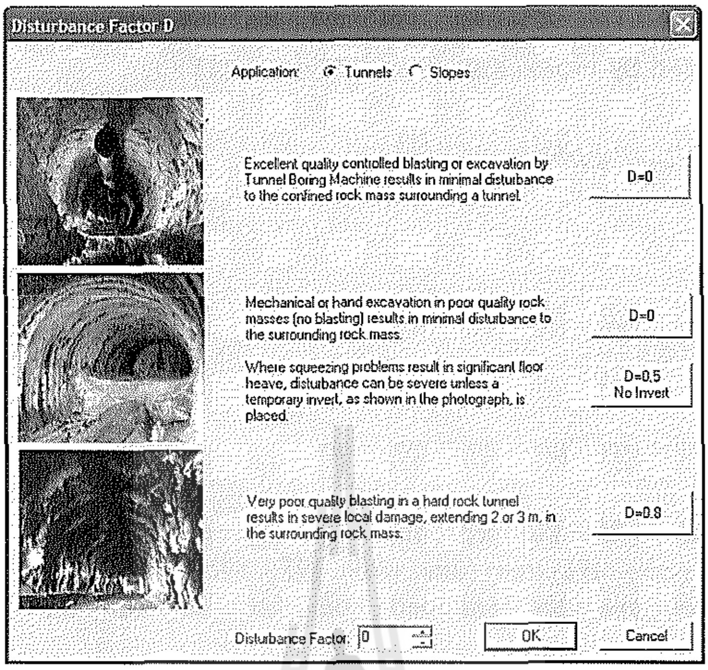
$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right)$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right)$$

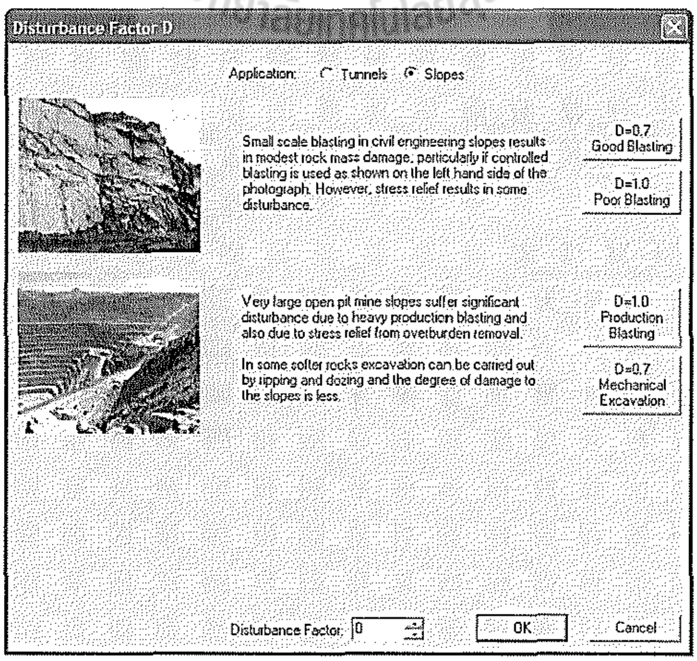
$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. Guidelines for the selection of D are discussed in a later section.

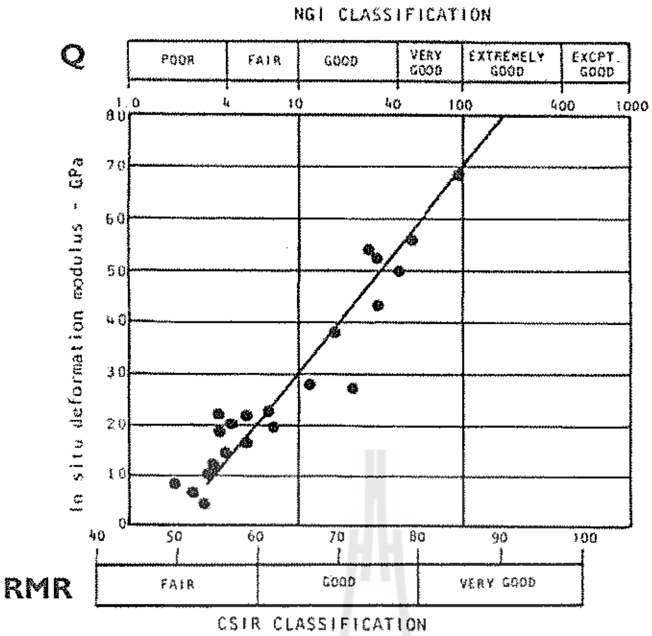
Factor D for Tunnels



Factor D for Slope



Deformability of Rock Mass



Deformability of Rock Mass

RMR > 55 (Bieniawski, 1978)

$E \cong 2\,RMR - 100$ **(GPa)**

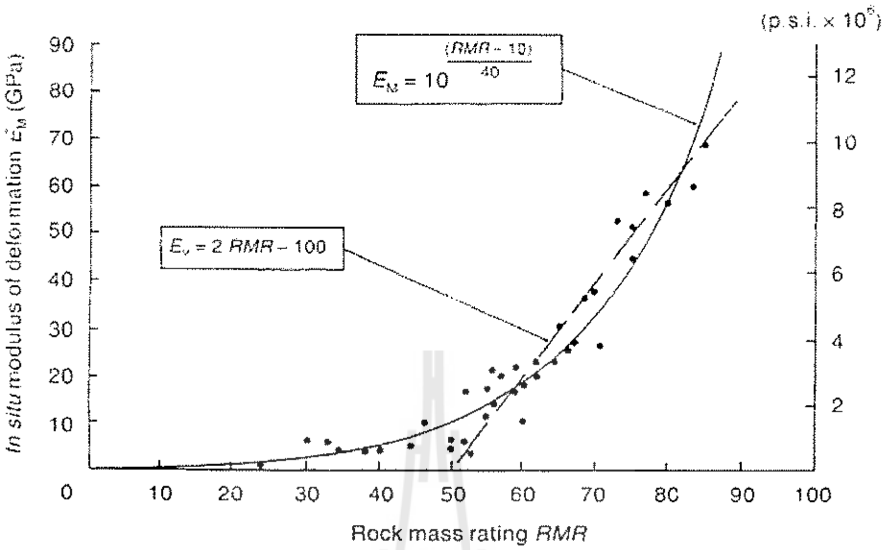
10 < RMR < 50 (Sarafim and Pereira, 1983)

$E \cong 10^{(RMR-10)/40}$ **(GPa)**

Hoek and Brown (1980)

$E \cong 17.5 \ln(Q) - 10.17$ **(GPa)**

Deformability of Rock Mass



Deformability of Rock Mass

The rock mass modulus of deformation is given by:

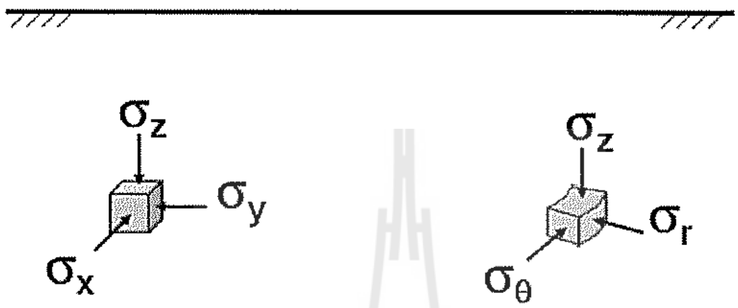
$$E_m(GPa) = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{((GSI-10)/40)} \qquad \text{for } \sigma_{ci} \le 100 \text{ MPa}$$

$$E_m(GPa) = \left(1 - \frac{D}{2}\right) \cdot 10^{((GSI-10)/40)} \qquad \text{for } \sigma_{ci} > 100 \text{ MPa}$$

(Hoek et al, 2002)

In situ stresses field

- Stress field before excavation is represented by 3 principal stresses:
  1. **Vertical stress** is generally equal to the overburden stress
  2. **Horizontal stresses** are influenced by tectonic stress (in rock) and earth pressure coefficient (in soil)
- If the excavation is below water table, it is necessary to take into consideration **water pressure** (effective stress law)



Vertical stress

- Depth below the surface, z = *1,000 m*
- Unit weight of the rock, $\gamma = 0.027$ MN/m^3
- The weight of the vertical column of rock? → *27 MPa*

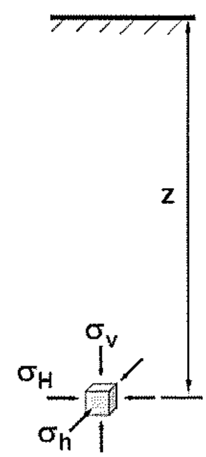
Vertical stress on the element

$$\sigma_v = \gamma z$$

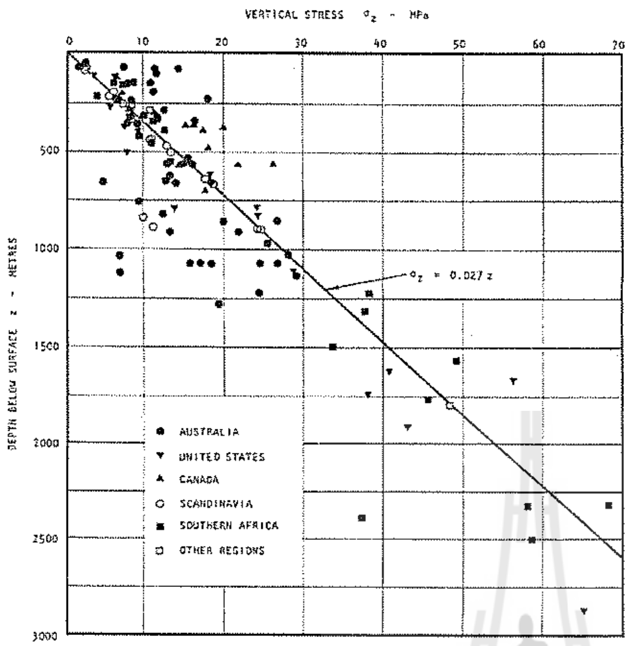
where σ_v is the vertical stress

γ is the unit weight of the overlying rock and

Z is the depth below surface



Vertical stress



Vertical stress measurements from mining and civil engineering projects around the world. (After Brown and Hoek 1978).

$$\sigma_v \approx 0.027 \text{ MPa/m}$$
$$\approx 1 \text{ psi/ft}$$

As a rule of thumb, taking the average density of rock into account, 40 m of overlying rock induces 1 MPa stress.

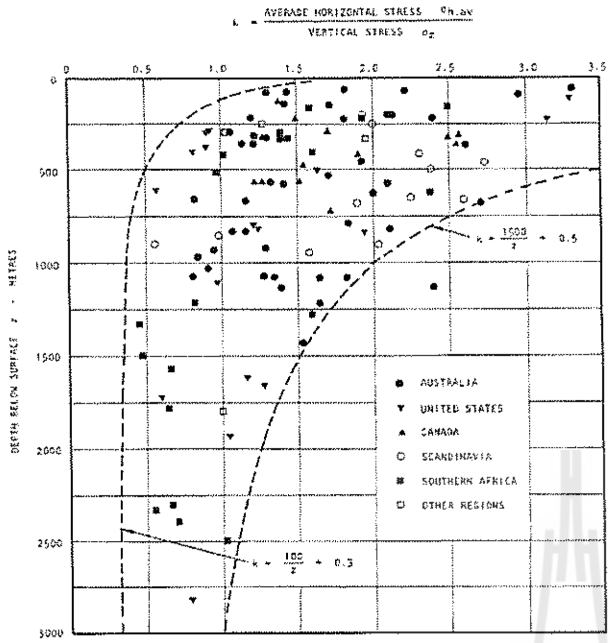
Horizontal stress

- Much more difficult to estimate than the vertical stresses
- Ratio of the average horizontal stress to the vertical stress, *k*
- k increases when shallow depth decreases

Horizontal stress on the element

$$\sigma_h = k\sigma_v = k\gamma z$$

Horizontal stress



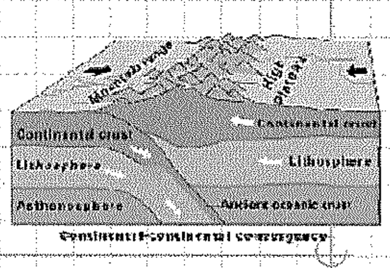
$(100/z)+0.3 < k < (1500/z)+0.5$

Reason for High Horizontal Stress

High horizontal stresses are caused by factors relating to erosion, tectonics, rock anisotropy, local effects near discontinuities, and scale effects:

Erosion - if horizontal stresses become 'locked in', then the erosion/removal of overburden (i.e. decrease in σ_V) will result in an increase in *K* ratio (σ_H/σ_V).

Tectonics - different forms of tectonic activity (e.g. subduction zones), can produce high horizontal stresses.



Horizontal stress

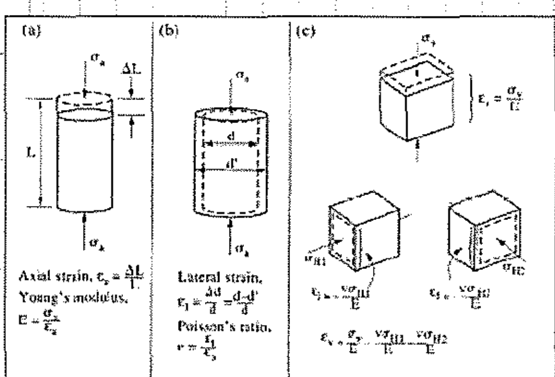
- Terzaghi and Richart (1952) suggested that, for a gravitationally loaded rock mass in which no lateral strain was permitted during formation of the overlying strata, the value of k is independent of depth and is given by

$$k = \nu/(1-\nu)$$

where ν *is the* Poisson's ratio of the rock mass.

Horizontal stress

The horizontal stress can be estimated using of elastic theory. If we consider the strain along any axis of a small cube at depth, then the total strain can be found from the strain due to the axial stress, subtracting the strain components due to the two perpendicular stresses.



Hudson & Harrison (1997)

For example:

$$\varepsilon_V = \frac{\sigma_V}{E} - \frac{\nu\sigma_{H1}}{E} - \frac{\nu\sigma_{H2}}{E}$$

$$\varepsilon_{H1} = \frac{\sigma_{H1}}{E} - \frac{\nu\sigma_{H2}}{E} - \frac{\nu\sigma_V}{E}$$

Horizontal stress

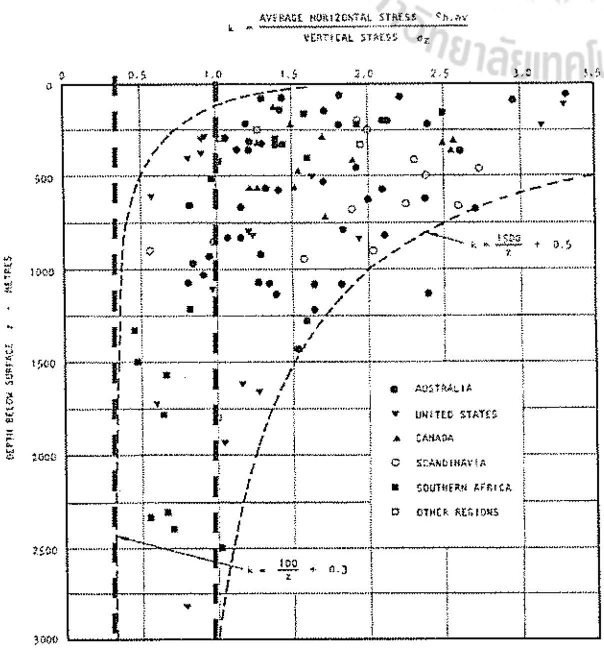
To provide an initial estimate of the horizontal stress, two assumptions are made:

- the two horizontal stresses are equal;
- there is no horizontal strain, i.e. both ε_{H1} and ε_{H2} are zero (e.g. because it is restrained by adjacent elements of rock).

Then we can take ε_{H1} as zero: $$0 = \frac{\sigma_{H1}}{E} - \frac{\nu\sigma_{H2}}{E} - \frac{\nu\sigma_V}{E}$$

And, because $\sigma_{H1} = \sigma_{H2}$: $$\sigma_H = \frac{\nu}{1-\nu}\sigma_V$$

Horizontal stress



Thus the ratio between the horizontal and vertical stress (referred to as $K = \sigma_H/\sigma_V$) is a function of the Poisson's ratio:

$$\frac{\sigma_H}{\sigma_V} = \frac{\nu}{1-\nu}$$

For a typical Poisson's ratio (ν) of 0.25, the resulting K ratio is 0.33. For a theoretical maximum of ν = 0.5, the maximum K ratio predicted is 1.0.

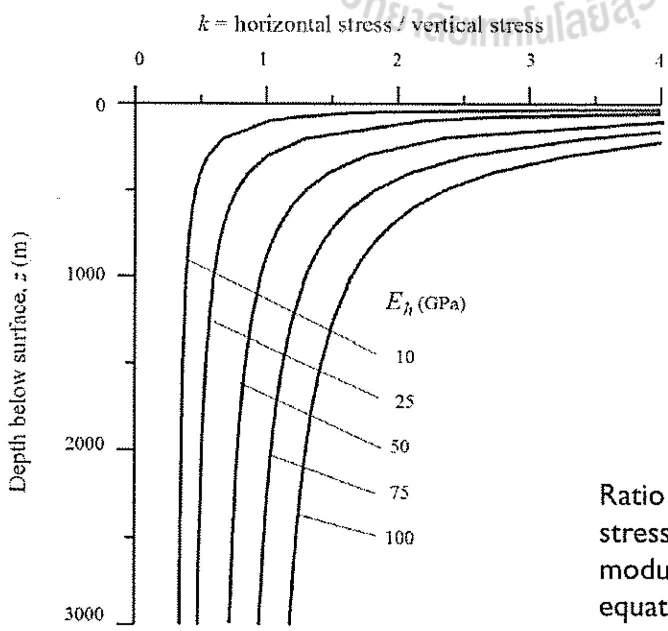
Horizontal stress

- Sheorey (1994) developed an elasto-static thermal stress model of the earth. This model considers curvature of the crust and variation of elastic constants, density and thermal expansion coefficients through the crust and mantle.
- He provide a simplified equation can be used for estimating the horizontal to vertical stress ratio *k*.

$$k = 0.25 + 7E_h\left(0.001 + \frac{1}{z}\right)$$

where z (m) is the depth below surface and E_h (GPa) is the average deformation modulus of the upper part of the earth's crust measured in a horizontal direction.

Horizontal stress

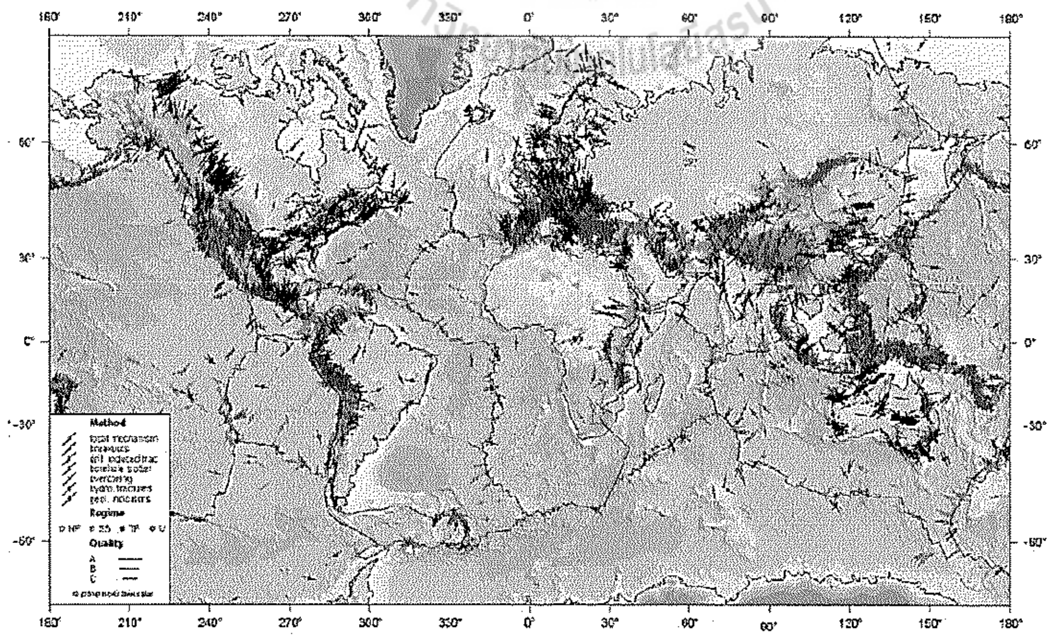


Ratio of horizontal to vertical stress for different deformation moduli based upon Sheorey's equation. (After Sheorey 1994).

The World stress map

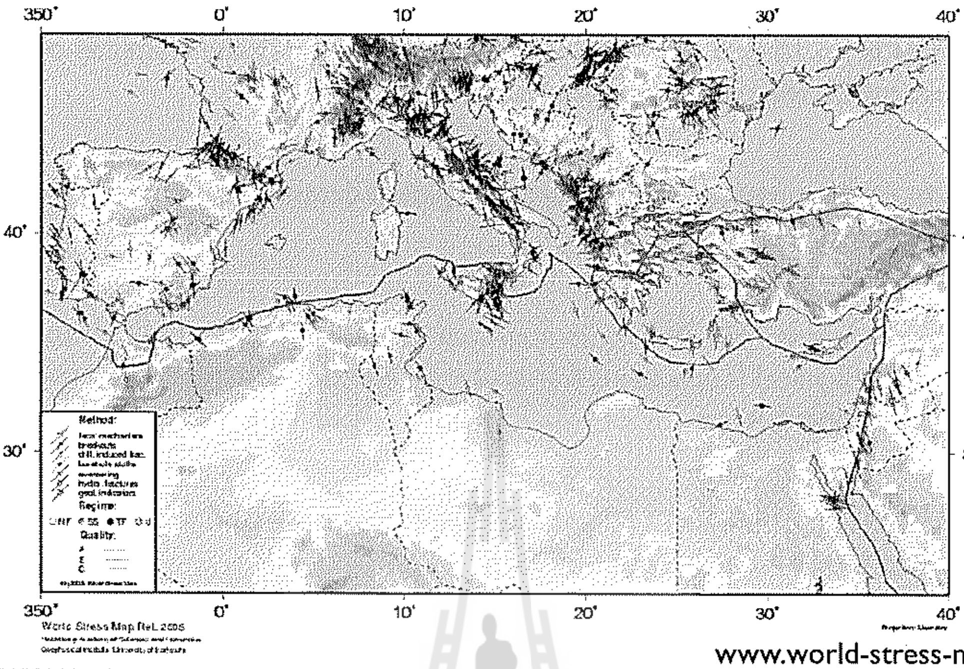
- The World Stress Map project, completed in July 1992, involved over 30 scientists from 18 countries and was carried out under the auspices of the International Lithosphere Project (Zoback, 1992).
- The aim of the project was to compile a global database of contemporary tectonic stress data.
- The World Stress Map (WSM) is now maintained and it has been extended by the Geophysical Institute of Karlsruhe University as a research project of the Heidelberg Academy of Sciences and Humanities.
- The WSM is an open-access database that can be accessed at www.world-stressmap.org (Reinecker et al, 2005)

World stress map giving orientations of the maximum horizontal compressive stress



www.world-stress-map.org

Stress map of the Mediterranean giving orientations of the maximum horizontal compressive stress

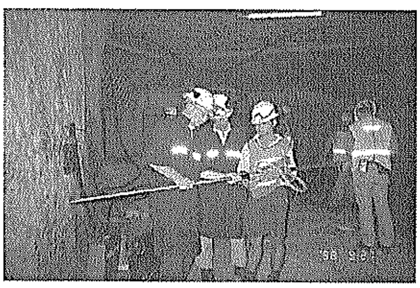


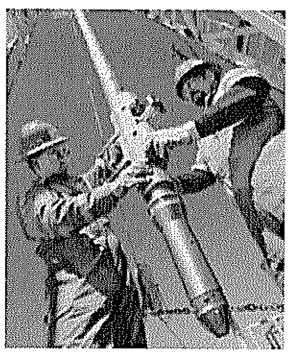
www.world-stress-map.org

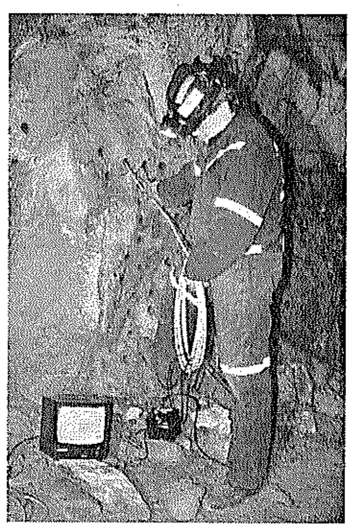
In-situ Test and Measurements

Objectives:

1. To determine in-situ stress (σ_v and σ_h)
2. To determine rock mass properties



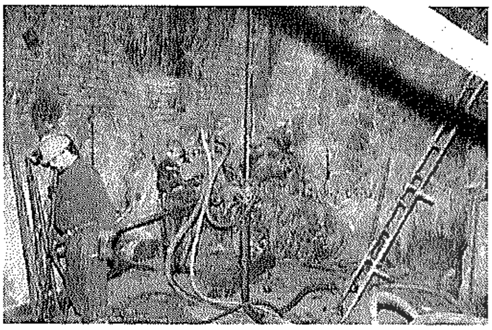


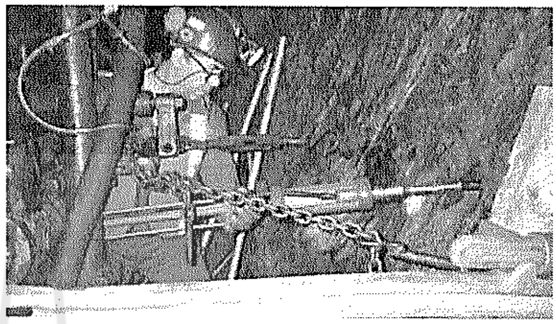


In-situ Test and Measurements

Measurements from:

- Borehole / Drill hole
- Outcrop
- Tunnel wall / Pillar





In-situ Test and Measurements

In-situ stress Measurement Methods :

1. Hydraulic Fracturing *
2. Flat Jack *
3. Overcoring *
4. Doorstopper
5. Undercoring

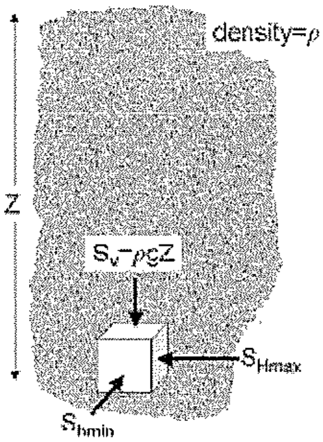
Elastic Modulus Measurement Methods :

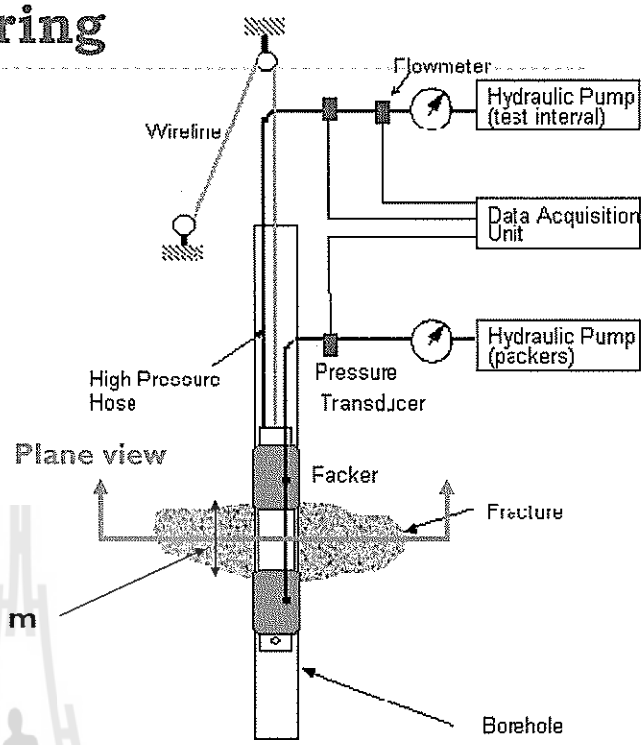
1. Plate Bearing Test
2. Dilatometer Test
3. Flat Jack Test

In-situ Direct Shear Test:

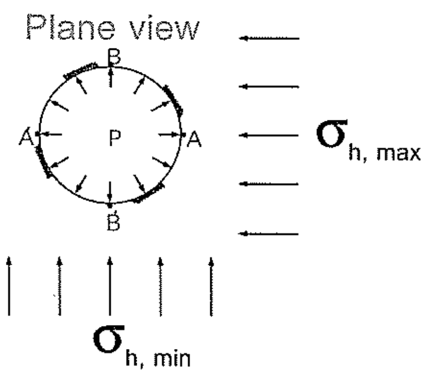
Hydraulic Fracturing

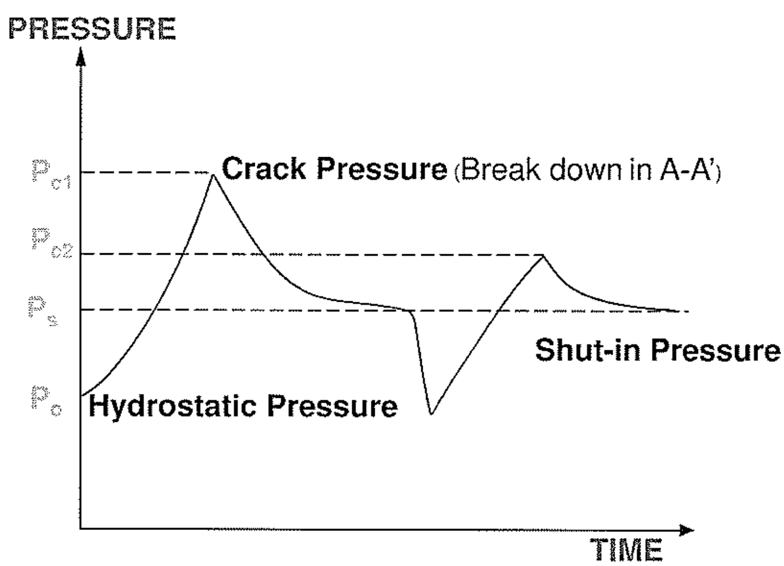
Measurement in Borehole/Drill hole





Hydraulic Fracturing Method

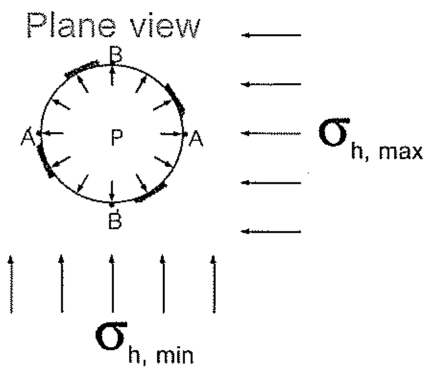




Hydraulic Fracturing Method

Assumptions:

- homogenous / continuous
- linear elastic
- isotropic



Applied from Kirsch Solution

Point A & A' (Radial Crack Occurred)

$P_x = \sigma_{h,max}$, $P_y = \sigma_{h,min}$, $r=a$, $\theta=0$
and P=0 (internal pressure)

$$\sigma_\theta = \frac{1}{2}\left\{(P_x + P_y)\left(1+\frac{a^2}{r^2}\right) - (P_x - P_y)\left(1+\frac{3a^4}{r^4}\right)\cos 2\theta\right\}$$

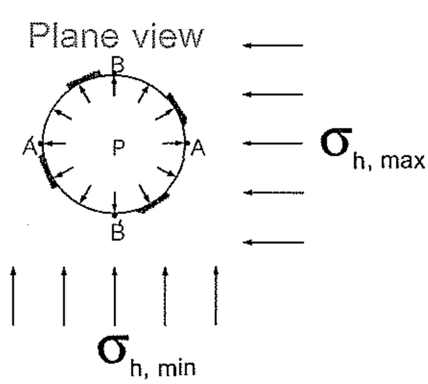
$$\sigma_\theta = 3\sigma_{h,\,min} - \sigma_{h,\,max}$$

Hydraulic Fracturing Method

Applied from Kirsch Solution

Point A & A' (Radial Crack Occurred)

$P_x = \sigma_{h,max}$, $P_y = \sigma_{h,min}$, $r=a$, $\theta=0$
and $P=P_{cl}$ (internal pressure)



Tangential stress at Point A or A'

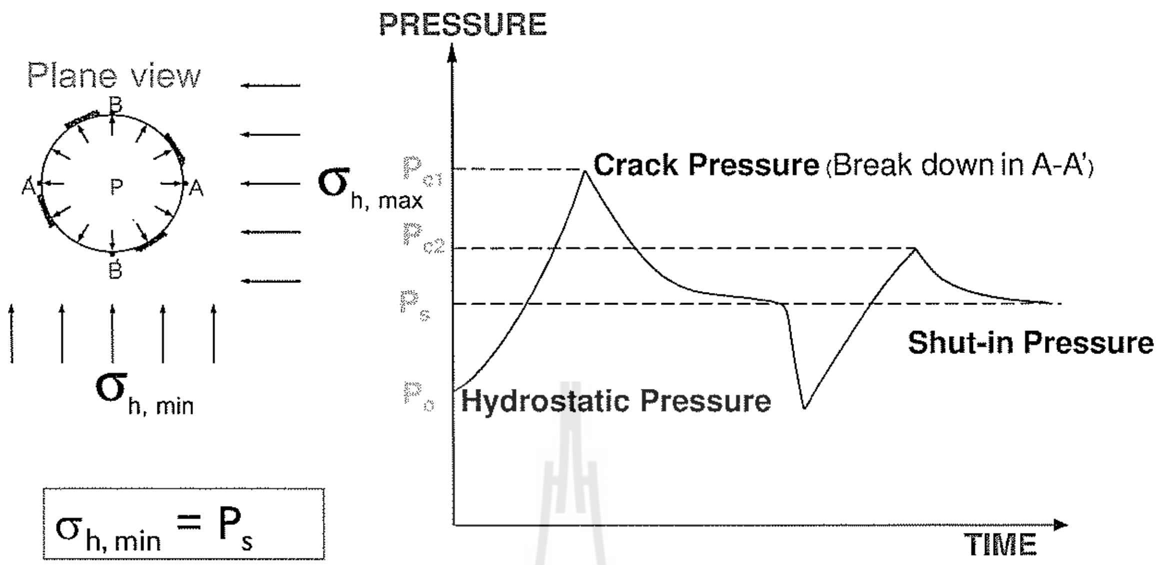
$$\sigma_\theta = 3\sigma_{h,\,min} - \sigma_{h,\,max} - P_{cl}$$

For Radial Crack Occurred ($\sigma_\theta = -T_o$)

T_0 = tensile strength of rock around borehole

$$3\sigma_{h,\,min} - \sigma_{h,\,max} - P_{cl} = -T_0$$

Hydraulic Fracturing Method



$\sigma_{h,\ min} = P_s$

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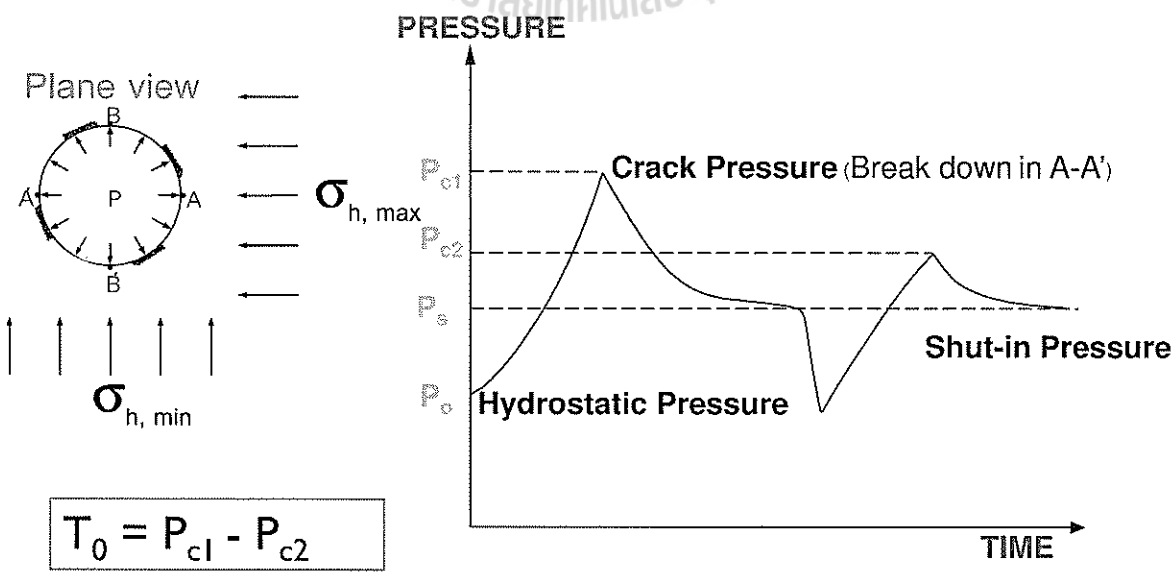
Hydraulic Fracturing Method



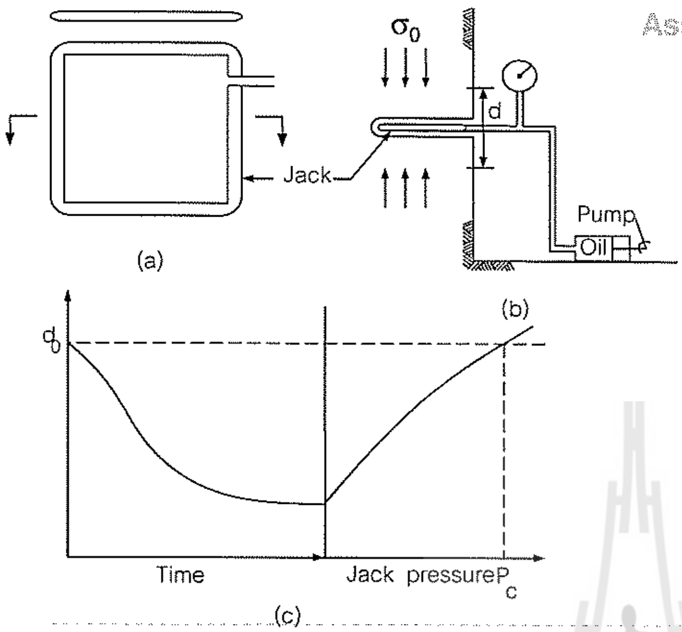
$T_0 = P_{c1} - P_{c2}$

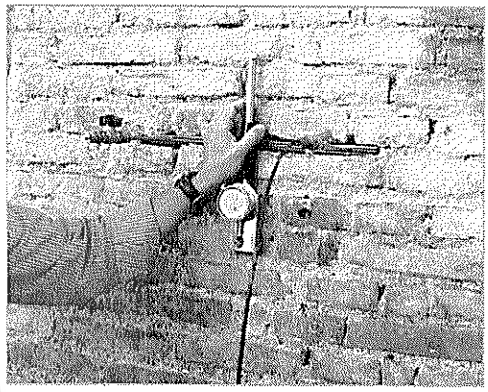
Flat Jack Method

Measurement in Tunnel wall / Pillar

Assumption = Perfectly Elastic

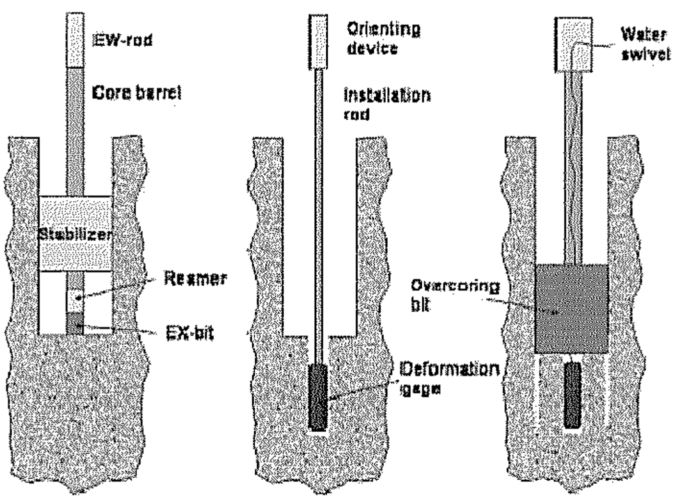
$$P_c = \sigma_0$$

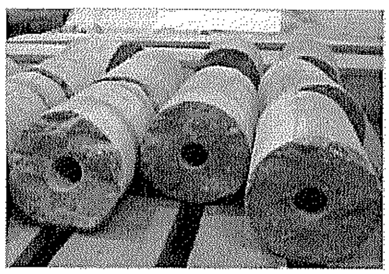


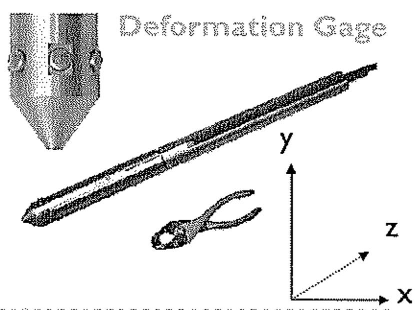


Overcoring Method

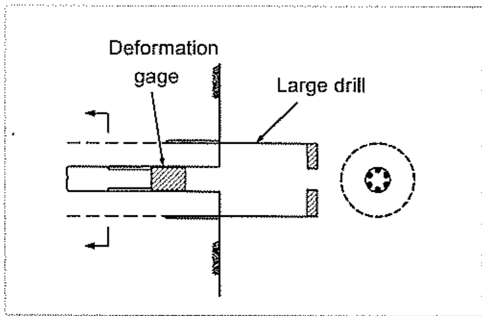
Measurement in Borehole/Drill hole
Tunnel wall / Pillar

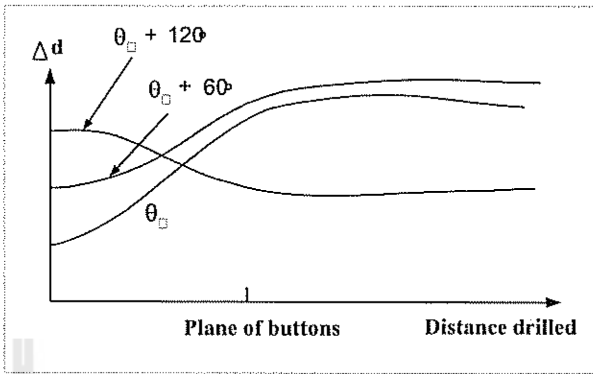






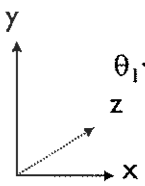
Overcoring Method

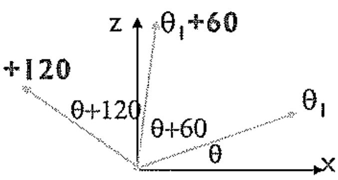




Change in diameter

$$\Delta d(\theta) = \sigma_x f_1 + \sigma_y f_2 + \sigma_z f_3 + \tau_{xz} f_4$$





Overcoring Method

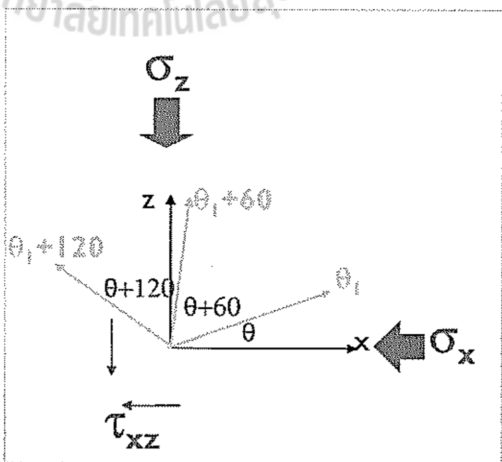
where

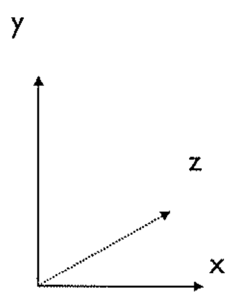
$$f_1 = d(1+2\cdot\cos 2\theta)\frac{1-\nu^2}{E} + \frac{d\nu^2}{E}$$

$$f_2 = -\frac{d\nu}{E}$$

$$f_3 = d(1-2\cdot\cos 2\theta)\frac{1-\nu^2}{E} + \frac{d\nu^2}{E}$$

$$f_4 = d(4\cdot\sin 2\theta)\frac{1-\nu^2}{E}$$





Overcoring Method

Equation

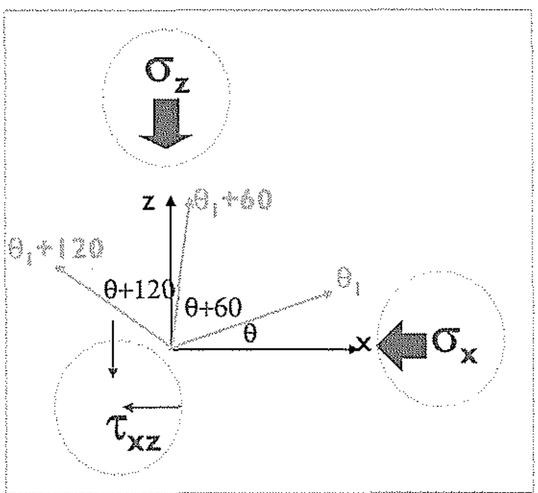
$$\begin{Bmatrix} \Delta d(\theta_1) & - f_2\,\sigma_y \\ \Delta d(\theta_1 + 60) & - f_2\,\sigma_y \\ \Delta d(\theta_1 + 120) & - f_2\,\sigma_y \end{Bmatrix} = \begin{bmatrix} f_{11} & f_{13} & f_{14} \\ f_{21} & f_{23} & f_{24} \\ f_{31} & f_{33} & f_{34} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_z \\ \tau_{xz} \end{Bmatrix}$$

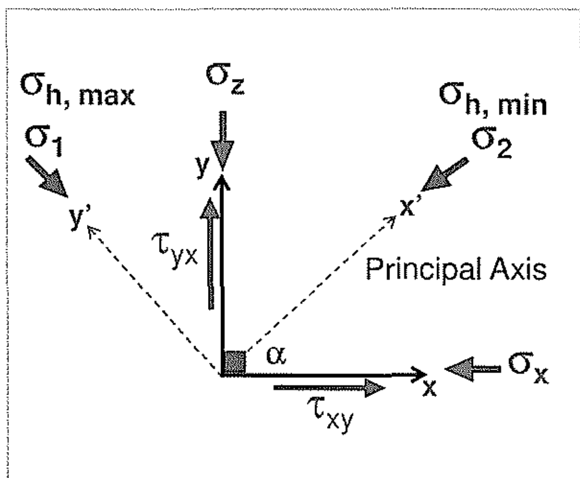
Δd = Change in diameter

f_{mn} ← n = 1,2,4 indicate the value of f_1, f_3 and f_4

m = 1,2,3 indicate the position of θ_1 , θ_1+60 and θ_1+120

Overcoring Method

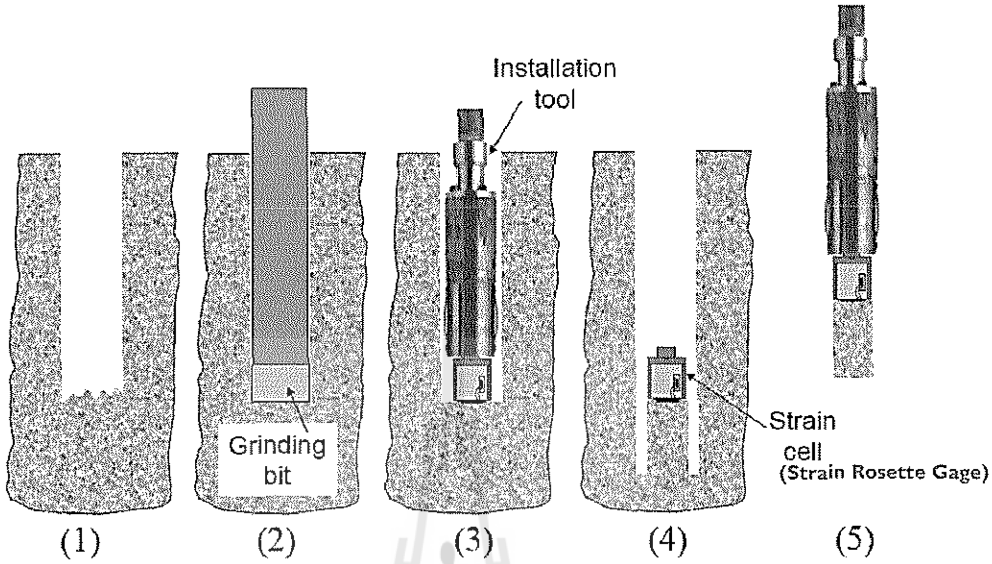




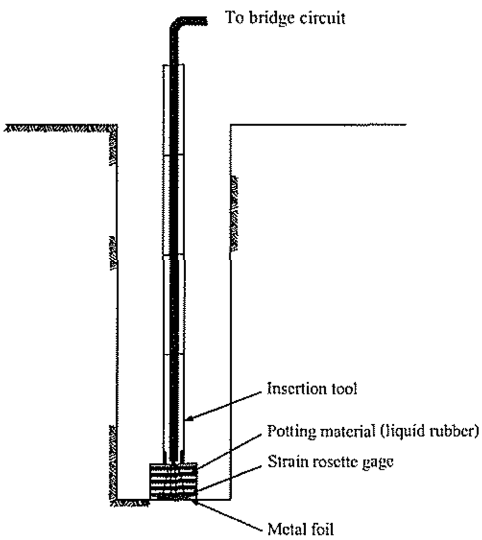
Horizontal Plane

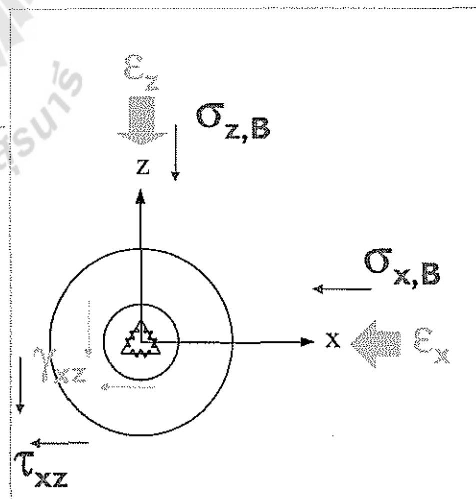
Doorstopper

Measurement in Borehole/Drill hole
Tunnel wall / Pillar



Doorstopper



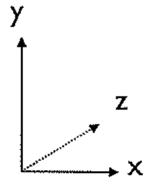


$$\begin{Bmatrix} \Delta\sigma_{x,B} \\ \Delta\sigma_{z,B} \\ \Delta\tau_{xz,B} \end{Bmatrix} = \frac{E}{1-\nu^2} \begin{bmatrix} 1 & \nu & 0 \\ \nu & 1 & 0 \\ 0 & 0 & \frac{1-\nu}{2} \end{bmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_z \\ \gamma_{xz} \end{Bmatrix}$$

Doorstopper

Calculate the stresses and shear stress

$$\begin{Bmatrix} \Delta\sigma_{x,B} \\ \Delta\sigma_{z,B} \\ \Delta\tau_{xz,B} \end{Bmatrix} = - \begin{bmatrix} a & c & b & 0 \\ b & c & a & 0 \\ 0 & 0 & 0 & d \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xz} \end{Bmatrix}$$



$a = 1.30$

$b = (0.085 + 0.15\,\nu - \nu^2)$

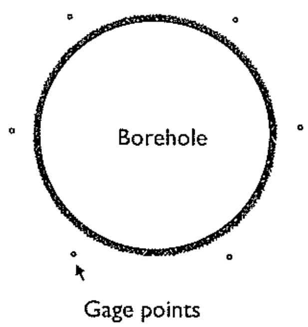
$c = (0.473 + 0.91\nu)$

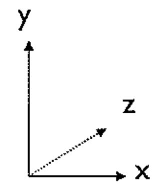
$d = (1.423 - 0.027\nu)$

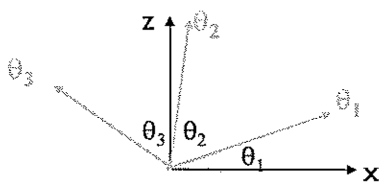
(Goodman, 1989)

Undercoring

Measurement in Borehole/Drill hole
Tunnel wall / Pillar







$$u_r = \sigma_x \cdot f_1 + \sigma_z \cdot f_2 + \tau_{xz} \cdot f_3$$

$$f_1 = \frac{1}{2E} \cdot \frac{a^2}{r}[(1+\nu) + H \cdot \cos 2\theta]$$

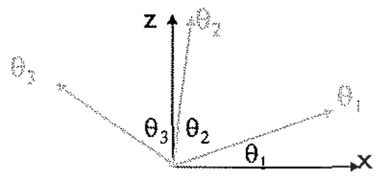
$$f_2 = \frac{1}{2E} \cdot \frac{a^2}{r}(1+\nu) - H \cdot \cos 2\theta$$

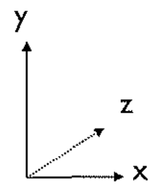
$$f_3 = \frac{1}{E} \cdot \frac{a^2}{r} H \cdot \sin 2\theta$$

$$H = 4 - (1+\nu)a^2 / r^2$$

Undercoring

$$\begin{Bmatrix} u_{r,1} \\ u_{r,2} \\ u_{r,3} \end{Bmatrix} = \begin{bmatrix} f_{11} & f_{12} & f_{13} \\ f_{21} & f_{22} & f_{23} \\ f_{31} & f_{32} & f_{33} \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_z \\ \tau_{xz} \end{Bmatrix}$$



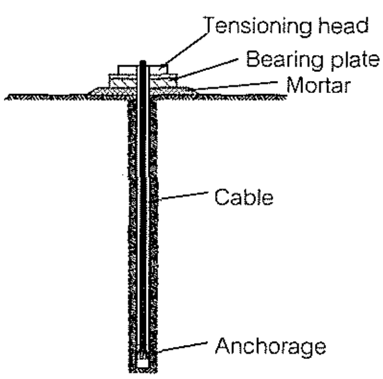


f_{mn} ← n = 1,2,3 indicate the value of f_1, f_2 and f_3

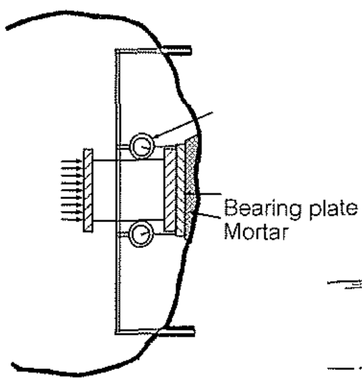
m = 1,2,3 indicate the position of θ_1 , θ_2 and θ_3

Plate Bearing Test

Measurement in Outcrop/Tunnel wall



Rigid Plate Bearing Test (ASTM D3494)



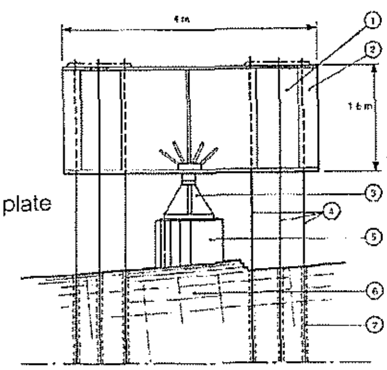


Plate Load Test

- Rigid Plate Bearing Test ASTM D3494
- Flexible Plate Bearing Test ASTM D3495

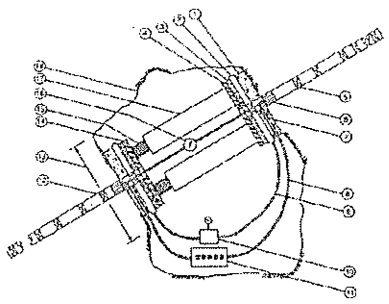


Figure 4.17 Typical set up for a uniaxial jacking test in which the load is applied through hydraulic flatjacks (Misterek et al., 1974, © ASTM, reprinted with permission).
1. Concrete pad. 2. 1 m diameter flatjack. 3. Particle board pad. 4. Top plate. 5. MPBX anchors – 5 or more/hole. 6. MPBX sensor head. 7. Rubber sleeve over lead wires. 8. Transducer lead wire. 9. Hydraulic hoses. 10. Hydraulic pump, 70 MPa. 11. Data-acquisition system MPa. 12. NX drill hole, depth ≈ 6 flatjack diameters. 13. Prepared diameter, 1.5 to 2 × flat jack diameter. 14. Base plate. 15. Screws for set up and removal. 16. Tunnel diameter gauge. 17. 254 mm diameter aluminum columns. 18. Tunnel surface.

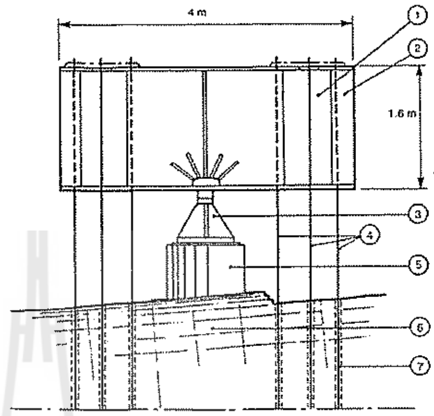


Figure 4.18 Typical arrangement of plate load test at ground surface (Pusch, 1992). 1. Hydraulic jacks. 2. Steel beam reaction head. 3. Steel lid. 4. Tie rods. 5. Concrete foundation. 6. Schistose gneiss. 7. 100 mm dia. anchor holes.

Rigid Plate Bearing Test

Designation: D 4394 – 04

Standard Test Method for Determining the In Situ Modulus of Deformation of Rock Mass Using the Rigid Plate Loading Method[1]

Rigid Plate Bearing Test

D 4394 – 04

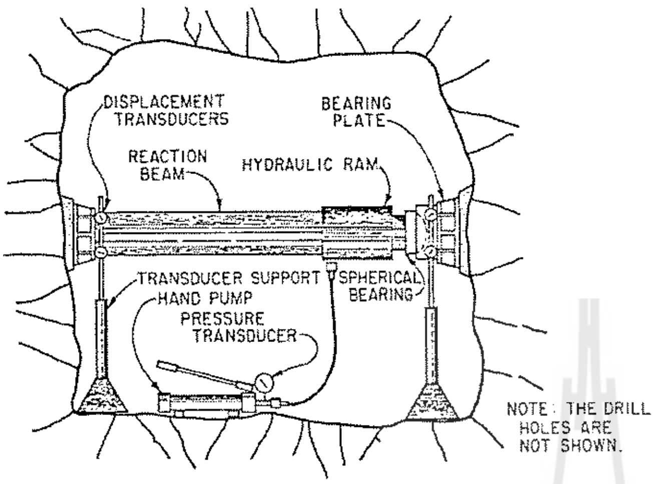


FIG. 4 Typical Rigid Plate Bearing Test Setup Schematic

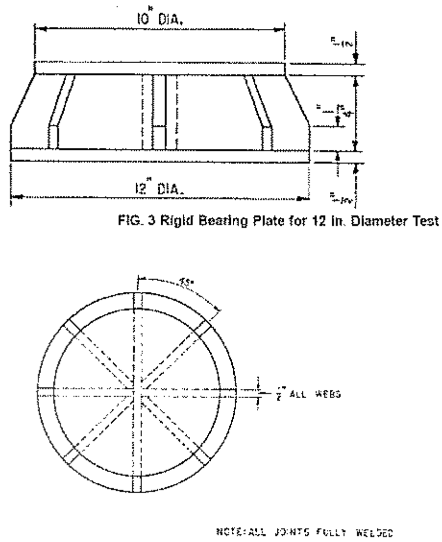
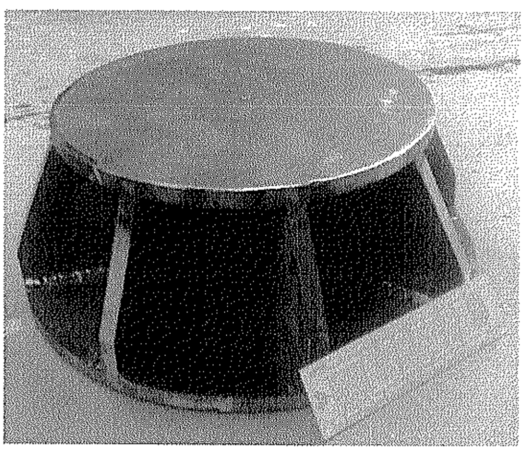
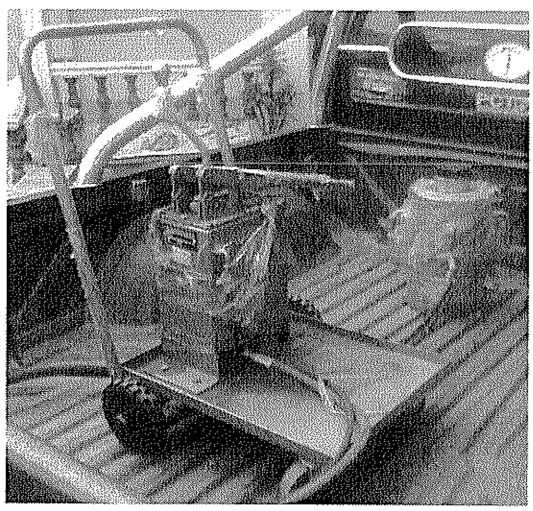


FIG. 3 Rigid Bearing Plate for 12 in. Diameter Test

Rigid Plate Bearing Test

Nam Ngum 3 Project





Rigid Plate Bearing Test

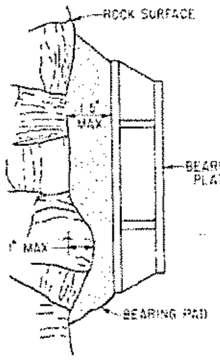


FIG. 5 Allowable Dimensions for Rock Surface and Bearing Pad

$$E = \frac{(1-\mu^2)\cdot P}{2W_a \cdot R}$$

where:

μ = Poisson's ratio of the rock,
P = total load on the rigid plate, lbf (kN),
W_a = average deflection of the rigid plate, in. (mm), and
R = radius of the rigid plate, in. (mm).

Flexible Plate Bearing Test

Designation: D 4395 – 04

Standard Test Method for
Determining the In Situ Modulus of Deformation of Rock Mass Using the Flexible Plate Loading Method[1]

Flexible Plate Bearing Test

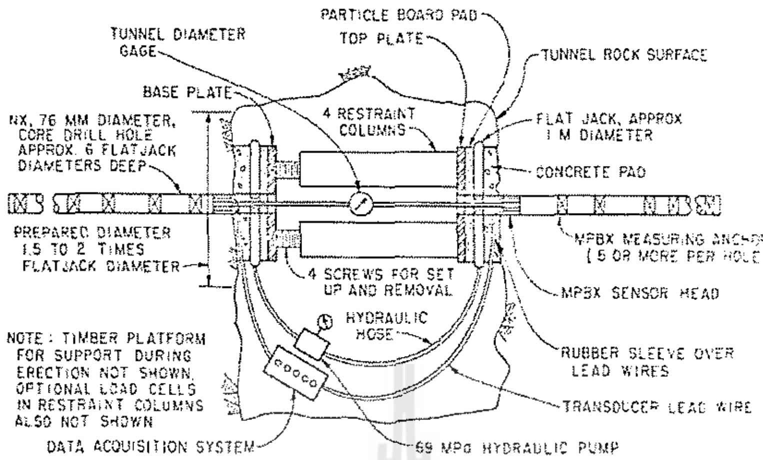


FIG. 3 Typical Flexible Plate Bearing Test Setup Schematic

Flexible Plate Bearing Test

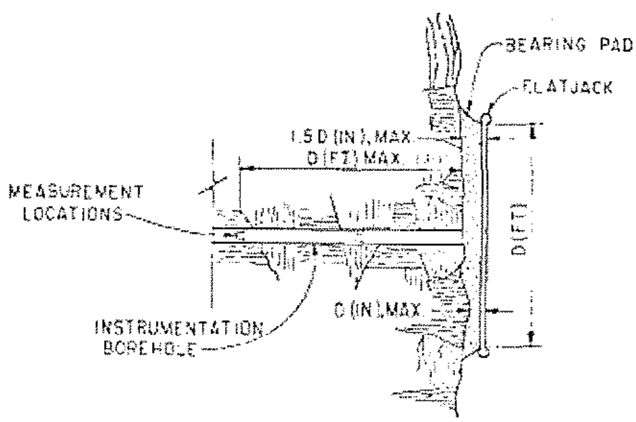


FIG. 4 Allowable Dimensions for Rock Surface and Bearing Pad, Flexible Plate Loading Test

Flexible Plate Bearing Test

E—Calculate the modulus, *E*, from the deflection at the center of a circularly loaded area at the rock surface as follows:

$$E = \frac{2(1 - \gamma^2)QR}{W_c}$$

where:
γ = Poisson's ratio of the rock,
Q = pressure on loaded area, lbf/in^2 (MPa),
R = radius of loaded area, in. (mm), and
W_c = deflection at center of loaded area, in. (mm).

Flexible Plate Bearing Test

Calculate the modulus, *E* from the deflection at the edge of a circularly loaded area at the rock surface as follows:

$$E = \frac{4(1 - \gamma^2)QR}{\pi W_e}$$

where:
γ = Poisson's ratio of the rock,
Q = pressure on loaded area, lbf/in^2 (MPa),
R = radius of loaded area, in. (mm), and
W_e = deflection at the edge of the loaded area, in. (mm).

Flexible Plate Bearing Test

Calculate the modulus, E, from the deflection at a point within the rock mass beneath the center of a circularly loaded area as follows:

$$E = \frac{2Q(1-\gamma^2)}{W_z}((R^2+Z^2)^{1/2}-Z) - \frac{QZ(1+\gamma)}{W_z}(Z(R^2+Z^2)^{-1/2}-1)$$

where:
Z = depth beneath center of loaded area, in. (mm), and
W_z = deflection at depth z, in. (mm).

Flexible Plate Bearing Test

Calculate the modulus, E, from the deflection at the center of an annularly loaded area at the rock surface as follows:

$$E = \frac{2Q(1-\gamma^2)(R_2-R_1)}{W_c}$$

where:
R_2 = outside radius of annulus, in. (mm), and
R_1 = inside radius of annulus, in. (mm).

Calculate the modulus, E, from the deflection at the edge of an annularly loaded area at the rock surface as follows:

$$E = \frac{4Q(1-\gamma^2)(R_2-R_1)}{\pi W_e}$$

Flexible Plate Bearing Test

Calculate the modulus, E, from the deflection at a point within the rock mass beneath the center of an annularly loaded area as follows:

$$E = \frac{2Q(1-\gamma^2)}{W_z}\left[(R_2^{\,2}+Z^2)^{1/2} - (R_1^{\,2}+Z^2)^{1/2}\right] + \frac{Z^2 \cdot Q(1+\gamma)}{W_z}\left[(R_1^{\,2}+Z^2)^{-1/2} - (R_2^{\,2}+Z^2)^{-1/2}\right]$$

The deflection, W_z, along the center line beneath the loaded area may be expressed in a general form (inches or millimetres) from equations Eq 3 or Eq 6 as follows:

$$W_z = \frac{Q}{E} \cdot K_z$$

Flexible Plate Bearing Test

From this, it follows that the modulus, E, may be calculated from the relative deflection between two positions below the center of the loaded area as follows:

$$E = Q\frac{K_{z_1} - K_{z_2}}{w_{z_1} - w_{z_2}}$$

where:

K_{z_1}, K_{z_2} = geometric coefficients for depths z_1 and z_2, respectively, and

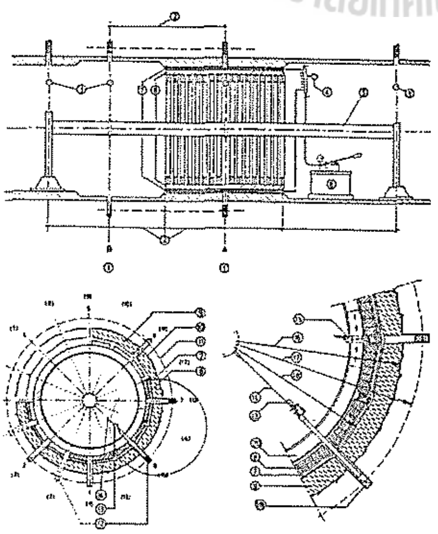
w_{z_1}, w_{z_2} = deflection at depths z_1 and z_2, respectively.

Radial Jacking Test

Designation: D 4506 – 02 (Reapproved 2006)

Standard Test Method for Determining the In Situ Modulus of Deformation of Rock Mass Using a Radial Jacking Test[1]

Radial Jacking Test



1. Measuring profile. 2. Distance equal to the length of active loading. 3. Control extensometer. 4. Pressure gage. 5. Reference beam. 6. Hydraulic pump. 7. Flat jack. 8. Hardwood lagging. 9. Shotcrete. 10. Excavation diameter. 11. Measuring diameter. 12. Extensometer drillholes. 13. Dial gage extensometer. 14. Steel rod. 15. Expansion wedges. 16. Excavation radius. 18. Inscribed Circle. 19. Rockbolt anchor. 20. Steel ring.

FIG. 1 Radial Jacking Test

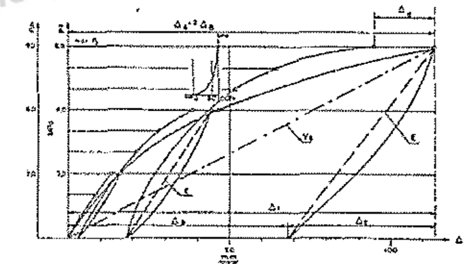


FIG. 2 Typical Graph of Applied Pressure Versus Displacement

Radial Jacking Test

$$p_1 = \frac{\Sigma b}{2 \cdot \pi \cdot r_1} p_m$$

where:

p_1 = distributed pressure on the lining at r_1, psi (MPa),
r_1 = radius, ft (m),
p_m = pressure in the flat jacks, psi (MPa), and
b = flat jack width (see Fig. 3), ft (m).

$$P_2 = \frac{r_1}{r_2} \cdot P_1 = \frac{\Sigma b}{2 \cdot \pi \cdot r_2} \cdot P_m$$

$$P_m \Sigma b = P_1 \cdot 2 \cdot r_1 \cdot \pi$$

$$P_1 = \frac{P_m \Sigma b}{2 \cdot \pi \cdot r_1}$$

$$P_2 = P_1 \frac{r_1}{r_2}$$

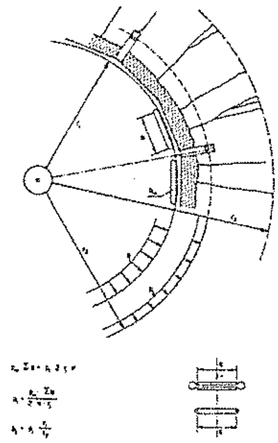


FIG. 3 Scheme of Loading Showing Symbols Used in the Calculations

Radial Jacking Test

$$\Delta_t = \Delta_p + \Delta_e$$

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \cdot \frac{(1+\nu)}{\nu}$$

$$D = \frac{p_2 \cdot r_2}{\Delta_t} \cdot \frac{(1+\nu)}{\nu}$$

where:

p_2 = maximum test pressure, and

ν = estimated value for Poisson's Ratio.

$$E = \frac{p_2 \cdot r_2}{\Delta_e} \cdot \left(\frac{\nu + 1}{\nu} + ln \frac{r_3}{r_2}\right)$$

$$D = \frac{p_2 \cdot r_2}{\Delta_t} \cdot \left(\frac{\nu + 1}{\nu} + ln \frac{r_3}{r_2}\right)$$

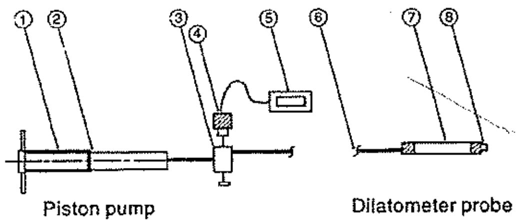
where:

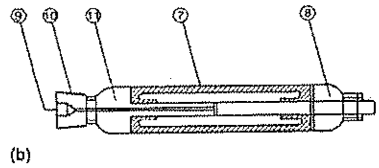
r_3 = radius to the limit of the assumed fissured and loosened zone, ft (m), and
ln = natural logarithm.

Dilatometer Test

Measurement in Borehole/Drill hole

- Advantage : can be made remote from surface as part of the exploration program.





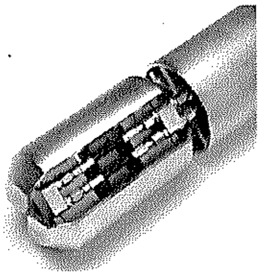


Figure 4.15 Dilatometer for making modulus measurements in boreholes (ISRM, 1987): (a) components of a dilatometer system; and (b) cross section showing fabrication details of CSM-type dilatometer.
1. Piston actuator.
2. Vernier.
3. Valve.
4. Pressure transducer.
5. Pressure readout.
6. High-pressure stainless-steel tubing.
7. Polyurethane rubber membrane.
8. Removable end cap.
9. High-pressure connection.
10. Pipe thread for insertion tool.
11. Fluid passage.

Dilatometer Test

$$G_d = k_R \frac{\pi L d^2}{\rho}$$ → Shear Modulus

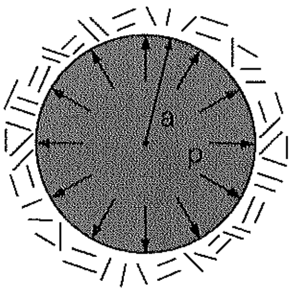
and

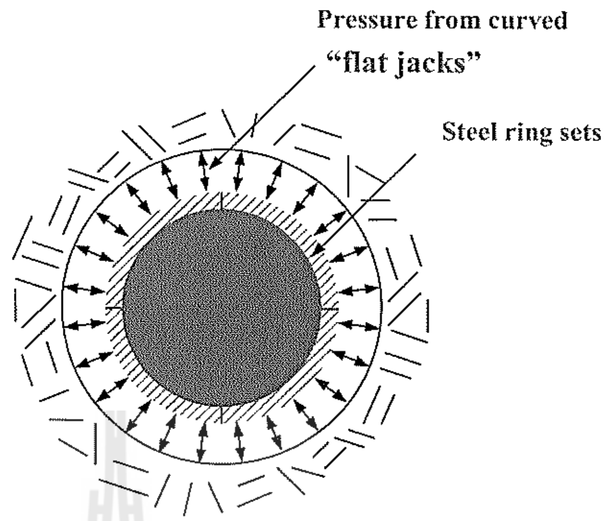
$$E_d = 2(1 + \nu_R) G_d$$ → Modulus of Elasticity

Where L = length of test section (cell membrane)
d = diameter of drill hole test section
ν_R = Poisson's ratio of rock
ρ = pump constant
(fluid volume displaced per turn of pump wheel)

$$k_R = \frac{k_s k_T}{(k_s - k_T)} \text{ (MPa/turn)}$$ → Stiffness of rock

Dilatometer Test

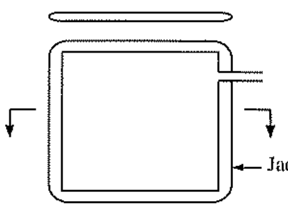


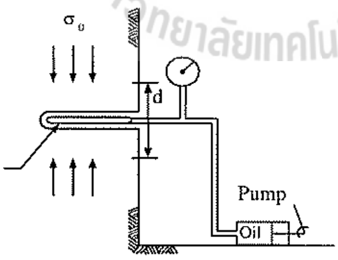


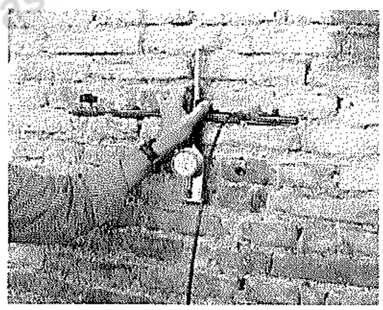
$$E = (1 + \nu)\Delta p \frac{a}{\Delta u}$$

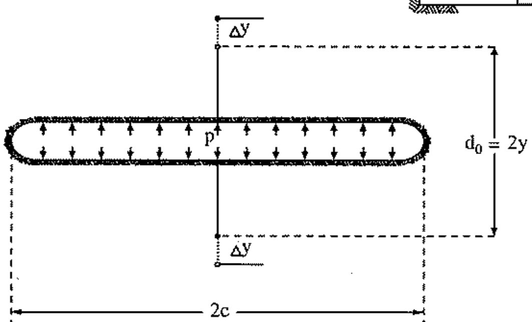
(Goodman et al., 1972)

Flat Jack Test









$$E = \frac{p(2c)}{2\Delta y}\left[(1-\nu)\left(\sqrt{1+\frac{y^2}{c^2}} - \frac{y}{c}\right) + \frac{(1+\nu)}{\sqrt{1+\frac{y^2}{c^2}}}\right]$$

(Goodman, 1989)

In-situ Direct Shear Tests

Designation: D 4554 – 02 (Reapproved 2006)

Standard Test Method for
In Situ Determination of Direct Shear Strength of Rock Discontinuities[1]

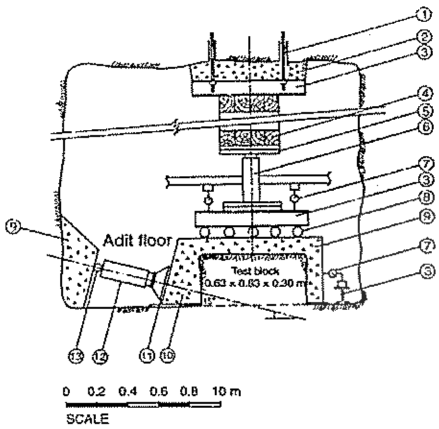


Figure 4.24 Typical set up for an *in situ* direct-shear test in an adit (Saint Simon *et al.*, 1979).
1. Rock anchor. 2. Hand-placed concrete. 3. WF beam. 4. Hardwood. 5. Steel plates. 6. 30 ton jack. 7. Dial gauge. 8. Steel rollers. 9. Reinforced concrete. 10. Bearing plate. 11. Styrofoam. 12. 50 ton jack. 13. Steel ball.

Direct Shear Tests

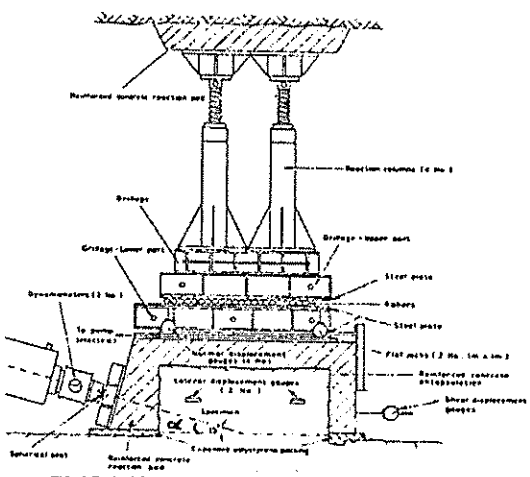
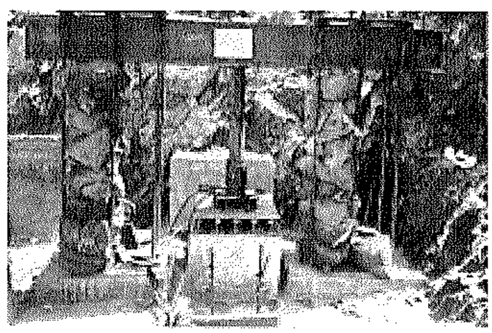
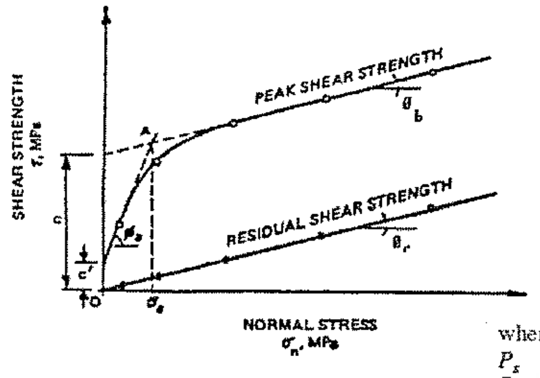


FIG. 3 Typical Arrangement of Equipment for In Situ Direct Shear Test



In situ Shear Test

Direct Shear Tests

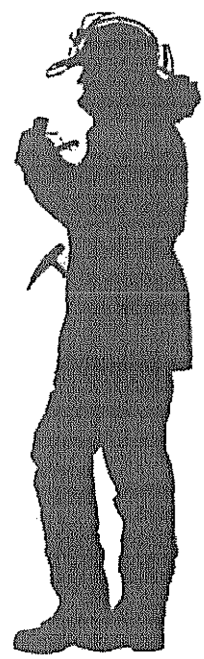


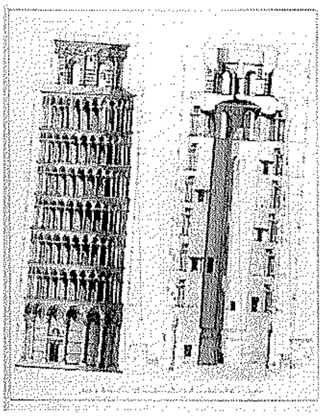
$$\text{Shear stress, } \tau = \frac{P_s}{A} = \frac{P_{sa}(\cos\alpha)}{A}$$

$$\text{Normal stress, } \sigma_n = \frac{P_n}{A} = \frac{P_{na} + P_{sa}(\sin\alpha)}{A}$$

where:

P_s = total shear force, MPa,
P_n = total normal force, MPa,
P_{sa} = applied shear force, MPa
P_{na} = applied normal force, MPa,
α = inclination of the applied shear force to the shear plane; if $\alpha = 0$, $\cos\alpha = 1$, and $\sin\alpha = 0$, and
A = area of shear surface overlap (corrected to account for shear displacement), mm.





434636 Foundations on Rock

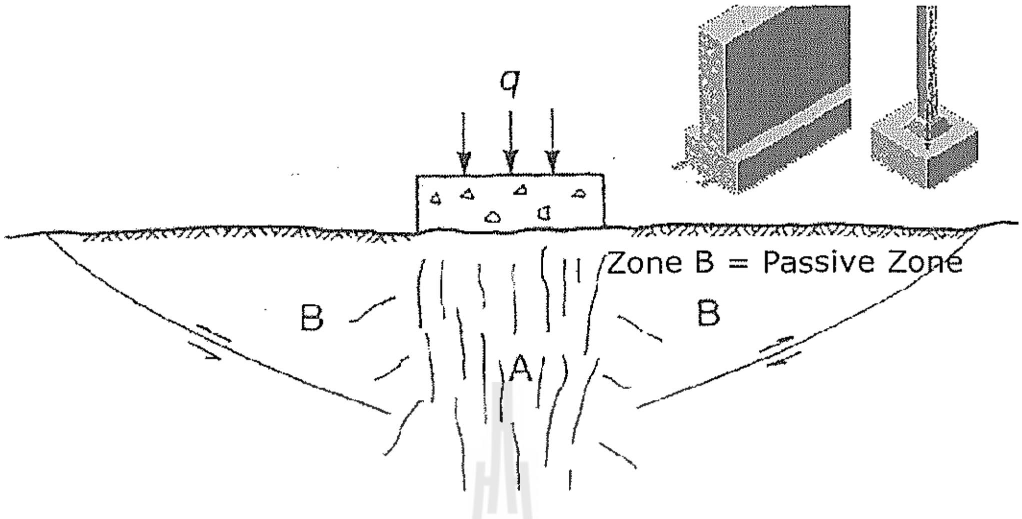
Topic 5 Bearing Capacity, Settlement & Stress Distribution

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Bearing Capacity of Foundations

1. Fracture and weathered rock
2. Shallow dipping bedding planes
3. Layered formations

Foundation on Fracture Rock



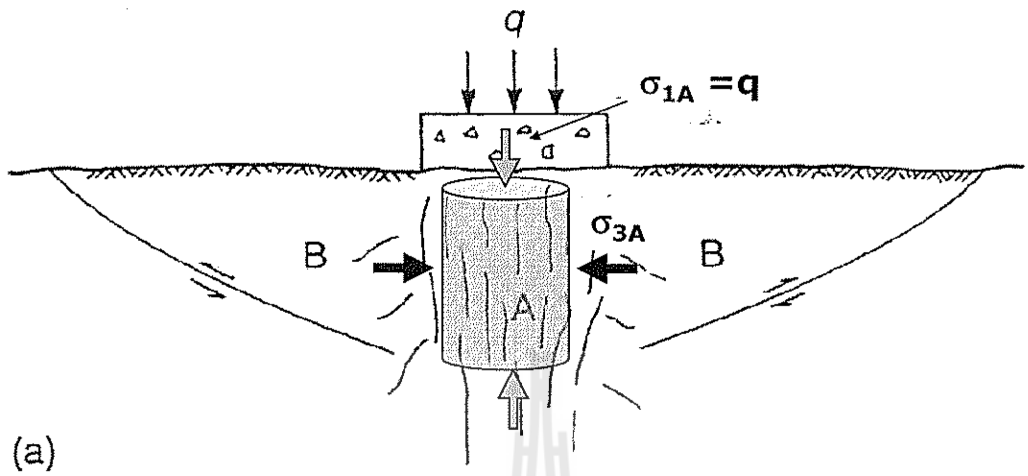
Effect of Fracture Intensity on Bearing Capacity

Peck et al. (1974)

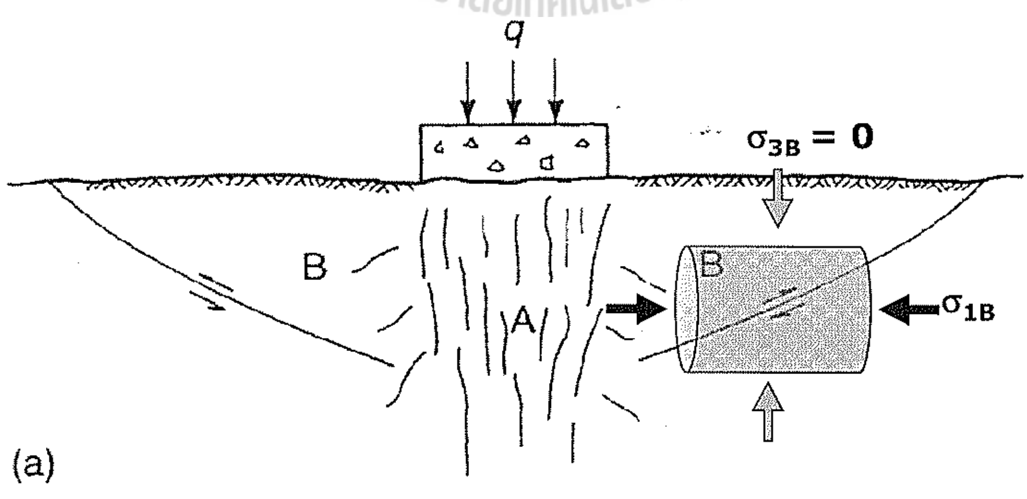
- RQD > 90% -no reduction
- RQD = 50 – 90 % - reduce baring pressure by factor of 0.25-0.7
- RQD < 50% - reduce baring pressure by factor of 0.25-0.1

Bearing Capacity of Fracture Rock

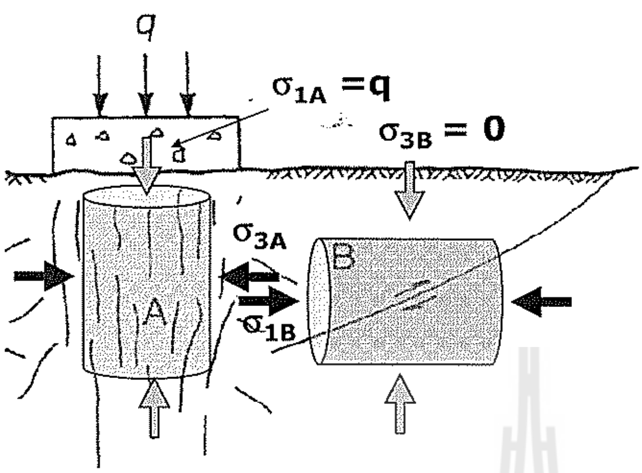
Zone A = Active Zone
Zone B = Passive Zone



Bearing Capacity of Fracture Rock



Bearing Capacity of Fracture Rock



In Zone A

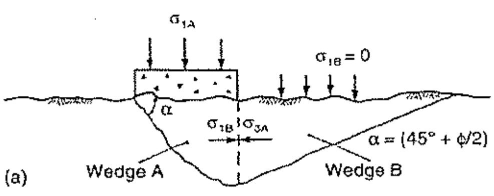
$\sigma_{1A} = q$
$\sigma_{3A} = \sigma_{1B}$

In Zone B

$\sigma_{3B} = 0$
$\sigma_{1B} = \sigma_{u(m)}$ $(= \sigma_{3A})$

Uniaxial compressive Strength of rock mass

Bearing Capacity of Fracture Rock

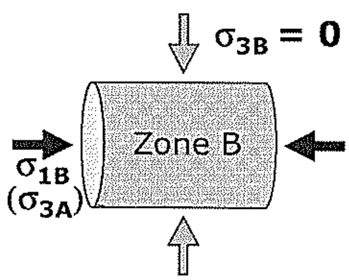


Hoek and Brown Strength Criterion

$$\sigma_1 = (m\sigma_{u(r)}\sigma_3 + s\sigma_{u(r)}^2)^{1/2} + \sigma_3$$

$$\sigma_{u(m)} = (s\sigma_{u(r)}^2)^{1/2}$$

(UCS in Rock Mass Condition)



Bering Capacity in Zone A

$\sigma_{1A} = q$ $\sigma_{3A} = \sigma_{1B} = \sigma_{u(m)}$

$$\sigma_1 = (m\sigma_{u(r)}(s\sigma_{u(r)}^2)^{1/2} + s\sigma_{u(r)}^2)^{1/2} + (s\sigma_{u(r)}^2)^{1/2}$$
$$= s^{1/2}\sigma_{u(r)}[1 + (ms^{-1/2} + 1)^{1/2}]$$

Shapes of Foundation

Allowable Bearing Capacity (q_a)

F.S. = Strength / Allowable Stress (q_a)

$$q_a = \frac{C_{f1} s^{1/2} \sigma_{u(r)} [1 + (m s^{-1/2} + 1)^{1/2}]}{FS}$$

F.S = 2-3

(Sowers, 1970)

Table 5.4 Correction factors for foundation shapes (L = length, B = width)

Foundation shape	C_{f1}	C_{f2}
Strip ($L/B > 6$)	1.0	1.0
Rectangular		
$L/B = 2$	1.12	0.9
$L/B = 5$	1.05	0.95
Square	1.25	0.85
Circular	1.2	0.7

Recessed Footing

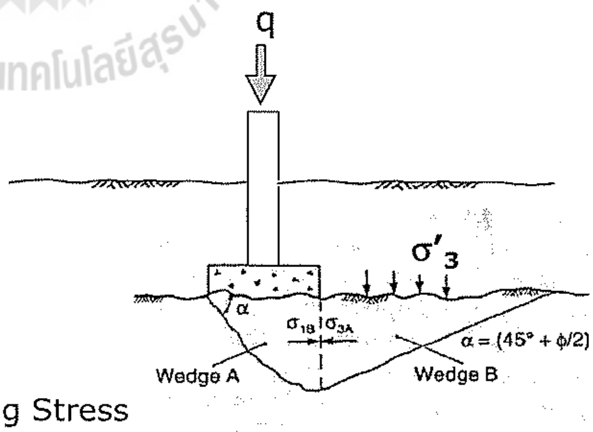
$$q_a = \frac{C_{f1}[(m \sigma_{u(r)} \sigma'_3 + s \sigma^2_{u(r)})^{1/2} + \sigma'_3]}{FS}$$

where

$$\sigma'_3 = (m \sigma_{u(r)} q_s + s \sigma^2_{u(r)})^{1/2} + q_s$$

Strength in Zone B

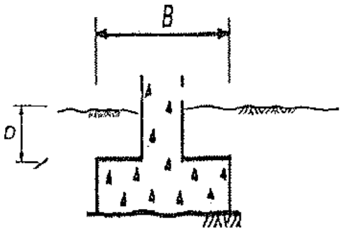
Confining Stress



Bearing Capacity on Soft Rock

Condition:

1. loading is vertical and concentric;
2. depth of embedment D is less than or equal to B;
3. foundation rock is uniform to depth below the maximum expected shear surface;
4. water level is lower than depth of the shear surface;
5. foundation rock has strength parameters defined by friction angle and cohesion;
6. friction and adhesion on the vertical sides of the footing are neglected.



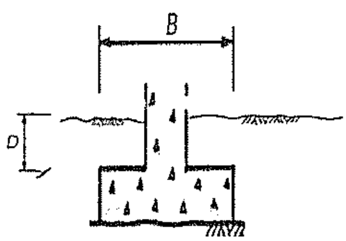
Bearing Capacity on Soft Rock

Bearing Capacity Factor for strip, square or circular footing

$$q_a = \frac{C_{f1} c N_c + C_{f2}(B\gamma_r/2)N_\gamma + \gamma D N_q}{FS}$$

Bell Solution

c = cohesion, C_{f1} and C_{f2} = correction factors
B = width of footing (or diameter)
γ = rock unit weight
D = depth of embankment
N_c, N_γ, N_q = bearing capacity factors



$$N_c = 2N_\phi^{1/2}(N_\phi + 1)$$ ← Cohesion

$$N_\gamma = 0.5N_\phi^{1/2}(N_\phi^2 - 1)$$ ← Density

$$N_q = N_\phi^2$$ ← Surcharge

$N_\phi = \tan^2(45+\phi/2)$

(Lambe and Whitman, 1969)

Correction Factors

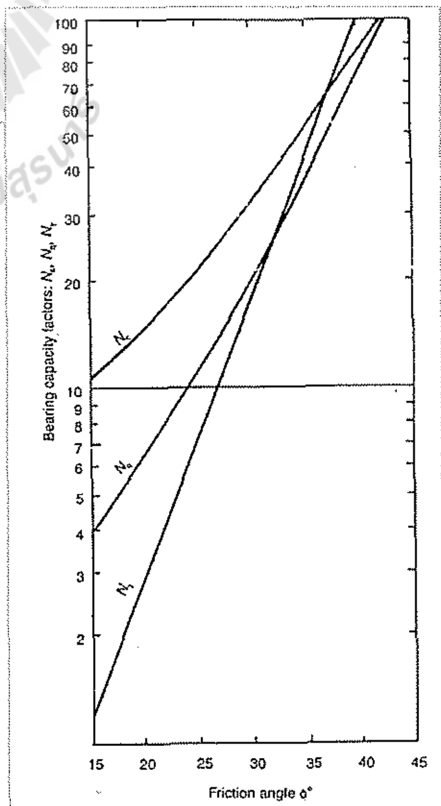
Table 5.4 Correction factors for foundation shapes (L = length, B = width)

Foundation shape	C_{f1}	C_{f2}
Strip ($L/B > 6$)	1.0	1.0
Rectangular		
$L/B = 2$	1.12	0.9
$L/B = 5$	1.05	0.95
Square	1.25	0.85
Circular	1.2	0.7

(Lambe and Whitman, 1969)

Using chart

$$N_c = 2\,N_\phi^{1/2}(N_\phi + 1)$$
$$N_\gamma = 0.5N_\phi^{1/2}(N_\phi^2 - 1)$$
$$N_q = N_\phi^2$$



(US Dept of the Navy, 1982)

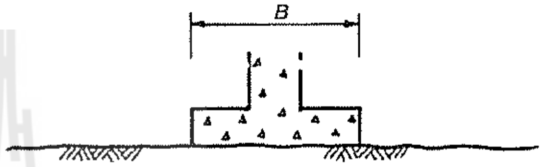
Footing on Ground

Weight of wedge of rock is ignored

D=0

$$q_a = \frac{C_{f1} c N_c + C_{f2}(B\gamma_r/2)N_\gamma + \gamma D N_q}{FS}$$

$$q_a = \frac{C_{f1} c N_c}{FS}$$

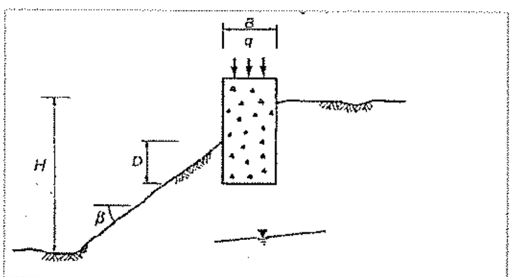


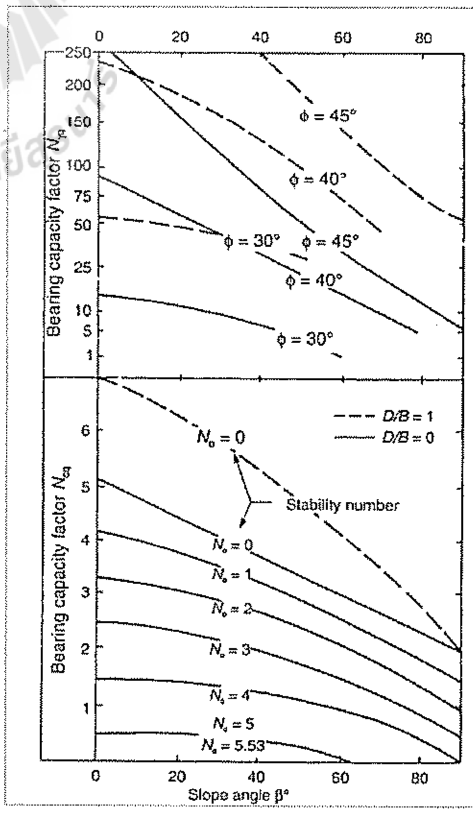
Footing on Slope Ground

$$q_a = \frac{C_{f1} c N_{cq} + (C_{f2} B\gamma_r/2)N_{\gamma q}}{FS}$$

$$N_o = \frac{\gamma_r H}{c}$$ ← Stability Number

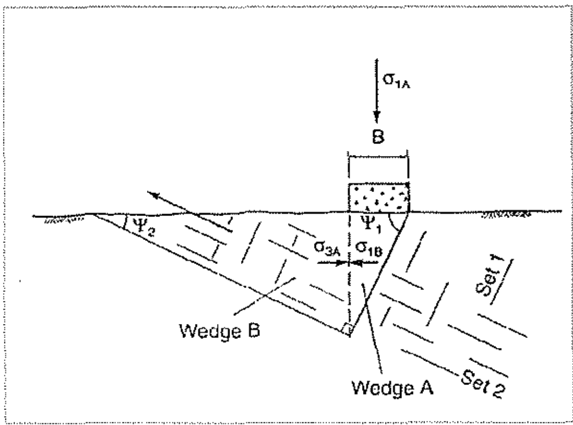
N_{cq}, $N_{\gamma q}$= bearing capacity factors





(US Dept of the Navy, 1982)

Shallow dipping bedded formation



Ladanyi and Roy, 1971

$$\sigma_{3A} = \left(\frac{\gamma B}{2\tan\psi_1}\right)N_{\phi2} + \left(\frac{c_2}{\tan\phi_2}\right)(N_{\phi2} - 1)$$

$$\sigma_{3A} = \left(q_s + \frac{\gamma B}{2}\tan\psi_1\right)N_{\phi2} + \left(\frac{c_2}{\tan\phi_2}\right)(N_{\phi2} - 1)$$

Case of surcharge load (q_s) around footing or be recessed into the ground

Allowable bearing capacity:

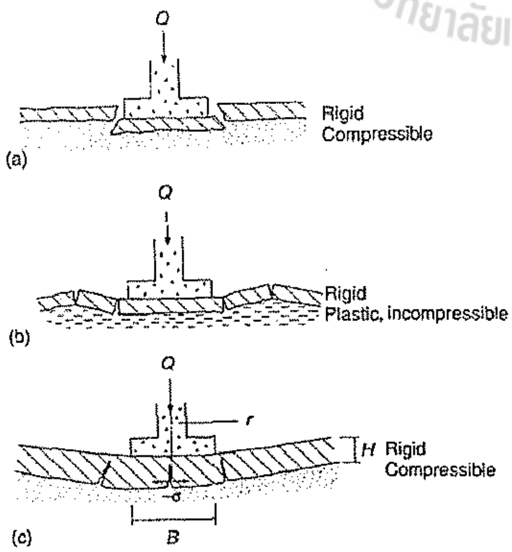
$$q_a = \frac{[\sigma_{3A}N_{\phi1} + (c_1/\tan\phi_1)(N_{\phi1} - 1)]}{FS}$$

Where:

ψ_1 = dip of joint set 1
c_1, c_2 = cohesion of joint set 1, 2

$N_{\phi1} = \tan^2(45 + \phi_1/2)$
$N_{\phi2} = \tan^2(45 + \phi_2/2)$

Layered Rock Foundation

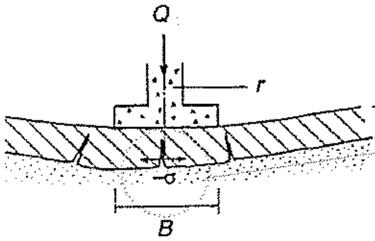


→ **Punching Failure**

→ **Bucking Failure**

→ **Bedding Failure**

Layered Rock Foundation



Tensile stress **(Roark and Young, 1970)**

$$\sigma_t = \frac{6M}{H^2}$$

M = maximum moment at center of slab

$$M = \frac{Q}{4\pi}\left[(1+\nu)\log_e\left(\frac{r}{r_0}\right)+1\right]$$

r = radius of circular slab supporting the load
H = thickness of slab
ν= Poisson's ratio
r_0 depend on relative dimensions

$$\text{if } B > H, \text{ then } r_0 = \frac{B}{2}$$

$$\text{if } B < H, \text{ then } r_0 = \left[1.6\left(\frac{B}{2}\right)^2 + H^2\right]^{1/2} - 0.675H$$

Settlement

4 types of settlements

1. From strain of intact rock, closure of fracture, compression of clay seem
2. From movement of rock block along shearing of fracture
3. From time-dependent (ductile rock)
4. Due to the subsidence (mine collapse)

Settlement on Elastic Rock

- Homogeneous and isotropic rock

$$\delta_v = \frac{C_d q B(1-\nu^2)}{E}$$

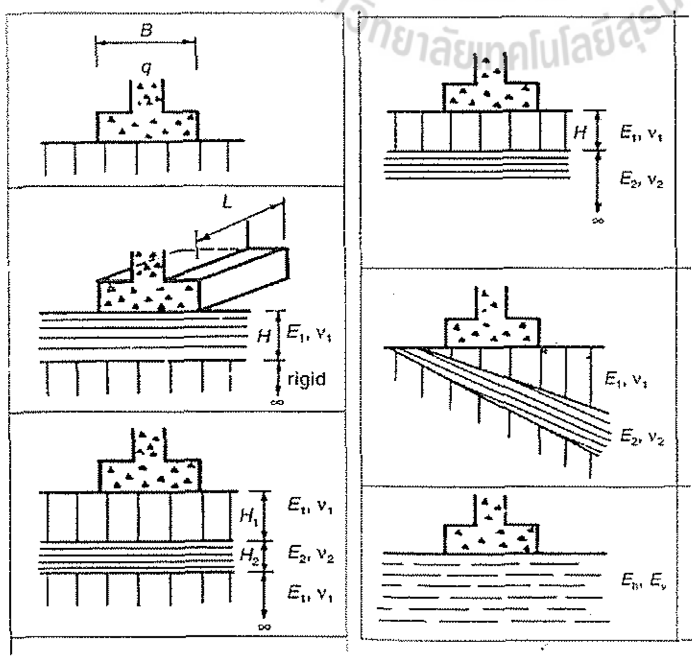
(As same as Plate Bearing Test)

C_d = rigidity factor
q= bearing pressure
B = footing width

Table 5.6 Shape and rigidity factors C_d for calculating settlements of points on loaded areas at the surface of an elastic half space (after Winterkorn and Fang, 1975)

Shape	*Center*	*Corner*	*Middle of short side*	*Middle of long side*	*Average*
Circle	1.00	0.64	0.64	0.64	0.85
Circle (rigid)	0.79	0.79	0.79	0.79	0.79
Square	1.12	0.56	0.76	0.76	0.95
Square (rigid)	0.99	0.99	0.99	0.99	0.99
Rectangle: *Length/width*					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
3	1.78	0.88	1.11	1.35	1.52
5	2.10	1.05	1.27	1.68	1.83
10	2.53	1.26	1.49	2.12	2.25
100	4.00	2.00	2.20	3.60	3.70
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

Settlement on Layered Formations



Settlement on Layered Formations

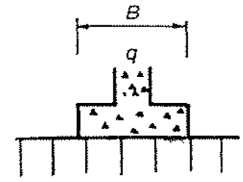
	Geological condition	Settlement calculation
	(a) Homogenous, isotropic, half space.	(a) Determine shape factor C_d from Table 5.6. (b) Calculate settlement using equation 5.18.

Table 5.6 Shape and rigidity factors C_d for calculating settlements of points on loaded areas at the surface of an elastic half space (after Winterkorn and Fang, 1975)

Shape	*Center*	*Corner*	*Middle of short side*	*Middle of long side*	*Average*
Circle	1.00	0.64	0.64	0.64	0.85
Circle (rigid)	0.79	0.79	0.79	0.79	0.79
Square	1.12	0.56	0.76	0.76	0.95
Square (rigid)	0.99	0.99	0.99	0.99	0.99
Rectangle: *Length/width*					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
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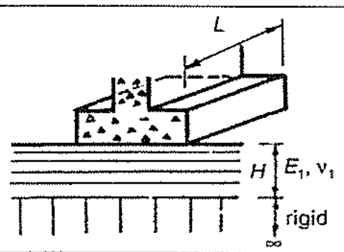
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

$$\delta_v = \frac{C_d q B(1-\nu^2)}{E}$$

Settlement on Layered Formations

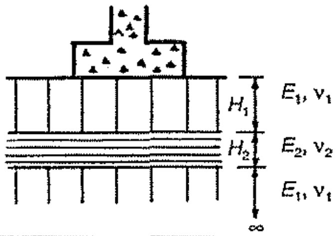
	(b) Compressible layer on rigid base.	(a) Determine ratios *H/B*, *L/B*. (b) Determine shape factor C'_d from Table 5.7. (c) Calculate settlement using equation 5.18

$$\delta_v = \frac{C_d q B(1-\nu^2)}{E}$$

Table 5.7 Values of the shape factor C'_d for settlement of the center of a uniformly loaded area on an elastic layer underlain by a rigid base (Winterkorn and Fang, 1975)

H/B	*Circle diameter = B*	*Rectangle shape*						
		L/B = 1	*L/B* = 1.5	*L/B* = 2	*L/B* = 3	*L/B* = 5	*L/B* = 10	*L/B* = ∞ *infinite strip*
0.1	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
0.25	0.24	0.24	0.23	0.23	0.23	0.23	0.23	0.23
0.5	0.48	0.48	0.47	0.47	0.47	0.47	0.47	0.47
1.0	0.70	0.75	0.81	0.83	0.83	0.83	0.83	0.83
1.5	0.80	0.86	0.97	1.03	1.07	1.08	1.08	1.08
2.5	0.88	0.97	1.12	1.22	1.33	1.39	1.40	1.40
3.5	0.91	1.01	1.19	1.31	1.45	1.56	1.59	1.60
5.0	0.94	1.05	1.24	1.38	1.55	1.72	1.82	1.83
∞	1.00	1.12	1.36	1.52	1.78	2.10	2.53	∞

Settlement on Layered Formations



(c) Compressible bed within stiffer formation $E_1 > E_2$.

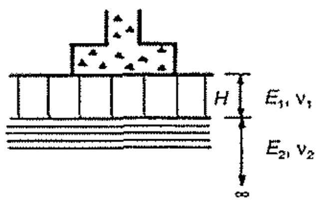
(a) Determine ratios $(H_1+H_2)/B$, L/B.
(b) Calculate weighted modulus E for upper two beds $E = (E_1H_1 + E_2H_2)/(H_1 + H_2)$.
(c) Determine shape factor C'_d for ratio $(H_1 + H_2)/B$ from Table 5.7.
(d) Calculate settlement using equation 5.18

Table 5.7 Values of the shape factor C'_d for settlement of the center of a uniformly loaded area on an elastic layer underlain by a rigid base (Winterkorn and Fang, 1975)

H/B	Circle diameter = B	Rectangle shape						
		$L/B = 1$	$L/B = 1.5$	$L/B = 2$	$L/B = 3$	$L/B = 5$	$L/B = 10$	$L/B = \infty$ infinite strip
0.1	0.09	0.09	0.09	0.09	0.09	0.09	0.09	0.09
0.25	0.24	0.24	0.23	0.23	0.23	0.23	0.23	0.23
0.5	0.48	0.48	0.47	0.47	0.47	0.47	0.47	0.47
1.0	0.70	0.75	0.81	0.83	0.83	0.83	0.83	0.83
1.5	0.80	0.86	0.97	1.03	1.07	1.08	1.08	1.08
2.5	0.88	0.97	1.12	1.22	1.33	1.39	1.40	1.40
3.5	0.91	1.01	1.19	1.31	1.45	1.56	1.59	1.60
5.0	0.94	1.05	1.24	1.38	1.55	1.72	1.82	1.83
∞	1.00	1.12	1.36	1.52	1.78	2.10	2.53	∞

$$\delta_v = \frac{C_d q B(1-\nu^2)}{E}$$

Settlement on Layered Formations



(d) Stiff bed overlying compressible formation $E_1 > E_2$.

(a) Determine ratios H/B, E_1/E_2.
(b) Determine correction factor a from Table 5.8.
(c) Determine shape factor C_d from Table 5.6.
(d) Calculate approximate settlement from equation 5.18 using elastic parametes E_2, ν_2 for overall foundation.
(e) Calculate actual settlement using equation 5.19.

Table 5.8 Elastic distortion settlement correction factor a, at the center of a circular uniformly loaded area on an elastic layer E_1 underlain by a less stiff elastic material E_2, of infinite depth; $\nu_1 = \nu_2 = 0.4$ (Winterkorn and Fang, 1975)

H/B	E_1/E_2				
	1	2	5	10	100
0	1.0	1.00	1.00	1.00	1.00
0.1	1.0	0.972	0.943	0.923	0.76
0.25	1.0	0.885	0.779	0.699	0.431
0.5	1.0	0.747	0.566	0.463	0.228
1.0	1.0	0.627	0.399	0.287	0.121
2.5	1.0	0.55	0.274	0.175	0.058
5.0	1.0	0.525	0.238	0.136	0.036
∞	1.0	0.500	0.200	0.100	0.010

$$\delta_v = a\delta_\infty$$

$$\delta_\infty = \frac{C_d q B(1-\nu^2)}{E}$$

Table 5.6 Shape and rigidity factors C_d for calculating settlements of points on loaded areas at the surface of an elastic half space (after Winterkorn and Fang, 1975)

Shape	Center	Corner	Middle of short side	Middle of long side	Average
Circle	1.00	0.64	0.64	0.64	0.85
Circle (rigid)	0.79	0.79	0.79	0.79	0.79
Square	1.12	0.56	0.76	0.76	0.95
Square (rigid)	0.99	0.99	0.99	0.99	0.99
Rectangle: Length/width					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
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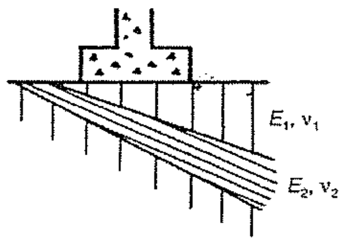
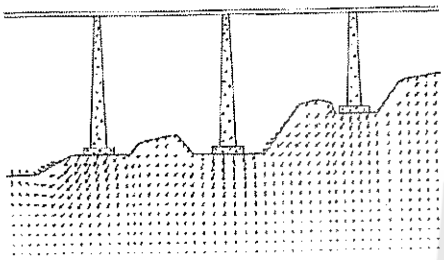
100	4.00	2.00	2.20	3.60	3.70
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

Settlement on Layered Formations

	(e) Inclined, non-uniform bed of compressible rock.	Use numerical analysis to accurately model foundation geometry. (Fig. 5.13)



Example for numerical analysis using FLAC

Settlement on Layered Formations

	(f) Transversly isotropic rock.	Use equations 5.20a–c, 5.21 and 5.22a–d.

Transversely isotropic rock parameters

E_z = vertical deformation modulus
E_h = horizontal deformation modulus
G_{hv} = shear modulus b/w horizontal and vertical plane

ν_{hh} Poisson's ratio for horizontal stress on the complimentary horizontal strain
ν_{hz} Poisson's ratio for horizontal stress on vertical strain
ν_{zh} Poisson's ratio for vertical stress on the horizontal strain

Settlement on Transversely isotropic rock

$$\delta_z = \frac{Q(c' + G_{hz})de(e^2 - \beta^2)}{2bG_{hz}[c' + d(e + \beta)^2][c' + d(e - \beta)^2]} \quad (5.20a)$$

← **Settlement**

β^2 negative:

$$\delta_z = \frac{Qe(ad)^{1/2}}{2b(ad - c'^2)} \quad (5.20b)$$

$\beta^2 = 0$:

$$\delta_z = \frac{Q(c' + G_{hz})de^3}{2bG_{zh}(c' + de^2)^2} \quad (5.20c)$$

The appropriate equation to use is defined by:

$$\beta^2 = \frac{ad - c'^2 - 2c'G_{zh} - 2G_{zh}(ad)^{1/2}}{4G_{zh}d} \quad (5.21)$$

Settlement on Transversely isotropic rock

The appropriate equation to use is defined by:

$$\beta^2 = \frac{ad - c'^2 - 2c'G_{zh} - 2G_{zh}(ad)^{1/2}}{4G_{zh}d} \quad (5.21)$$

← **β^2 Factor**

The factors a, c', d, and e^2 are defined by the following equations:

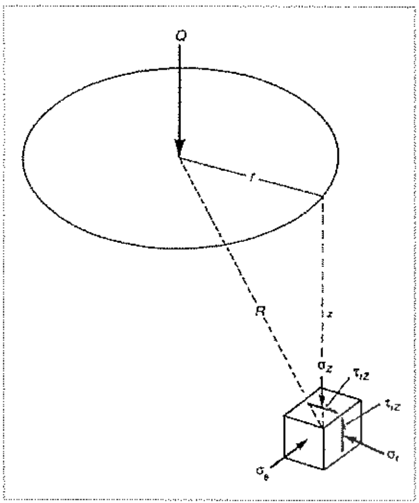
$$a = \frac{E_h(1 - \nu_{hz}\nu_{zh})}{(1 + \nu_{hh})(1 - \nu_{hh} - 2\nu_{hz}\nu_{zh})} \quad (5.22a)$$

$$c' = \frac{E_h\nu_{zh}}{1 - \nu_{hh} - 2\nu_{hz}\nu_{zh}} \quad (5.22b)$$

$$d = \frac{E_h\nu_{zh}(1 - \nu_{hh})}{\nu_{hz}(1 - \nu_{hh} - 2\nu_{hz}\nu_{zh})} \quad (5.22c)$$

$$e^2 = \frac{ad - c'^2 - 2c'G_{zh} + 2G_{zh}(ad)^{1/2}}{4G_{zh}d} \quad (5.22d)$$

Stress Distribution in Isotropic Rock



Boussinesq's Equations

$$\sigma_z = \frac{3Q}{2\pi}\frac{z^3}{R^5} = \frac{3Q}{2\pi z^2}\frac{1}{\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$$

$$\sigma_r = \frac{Q}{2\pi}\left[\frac{3zr^2}{R^5} - \frac{1-2\nu}{R(R+z)}\right]$$

$$\sigma_\theta = \frac{Q}{2\pi}(1-2\nu)\left[\frac{1}{R(R+z)} - \frac{z}{R^3}\right]$$

$$\tau_{rz} = \frac{3Q}{2\pi}\frac{z^2 r}{R^5}$$

$$\tau_{\theta z} = \tau_{r\theta} = 0$$

Cylindrical Coordinate

(Boussinesq, 1885)

Distributed Loads

$$\sigma_z = qI_z$$

Circular Load Area

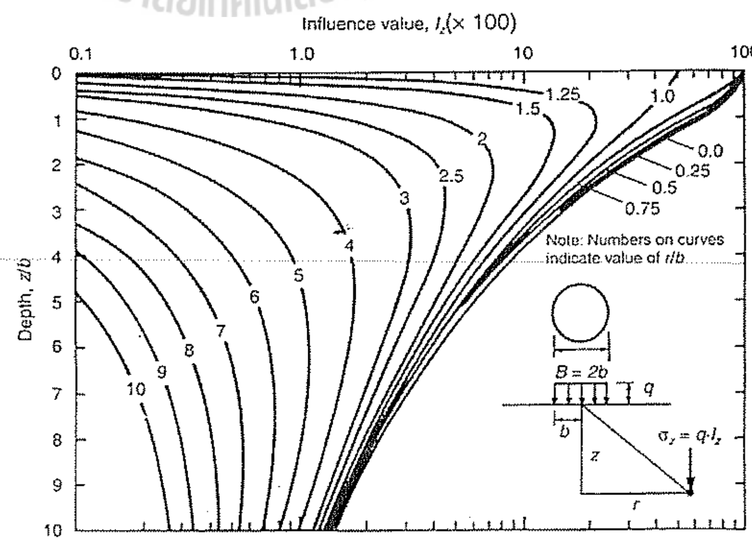


Figure 5.18 Influence diagram for vertical normal stress σ_z at various points within an elastic half space under a uniformly loaded circular area (Winterkorn and Fang, 1975).

Rectangular Load Area

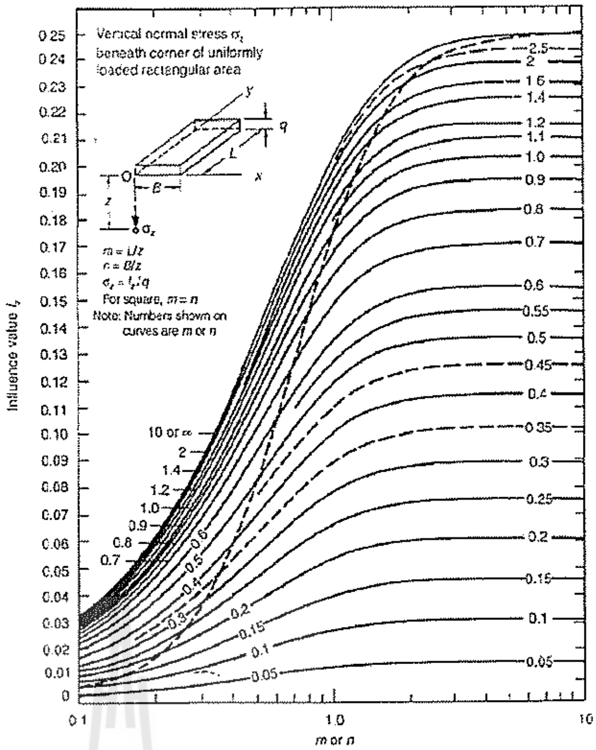


Figure 5.19 Influence diagram for vertical normal stress at a point within an elastic half space beneath the corner of a uniformly loaded rectangular area (Winterkorn and Fang, 1975).

Line Load

$$\sigma_r = \frac{2Q\cos\theta}{\pi r}$$

$$\sigma_\theta = \tau_{r\theta} = 0$$

Q = line load (N/m)
θ= angle form vertical
R = radius distance from Q

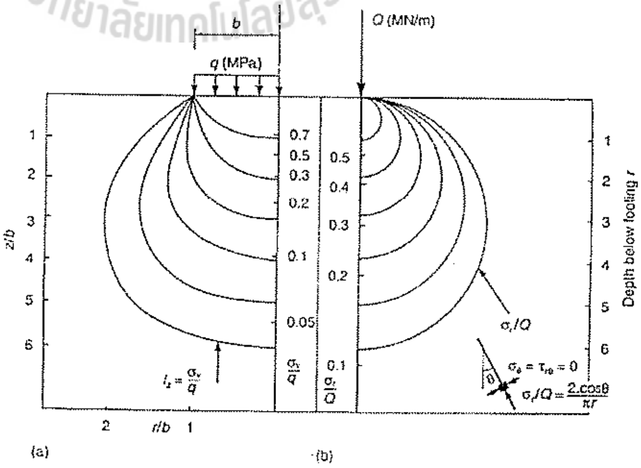
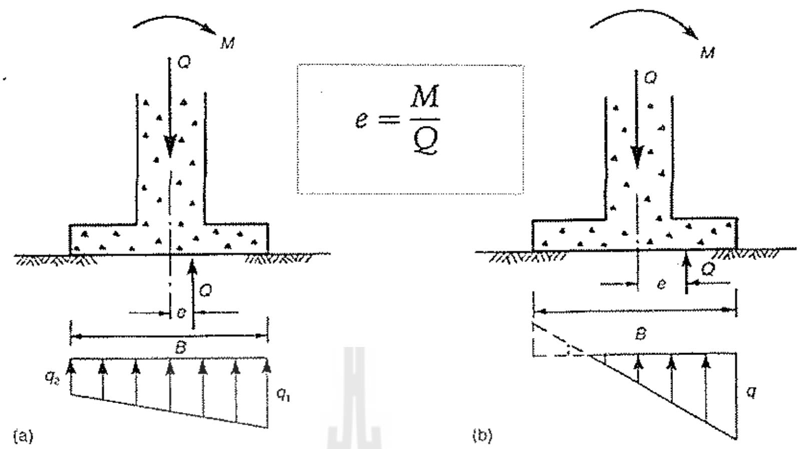


Figure 5.21 Stress contours for footings located on isotropic linear elastic half-space: (a) vertical normal stresses beneath uniformly loaded circular area, radius b; and (b) radial stresses beneath line load.

Eccentrically Load Footing

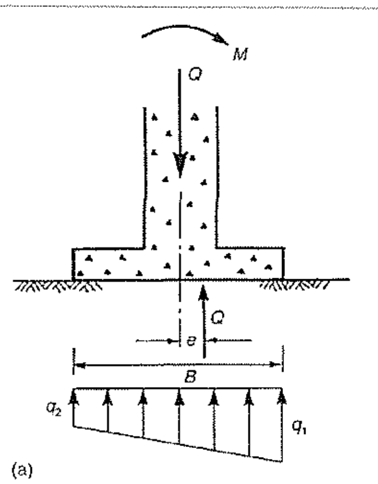


$$e = \frac{M}{Q}$$

Figure 5.26 Stress conditions produced by footings subjected to overturning: (a) $e < B/6$; (b) $e > B/6$.

Eccentrically Load Footing

e < B/6



Strip Footing

$$q_1 = \frac{Q}{B}\left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{Q}{B}\left(1 - \frac{6e}{B}\right)$$

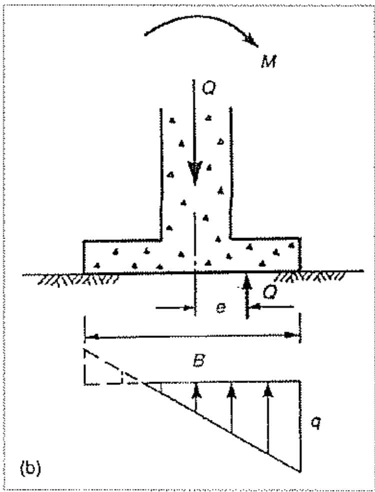
Rectangular Footing

$$q_1 = \frac{Q}{BL}\left(1 + \frac{6e}{B}\right)$$

$$q_2 = \frac{Q}{BL}\left(1 - \frac{6e}{B}\right)$$

Eccentrically Load Footing

e > B/6

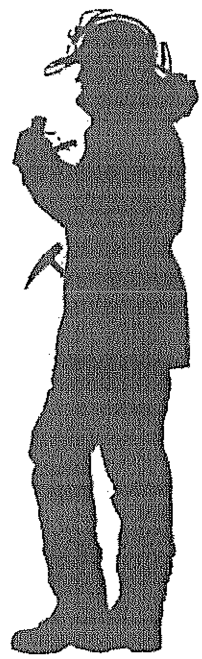


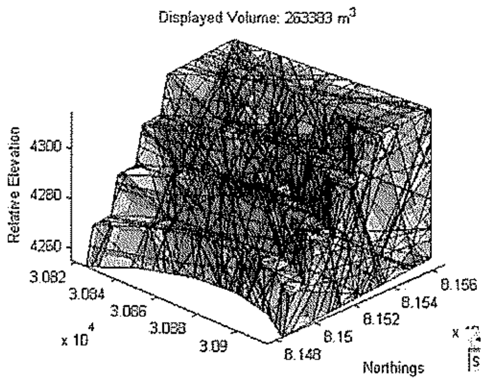
Strip Footing

$$q = \frac{2Q}{3(B/2 - e)}$$

Rectangular Footing

$$q = \frac{2Q}{3L(B/2 - e)}$$





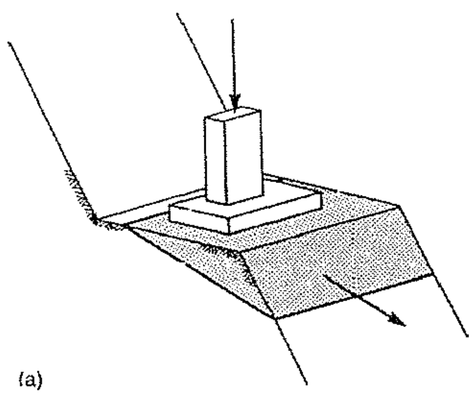
434636 Foundations on Rock

Topic 6 Stability of Foundation

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

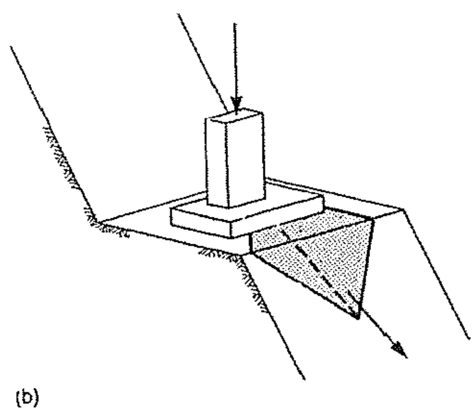
Stability of Sliding Block

Planar Sliding Failure



(a)

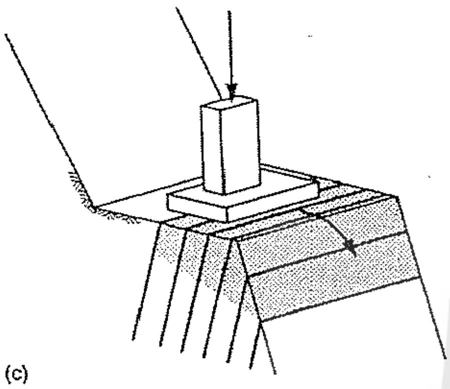
Wedge Sliding Failure



(b)

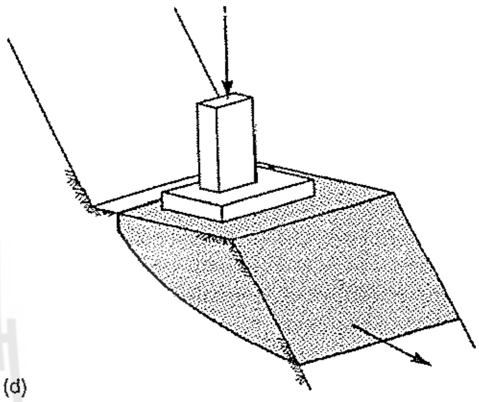
Stability of Sliding Block

Toppling Failure



(c)

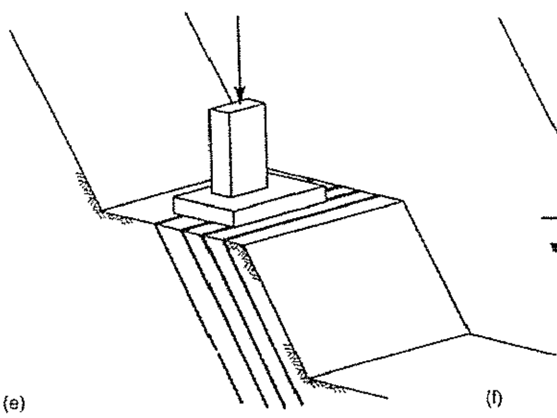
Circular Failure



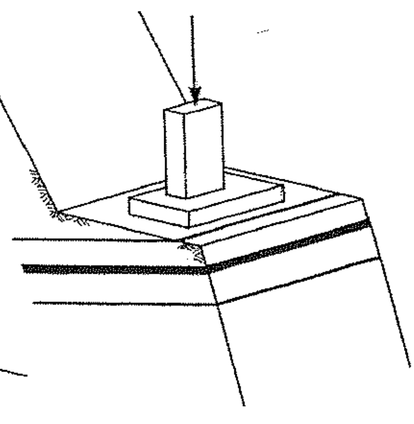
(d)

Stability of Sliding Block

Stable Condition



(e)



(f)

Stability of Sliding Block

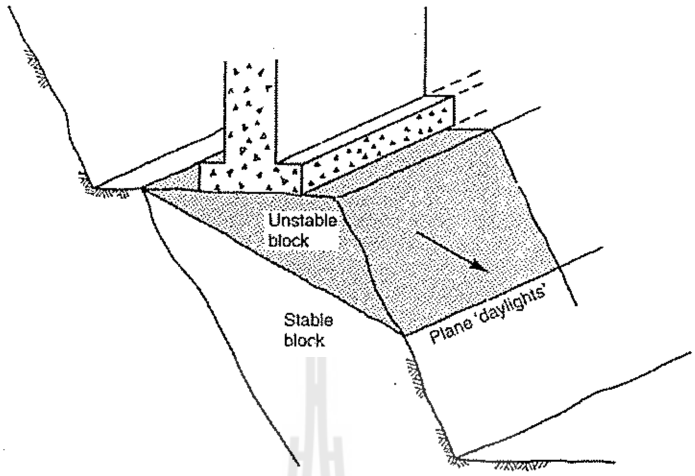


Figure 6.2 Stability of sliding block related to dip of sliding surface.

Deterministic Stability Analysis

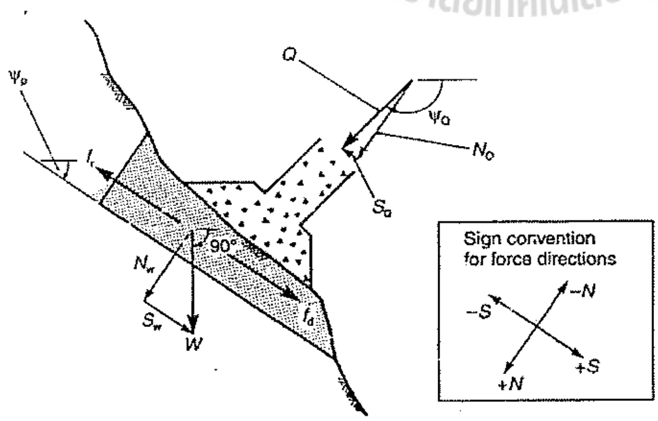


Figure 6.3 Resolution of forces in foundation to determine normal N and shear S components on potential failure surface.

$$FS = \frac{f_r}{f_d}$$

← Resisting Force

← Displacing / Driving Force

Deterministic Stability Analysis

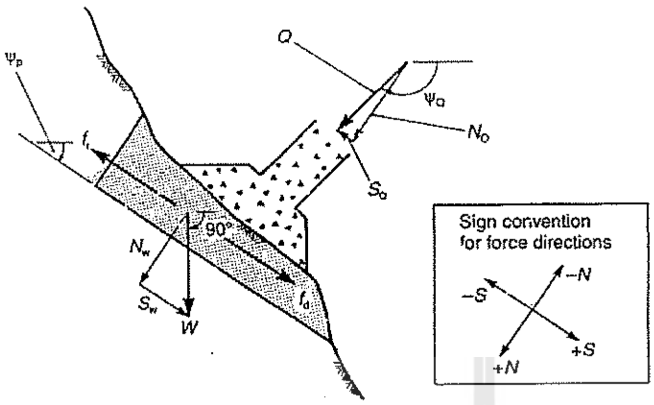


Figure 6.3 Resolution of forces in foundation to determine normal N and shear S components on potential failure surface.

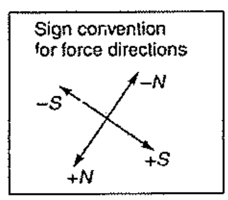
Normal force, $N_Q = Q\sin(\psi_Q - \psi_p)$ (6.2)

Shear force, $S_Q = Q\cos(\psi_Q - \psi_p)$ (6.3)

where ψ_p is the dip of sliding surface ($0° < \psi_p < 90°$).

Deterministic Stability Analysis

For Morh-Coulomb Material



$$\tau = c + \frac{\Sigma N}{A}\tan\phi$$ ← Shear Stress

or

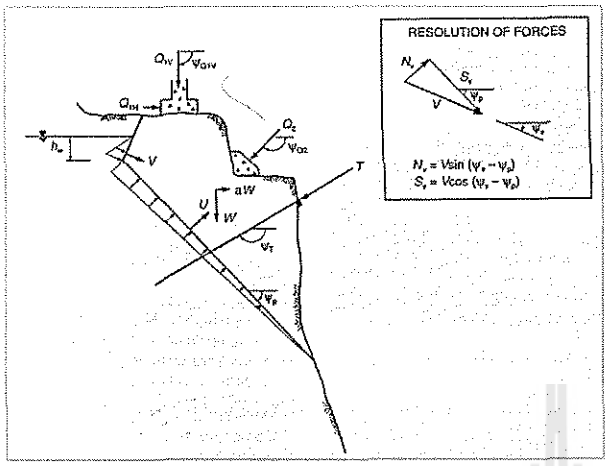
$$f_r = cA + \Sigma N \tan\phi$$ ← Shear Force

$$\Sigma N = W\sin(90 - \psi_p) + Q\sin(\psi_Q - \psi_p)$$ ← Total Normal Force

$$f_d = \Sigma S = W\cos(90 - \psi_p) + Q\cos(\psi_Q - \psi_p)$$ ← Total Displacing Force

$$FS = \frac{\tan\phi}{\tan\psi_p}$$ ← If cohesion = 0 and vertical foundation load

Deterministic Stability Analysis



Foundation Load Q_1 and Q_2

Water Force U and V

$$U = \frac{A h_w \gamma_w}{2}$$

$$V = \frac{h_w^2 L \gamma_w}{2 \cos \psi_v}$$

Earthquake Force

Pseudo-static, aW

Artificial Support Force T

Optimum plunge

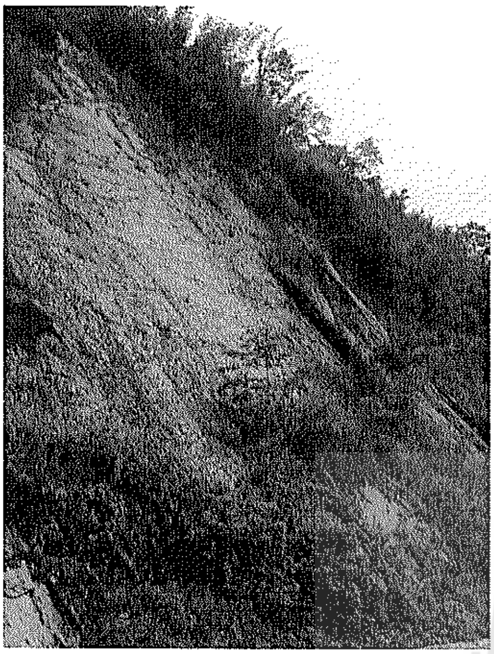
$\psi_{Topt} \sim (180 + \psi_p - \phi)$

Plane Failure

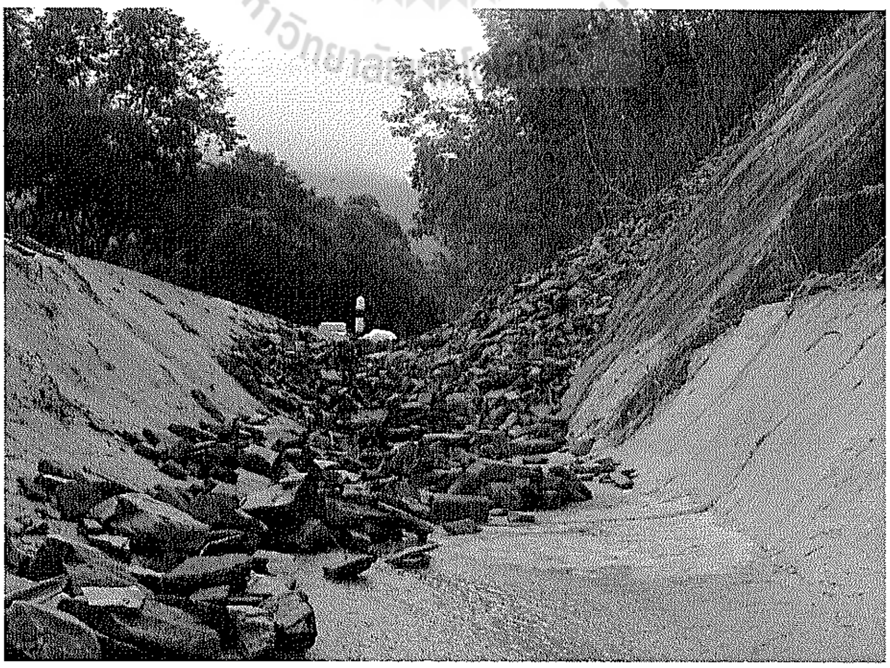
Plane Failure

Plane Failure

Plane Failure



Plane Failure



General Condition for Plane Failure

- Rare
- Strike of sliding plane // strike of slope face (± 20 degrees)
- Daylight ($\psi_f > \psi_p$)
- Overcome friction angle ($\psi_p > \phi$)
- Upper end of sliding surface intersects upper slope / tension crack
- Release surface

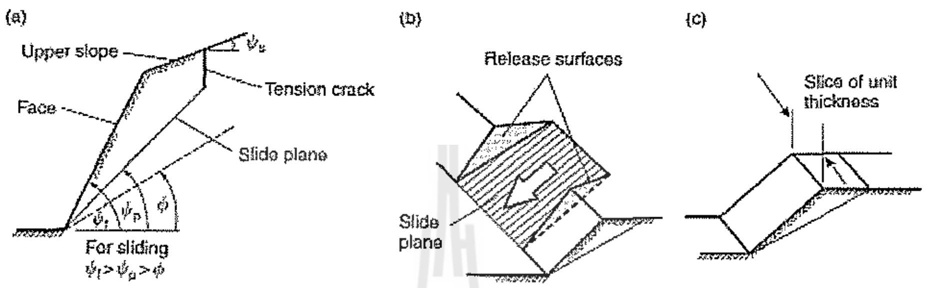


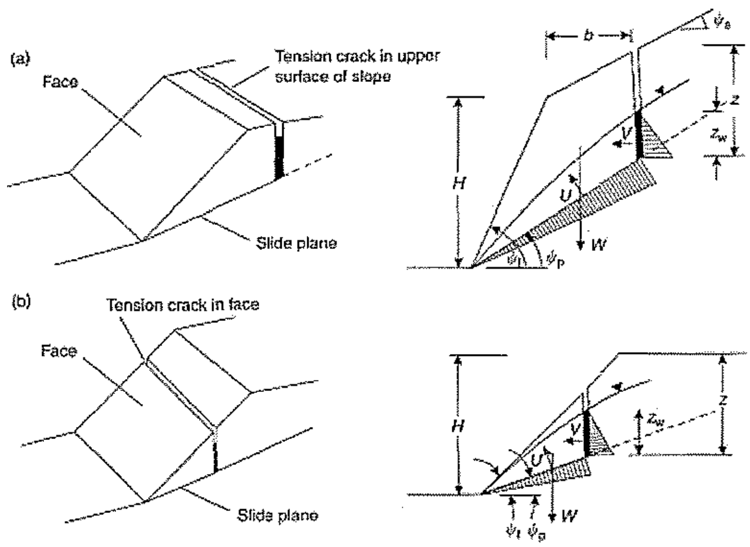
Figure 6.2 Geometry of slope exhibiting plane failure: (a) cross-section showing planes forming a plane failure; (b) release surfaces at ends of plane failure; (c) unit thickness slide used in stability analysis.

Plane Failure Analysis

The geometry of the slope is defined two cases:

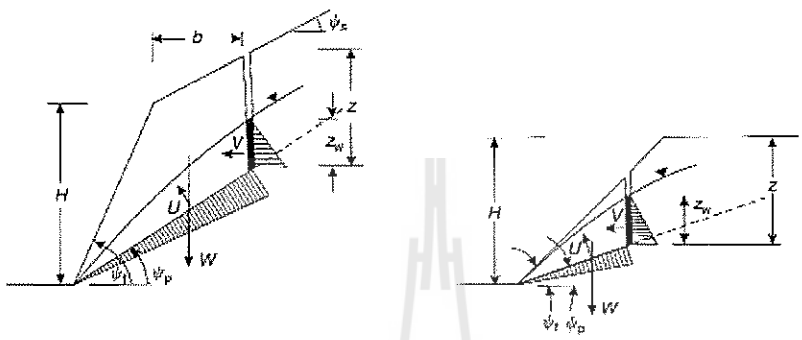
(a) A slope having a tension crack in its upper surface

(b) A slope with a tension crack in its face.

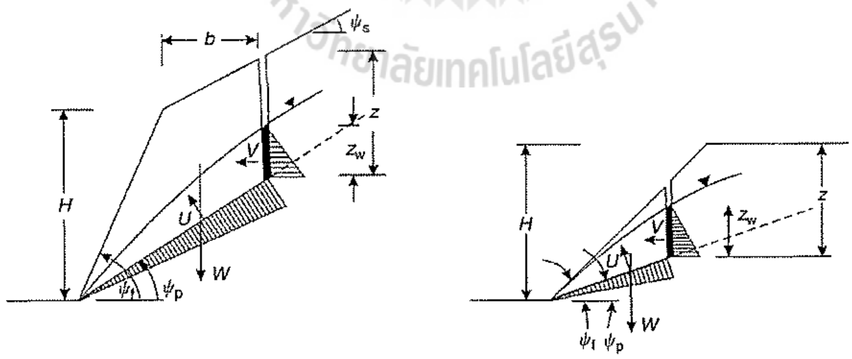


Assumptions Required for Analysis

- Both sliding surface and tension crack strike parallel to the slope surface.
- The tension crack is vertical and is filed with water to a depth z_w.
- Water in sliding surface and tension crack subjected to atmospheric pressure.
- All forces act through the centroid of the sliding mass.
- Using Coulomb criterion, $\tau = c + \sigma \text{ Tan } \phi$
- Release surfaces is no resistance to sliding.



Symbols



A	=	area of sliding block	ψ_f	=	dip angle of slope face
U	=	uplift force	ψ_p	=	dip angle of failure plane
V	=	water pressure in tension crack	ψ_s	=	dip angle of upper slope face
H	=	slope height	γ_w	=	unit weight of water
b	=	horizontal distance b/w slope crest & tension crack	γ_r	=	unit weight of rock
			z	=	depth of tension crack
W	=	weight of sliding block	z_w	=	vertical depth of filled water

F.S. Calculations

$$F.S. = \frac{\text{Resisting Force}}{\text{Driving Force}}$$

$$F.S. = \frac{cA + (W.\cos\psi_p - U - V.\sin\psi_p)\tan\phi}{W.\sin\psi_p + V.\cos\psi_p}$$

where

$A = (H + b.\tan\psi_s - z).\ \mathrm{cosec}\ \psi_p$

$U = \frac{1}{2}\gamma_w . z_w (H + b.\tan\psi_s - z) . \mathrm{cosec}\ \psi_p$

$V = \frac{1}{2}\gamma_w . z^2_w$

F.S. Calculations

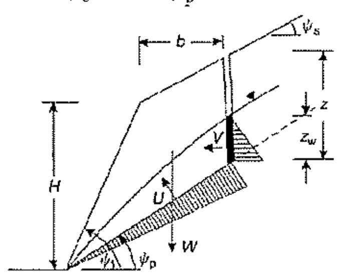
For the tension crack in the upper slope surface

$W = \gamma_r[(1\text{-}\cot\psi_f\ \tan\psi_p)(bH + \frac{1}{2} H^2 \cot\psi_f) + \frac{1}{2} b^2 (\tan\psi_s - \tan\psi_p)]$

(for ψ_s= dip angle of upper slope face)

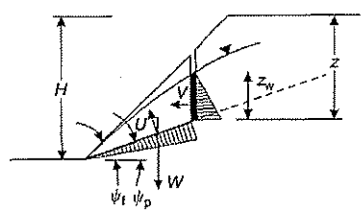
$W = \frac{1}{2}\gamma_r H^2 [(1 - (z/H)^2) \cot\psi_p - \mathrm{Cot}\ \psi_f]$

(for ψ_s= 0, upper slope face is horizontal)



For the tension crack in the slope face

$W = \frac{1}{2}\gamma_r H_2 [(1 - z/H)^2\ \cot\psi_p (\cot\psi_p . \tan\psi_f - 1)]$



Analysis of Failure on a Rough Plane

For dry slope, U=V=0

$$F.S. = \frac{\tau A}{W \sin \psi_p}$$

$$F.S. = \frac{\sigma \tan(\phi + JRC \log_{10}(\sigma_j / \sigma))A}{W \sin \psi_p}$$

Sub $\sigma = \frac{W \cos \psi_p}{A}$ in Equation

$$F.S. = \frac{\tan(\phi + JRC \log_{10}(\sigma_j / \sigma))}{\tan \psi_p}$$ Barton Criterion

$$F.S. = \frac{\tan(\phi + i)}{\tan \psi_p}$$ Patton Criterion

Reinforcement of a Slope

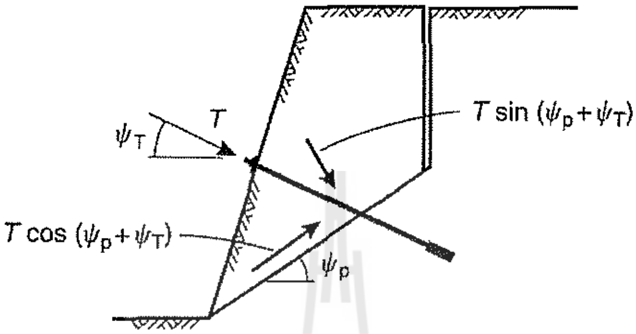
-Reinforcement with Tensioned Anchors

-Reinforcement with Fully Grouted Untensioned Dowels

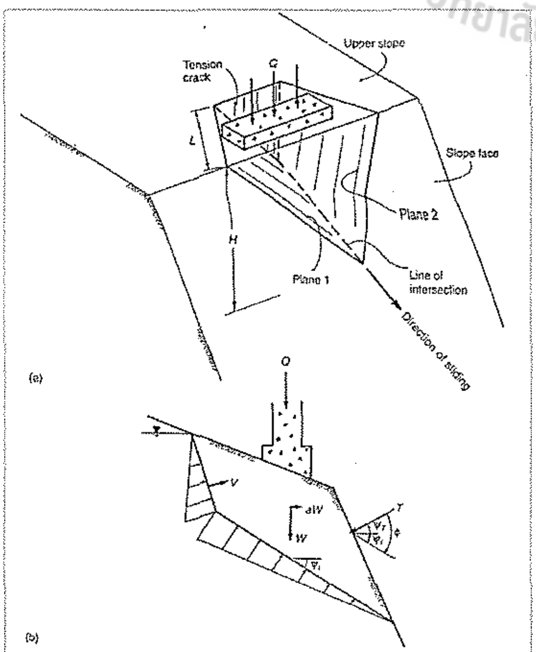
-Reinforcement with Buttresses

Reinforcement with Tensioned Anchors

$$F.S. = \frac{cA + (W.\cos\psi_p - U - V.\sin\psi_p + T.\cos(\psi_T + \psi_p))\tan\phi}{W.\sin\psi_p + V.\cos\psi_p - T.\sin(\psi_T + \psi_p)}$$



Stability of Wedge Blocks



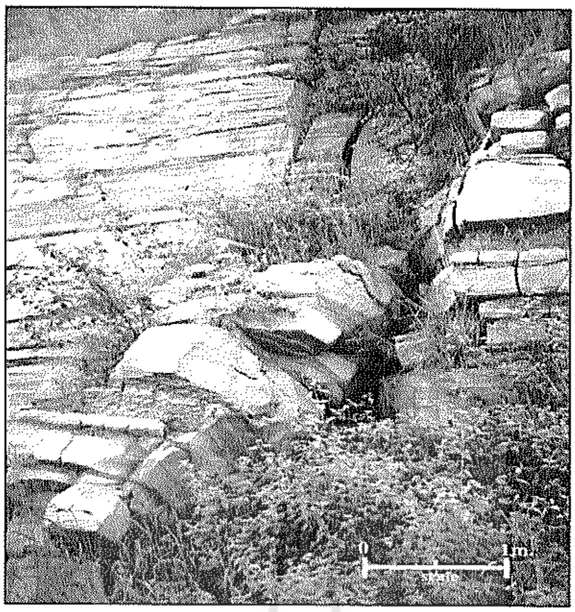
$$FS = \frac{f_r}{f_d}$$ ← Resisting Force / ← Displacing / Driving Force

$$f_r = N_1\tan\phi_1 + N_2\tan\phi_2 + c_1A_1 + c_2A_2$$

$$f_d = f'(W, T, E, V)$$

W = weight of wedge
T = tension supported force
E = external load (Q)
V = water force

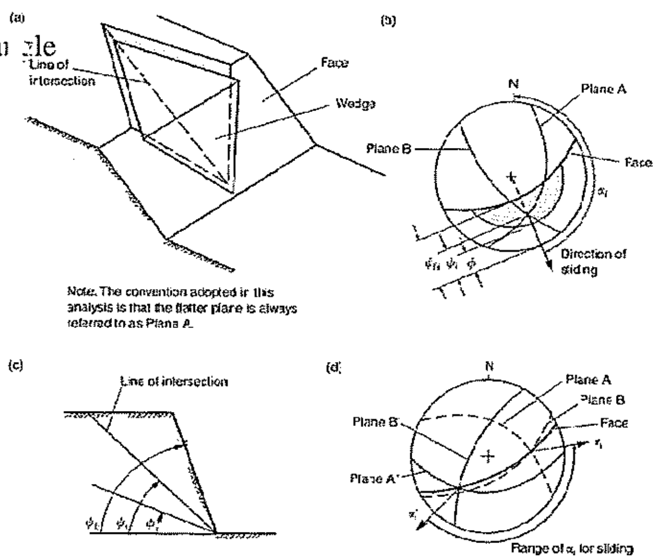
Wedge Failure



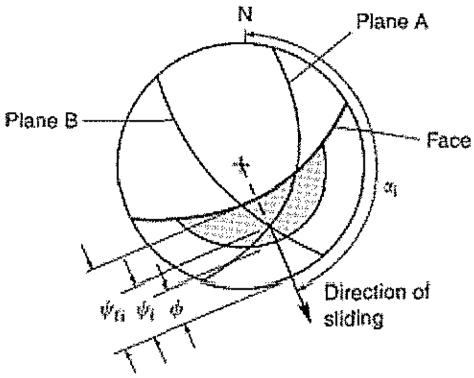
General Condition for Wedge Failure

General Condition for Wedge Failure

- Two plane always intersect in a line (trend α_i and plunge ψ_i)
- Daylight and overcome friction angle ($\psi_{fi} > \psi_i > \phi$)
- Line of intersection is between α_i and α_i'



Trend α_i and Plunge ψ_i



$$\alpha_i = \tan^{-1}\left(\frac{\tan\psi_A \cos\alpha_A - \tan\psi_B \cos\alpha_B}{\tan\psi_B \sin\alpha_B - \tan\psi_A \sin\alpha_A}\right)$$

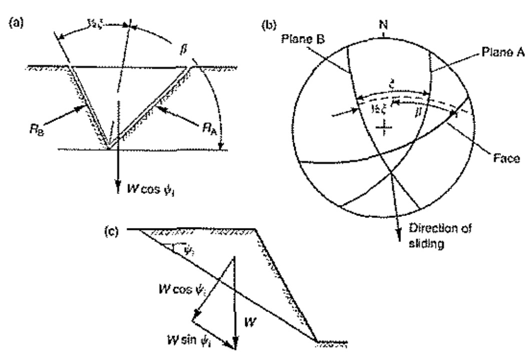
$$\psi_i = \tan\psi_A \cos(\alpha_A - \alpha_i) = \tan\psi_B \cos(\alpha_B - \alpha_i)$$

Analysis of Wedge Failure

- The F.S. of wedge assuming that sliding is resisted by <u>friction only</u> and that the friction angle ϕ is the same for both planes

$$F.S. = \frac{(R_A + R_B)\tan\phi}{W \sin\psi_i}$$

Where R_A and R_B are the normal reactions provided by planes A and B



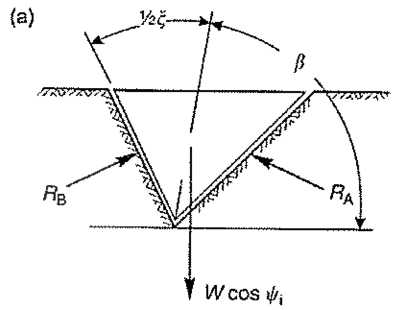
Analysis of Wedge Failure

- In order to find R_A and R_B, resolve horizontally and vertically in the view along the line of intersection :

$$R_A \sin(\beta - \tfrac{1}{2}\xi) = R_B \sin(\beta - \tfrac{1}{2}\xi)$$
$$R_A \cos(\beta - \tfrac{1}{2}\xi) + R_B \cos(\beta + \tfrac{1}{2}\xi) = W \cos \psi_i$$

- Solving for R_A and R_B and adding :

$$R_A + R_B = \frac{W.\mathrm{Cos}\,\psi_i.\mathrm{Sin}\beta}{\mathrm{Sin}\frac{1}{2}\xi}$$



- Hence :

$$F.S. = \frac{\sin\beta}{\sin\frac{1}{2}\xi}.\frac{\tan\phi}{\tan\psi_i}$$

Wedge Analysis including Cohesion, Friction and Water Pressure

The numbering used throughout this book is as follows:

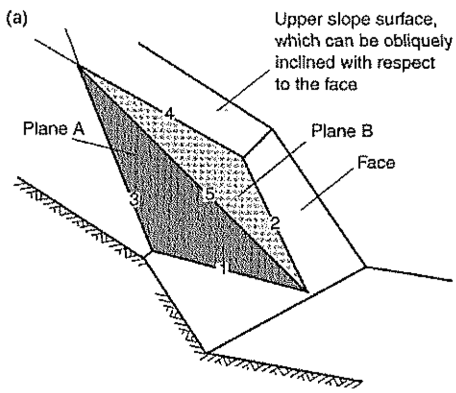
1 – Intersection of plane A with the slope face

2 – Intersection of plane B with the slope face

3 – Intersection of plane A with upper slope surface

4 – Intersection of plane B with upper slope surface

5 – Intersection of plane A and B

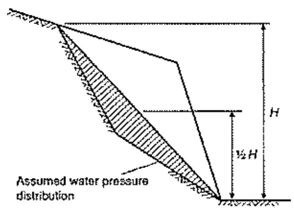


Wedge Analysis including Cohesion, Friction and Water Pressure

The factor of safety

$$F.S. = \frac{3}{\gamma_r H}(c_A X + c_B Y) + \left(A - \frac{\gamma_w}{2\gamma} X\right)\tan\phi_A + \left(B - \frac{\gamma_w}{2\gamma} Y\right)\tan\phi_B$$

where
- c_A and c_B = cohesive strengths of planes A and B
- ϕ_A and ϕ_B = angles of friction on planes A and B
- γ_r = unit weight of the rock
- γ_w = unit weight of water
- H = total height of the wedge
- X, Y, A and B = dimensionless factors which depend upon the geometry of the wedge.



Wedge Analysis including Cohesion, Friction and Water Pressure

The values of parameters X, Y, A and B :

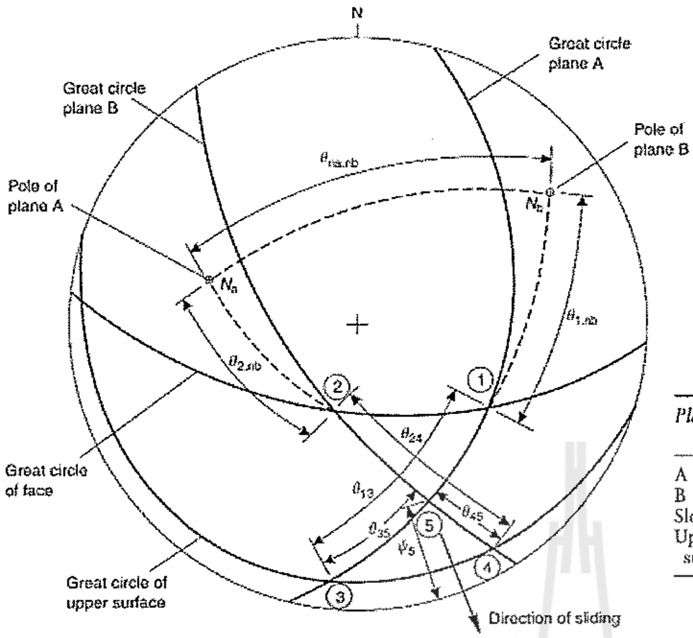
$$X = \frac{\sin\theta_{24}}{\sin\theta_{45}\cos\theta_{2.na}}$$

$$Y = \frac{\sin\theta_{13}}{\sin\theta_{35}\cos\theta_{1.na}}$$

$$A = \frac{\cos\psi_a - \cos\psi_b.\cos\theta_{na.nb}}{\sin\psi_5 \sin^2\theta_{2na.nb}}$$

$$B = \frac{\cos\psi_b - \cos\psi_a.\cos\theta_{na.nb}}{\sin\psi_5 \sin^2\theta_{2na.nb}}$$

Stereoplot of data



Plane	*Dip*	*Dip direction*	*Properties*
A	45	105	$\phi_A = 20°$, $c_A = 24$ kPa
B	70	235	$\phi_B = 30°$, $c_B = 48$ kPa
Slope face	65	185	
Upper surface	12	195	

Wedge stability calculation sheet

Input data	*Function value*	*Calculated values*
$\psi_a = 45°$ $\psi_b = 70°$ $\psi_5 = 31.2°$ $\psi_{na.nb} = 101°$	$\cos\psi_a = 0.707$ $\cos\psi_b = 0.342$ $\sin\psi_5 = 0.518$ $\cos\psi_{na.nb} = -0.191$ $\sin\psi_{na.nb} = 0.982$	$A = \dfrac{\cos\psi_a - \cos\psi_b\cos\theta_{na.nb}}{\sin\psi_5\sin^2\theta_{na.nb}} = \dfrac{0.707 + 0.342 \times 0.191}{0.518 \times 0.964} = 1.548$ $B = \dfrac{\cos\psi_b - \cos\psi_a\cos\theta_{na.nb}}{\sin\psi_5\sin^2\theta_{na.nb}} = \dfrac{0.342 + 0.707 \times 0.191}{0.518 \times 0.964} = 0.956$
$\theta_{24} = 65°$ $\theta_{45} = 25°$ $\theta_{2.na} = 50°$	$\sin\theta_{24} = 0.906$ $\sin\theta_{45} = 0.423$ $\cos\theta_{2.na} = 0.643$	$X = \dfrac{\sin\theta_{24}}{\sin\theta_{45}\cos\theta_{2.na}} = \dfrac{0.906}{0.423 \times 0.643} = 3.336$
$\theta_{13} = 62°$ $\theta_{35} = 31°$ $\theta_{1.nb} = 60°$	$\sin\theta_{13} = 0.883$ $\sin\theta_{35} = 0.515$ $\cos\theta_{1.nb} = 0.500$	$Y = \dfrac{\sin\theta_{13}}{\sin\theta_{35}\cos\theta_{1.nb}} = \dfrac{0.883}{0.515 \times 0.5} = 3.429$
$\phi_A = 30°$ $\phi_B = 20°$ $\gamma_r = 25\ \text{kN/m}^3$ $\gamma_w = 9.81\ \text{kN/m}^3$ $c_A = 24$ kPa $c_B = 48$ kPa $H = 40$ m	$\tan\phi_A = 0.577$ $\tan\phi_B = 0.364$ $\gamma_w/2\gamma_r = 0.196$ $3c_A/\gamma H = 0.072$ $3c_B/\gamma H = 0.144$	$FS = \dfrac{3}{\gamma_r H}(c_A X + c_B Y) + \left(A - \dfrac{\gamma_w}{2\gamma_r}X\right)\tan\phi_A + \left(B - \dfrac{\gamma_w}{2\gamma_r}Y\right)\tan\phi_B$ $FS = 0.241 + 0.494 + 0.893 - 0.376 + 0.348 - 0.244 = 1.36$

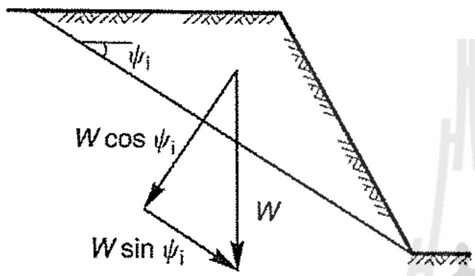
Analysis of Wedge Failure

- In other words:

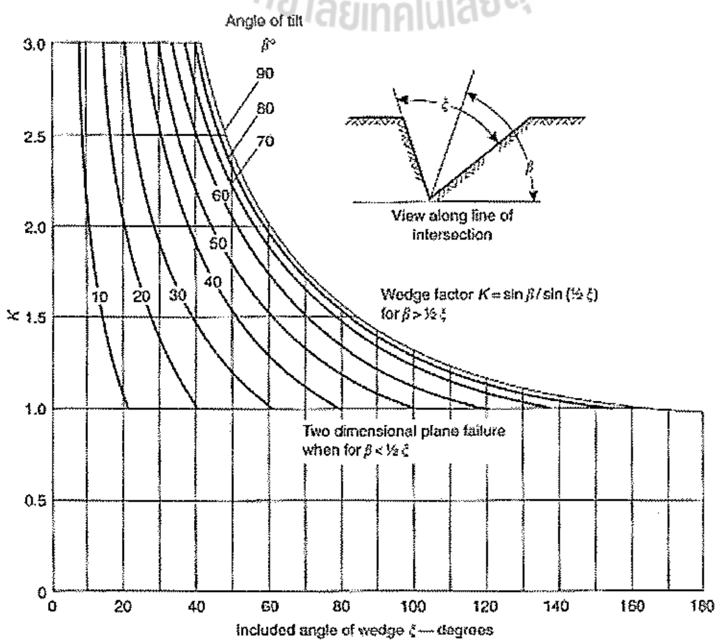
$$F.S._w = K\ F.S._p$$

Where $F.S._w$ = factor of safety of a wedge supported by friction only.

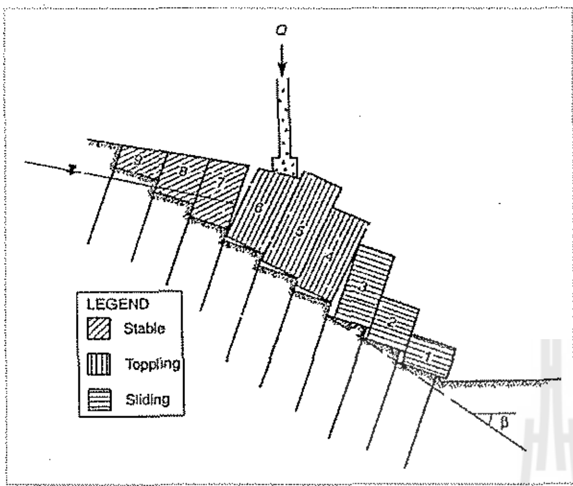
$F.S._p$ = factor of safety of a plane failure in which the slope face is inclined at ψ_{fi} and the failure plane is inclined at ψ_i.



Wedge Factor, K



Stability of Toppling Blocks



$$N_n = W_n \cos\psi_b - (P_{n-1} - P_n)\tan\phi_s - \tfrac{1}{2}(y_w + z_w)\gamma_w \Delta x + Q\sin(\psi_Q - \psi_b) \quad (6.15)$$

$$S_n = W_n \sin\psi_b - (P_{n-1} - P_n) + \tfrac{1}{2}(y_w^2 - z_w^2)\gamma_w + Q\cos(\psi_Q - \psi_b) \quad (6.16)$$

Stability of Toppling Blocks

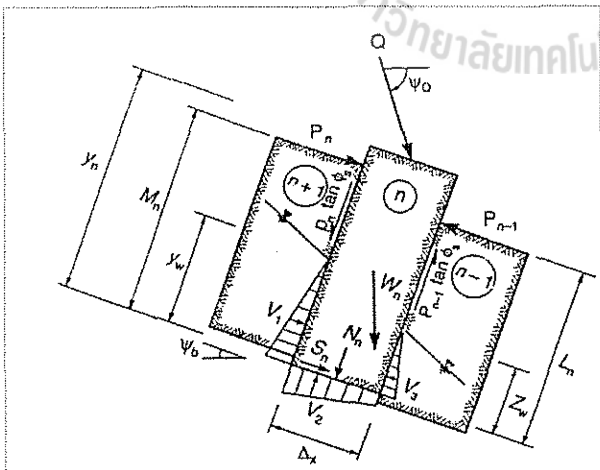


Figure 6.10 Forces acting on a toppling block.

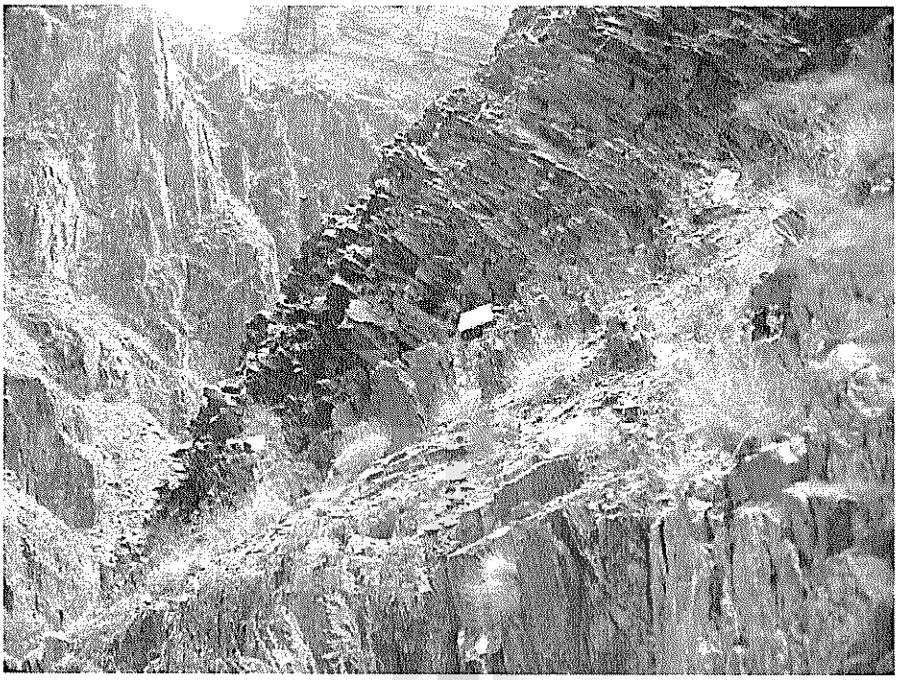
$$P_{n-1,t} = \frac{1}{L_n}\left\{P_n(M_n - \Delta x \tan\phi_s) + \frac{W_n}{2}(y_n \sin\psi_b - \Delta x\cos\psi_b) + V_1\frac{y_w}{3} + \gamma_w\frac{(\Delta x)^2}{2}\left(\frac{y_w}{2} + \frac{z_w}{3}\right) - V_3\frac{z_w}{3} + Q\left[\sin(\psi_Q - \psi_b)\frac{\Delta x}{2} - \cos(\psi_Q - \psi_b)y_n\right]\right\} \quad (6.17)$$

$$V_1 = \tfrac{1}{2}\gamma_w y_w^2 \qquad V_3 = \tfrac{1}{2}\gamma_w z_w^2$$

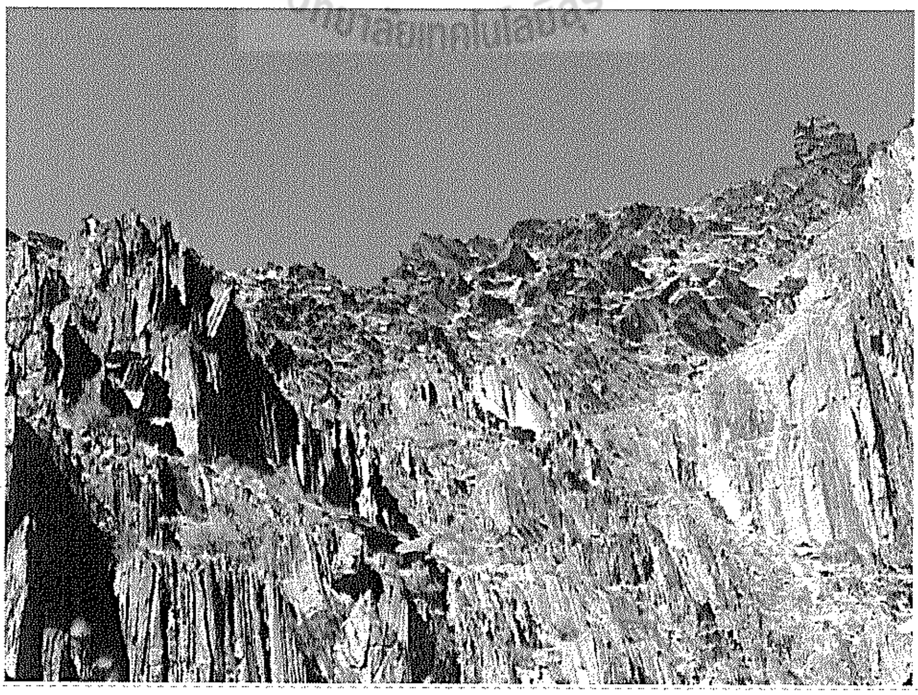
$$P_{n-1,s} = P_n + \{-W(\cos\psi_p \tan\phi_b - \sin\psi_b) + V_1 - V_2\tan\phi_b - V_3 + Q[-\sin(\psi_Q - \psi_b)\tan\phi_b + \cos(\psi_Q - \psi_b)]\} \times (1 - \tan\phi_s \tan\phi_b)^{-1} \quad (6.20)$$

$$V_2 = \tfrac{1}{2}\gamma_w(y_w + z_w)\Delta x$$

Toppling Failure



Toppling Failure

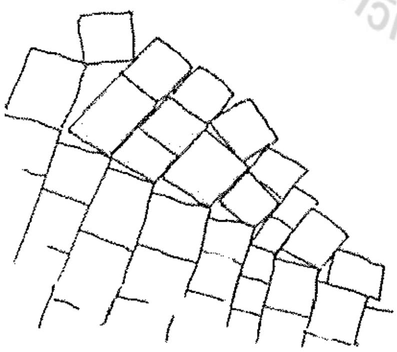


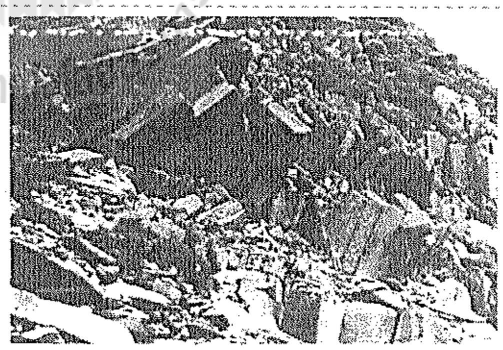
Type of Toppling Failure

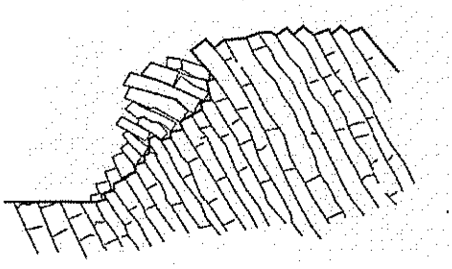
Goodman and Bray (1976)

- Block Toppling
- Flexural Toppling
- Block-Flexural Toppling
- Secondary Toppling Modes

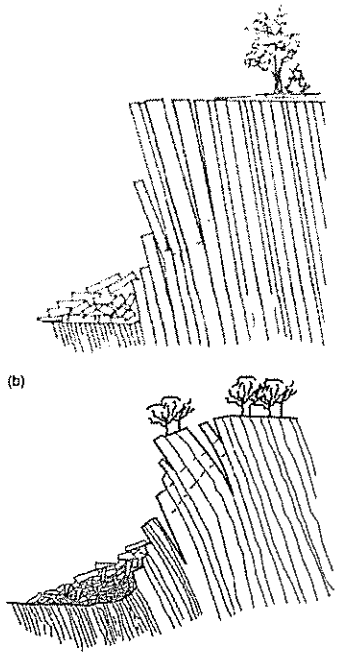
1. Block Toppling



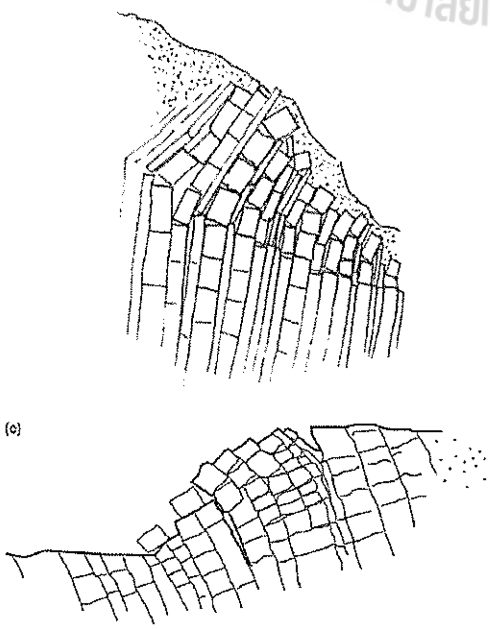


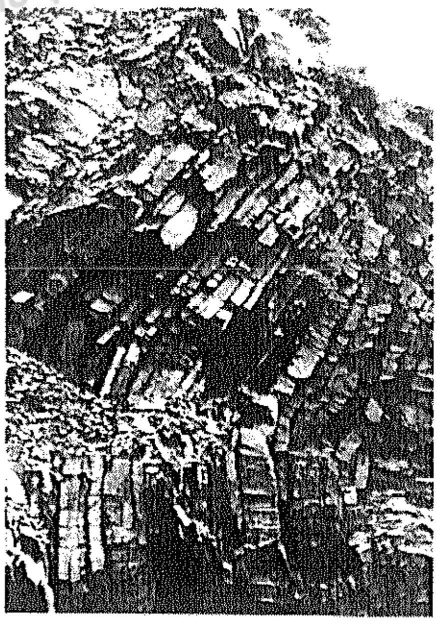


2. Flexural Toppling

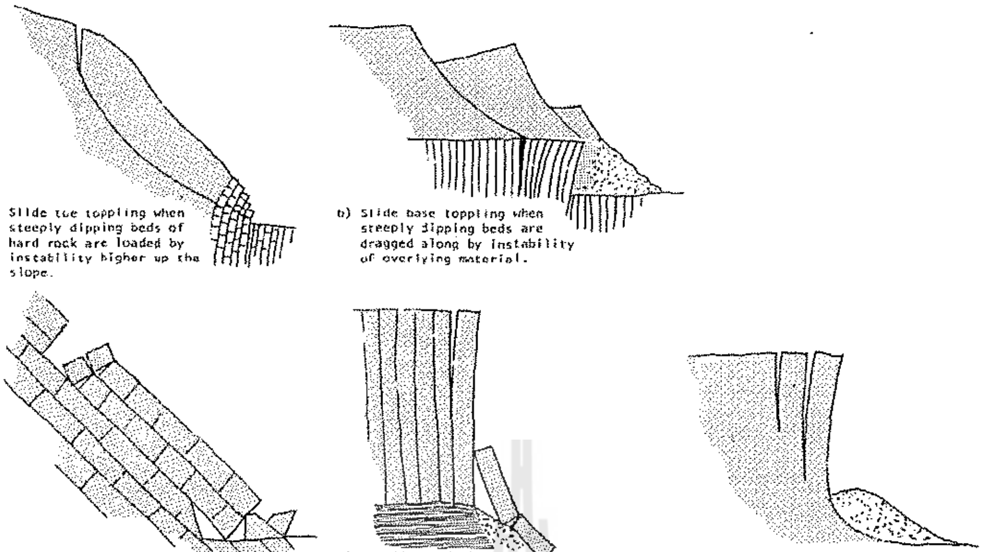


3. Block-Flexural Toppling





4. Secondary Toppling Modes



a) Slide toe toppling when steeply dipping beds of hard rock are loaded by instability higher up the slope.

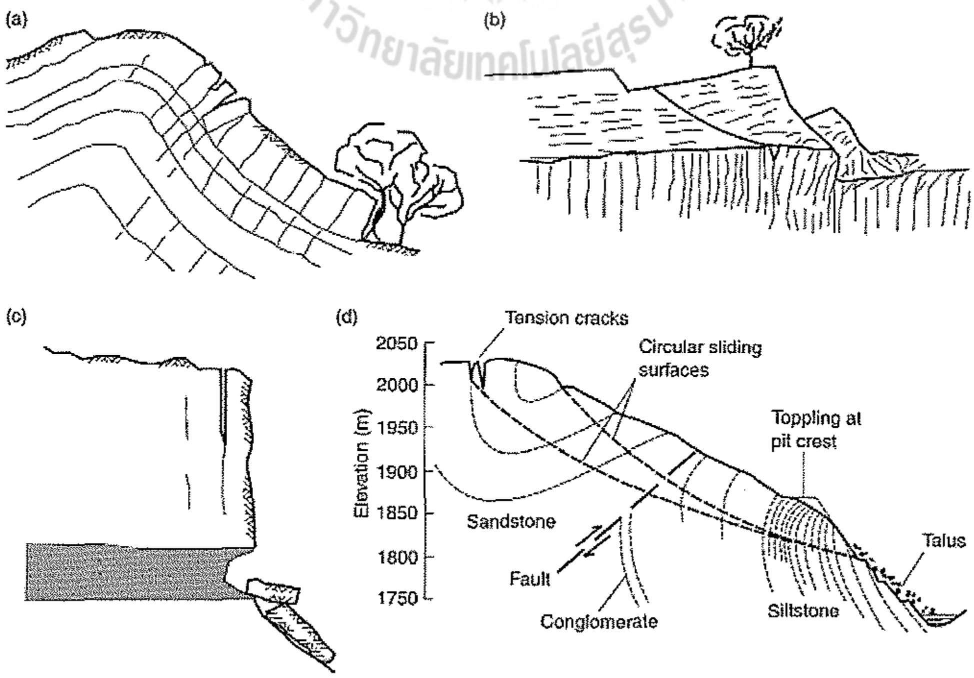
b) Slide base toppling when steeply dipping beds are dragged along by instability of overlying material.

c) Slide head toppling when movement lower in the slope frees block to topple.

d) Toppling and slumping of columnar rock by weathering of underlying material.

e) Tension crack toppling in cohesive materials.

4. Secondary Toppling Modes (cont.)

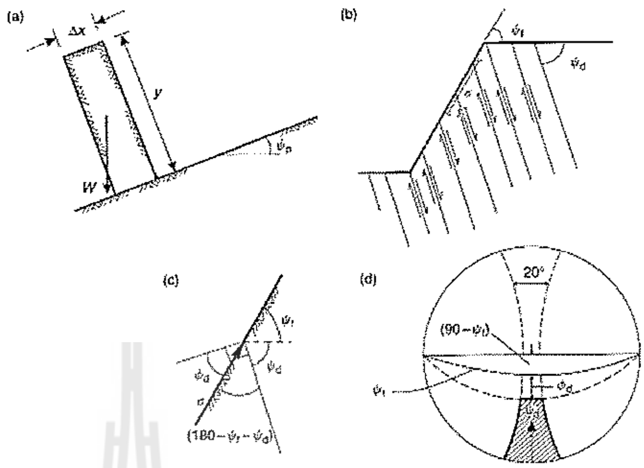


Kinematics of Block Toppling Failure

1. Block Shape Test
   $\psi_p < \phi_p$ (Stable)
   $\Delta x/y < \tan \psi_p$ (Topple)

2. Inter-Layer Slip Test
   $(180 - \psi_f - \psi_d \geq (90 - \phi_d)$
   or $\psi_d \geq (90 - \psi_f) + \phi_d$

3. Block Alignment Test
   $|(\alpha_f - \alpha_d)| < 10°$



Limit Equilibrium Analysis of Toppling on a Stepped Base

1. Block Geometry
2. Block Stability
3. Calculation Procedure for Toppling Stability of s System of Blocks
4. Cable Force Required to Stability a Slope
5. Factor of Safety for Limiting Equilibrium Analysis
6. Application of External Force to Toppling Slopes

Limit Equilibrium Analysis of Toppling on a Stepped Base

1. Block Geometry

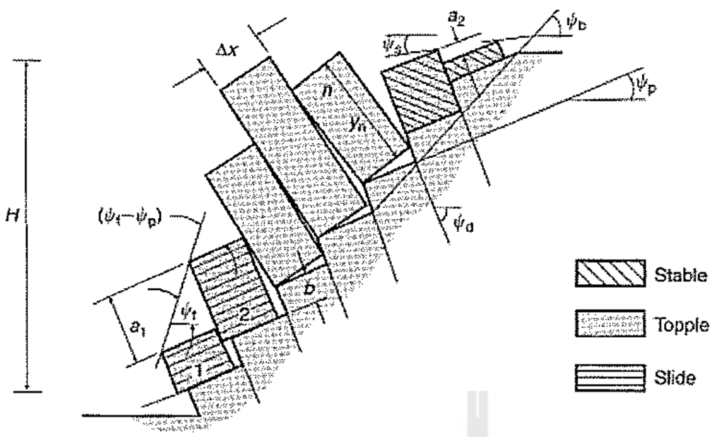
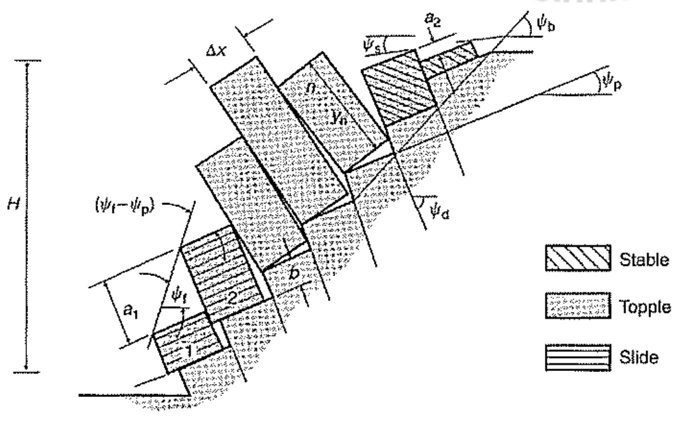


Figure 9.7 Model for limiting equilibrium analysis of toppling on a stepped base (Goodman and Bray, 1976).

$$n = \frac{H}{\Delta x}\left[\operatorname{cosec}(\psi_b) + \left(\frac{\cot(\psi_b) - \cot(\psi_f)}{\sin(\psi_b - \psi_f)}\right)\sin(\psi_s)\right]$$

Limit Equilibrium Analysis of Toppling on a Stepped Base

1. Block Geometry



in position below crest of slope

$y_n = n(a_1 - b)$

above the crest

$y_n = y_{n-1} - a_2 - b$

$a_1 = \Delta x \tan(\psi_f - \psi_p)$

$a_2 = \Delta x \tan(\psi_p - \psi_s)$

$b = \Delta x \tan(\psi_b - \psi_p)$

ψ_p = dip of the base of the block

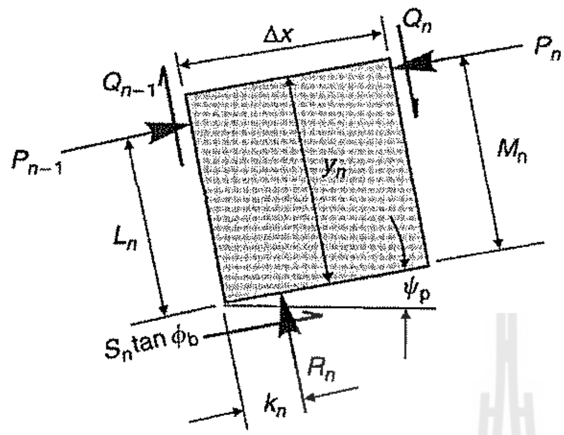
ψ_d = dip of the orthogonal planes forming the faces of the block = $(90 - \psi_p)$

ψ_b = dip of the base plane (a stepped surface with an overall dip)

Limit Equilibrium Analysis of Toppling on a Stepped Base

1. Block Geometry

(a)



in position below crest of slope

$M_n = y_n$

$L_n = y_n - a_1$

is the slope crest

$M_n = y_n - a_2$

$L_n = y_n - a_1$

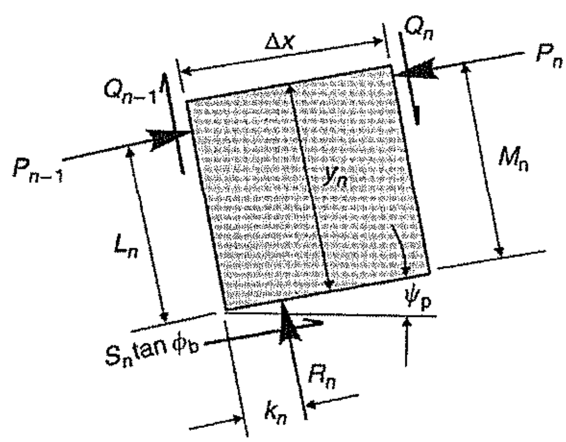
above the slope crest

$M_n = y_n - a_2$

$L_n = y_n$

Limit Equilibrium Analysis of Toppling on a Stepped Base

(a)



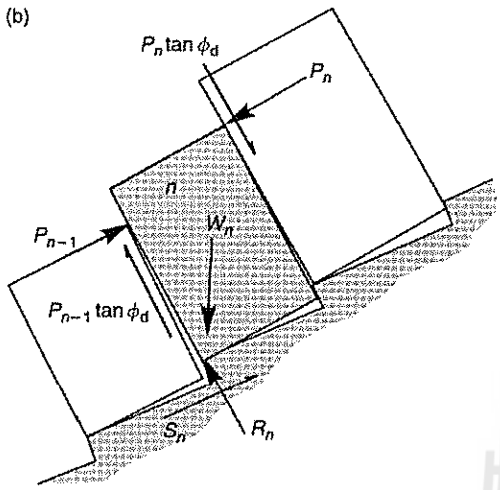
For limit friction on the side of block

$Q_n = P_n \tan \phi_d$

$Q_{n-1} = P_{n-1} \tan \phi_d$

ϕ_d = friction angle of the side of block

Limit Equilibrium Analysis of Toppling on a Stepped Base



normal and shear force acting on the base of block

$R_n = W_n \cos \psi_p + (P_n - P_{n-1}) \tan \phi_d$

$S_n = W_n \sin \psi_p + (P_n - P_{n-1})$

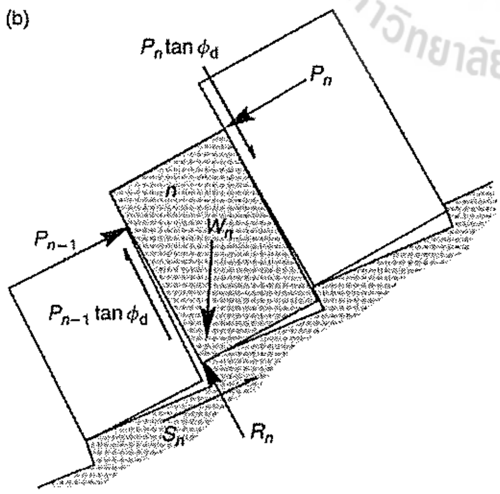
ϕ_d = friction angle of the side of block

check for sliding does not occur on the base

$R_n > 0$

$|S_n| > R_n \tan \phi_p$

Limit Equilibrium Analysis of Toppling on a Stepped Base



to prevent toppling

rotational equilibrium

$$P_{n-1,t} = [P_n(M_n - \Delta x \tan \phi_d) + (W_n/2)(y_n \sin \psi_p - \Delta x \cos \psi_p)] / L_n$$

to prevent sliding

$$P_{n-1,s} = P_n - [\{W_n(\cos \psi_p \tan \phi_p - \sin \psi_p)\}/\{1 - \tan \phi_p \tan \phi_d\}]$$

If $P_{n-1,t} > P_{n-1,s}$,block is on point of toppling

If $P_{n-1,t} < P_{n-1,s}$,block is on point of sliding

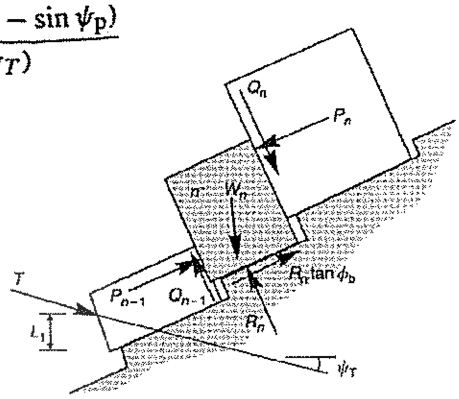
Cable Force Required to Stability a Slope

the anchor tension required to prevent toppling of block 1

$$T_t = \frac{W_1/2(y_1 \sin\psi_p - \Delta x \cos\psi_p) + P_1(y_1 - \Delta_x \tan\phi_d)}{L_1 \cos(\psi_p + \psi_T)}$$

the anchor tension required to prevent sliding of block 1

$$T_s = \frac{P_1(1 - \tan\phi_p \tan\phi_d) - W_1(\tan\phi_p \cos\psi_p - \sin\psi_p)}{\tan\phi_p \sin(\psi_p + \psi_T) + \cos(\psi_p + \psi_T)}$$



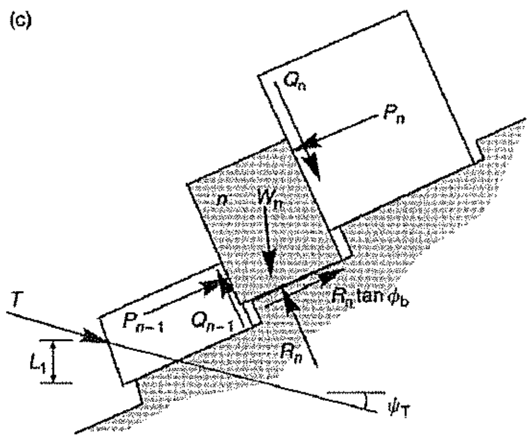
Cable Force Required to Stability a Slope

when the force T is applied to block 1,
the normal and shear force on the base are,

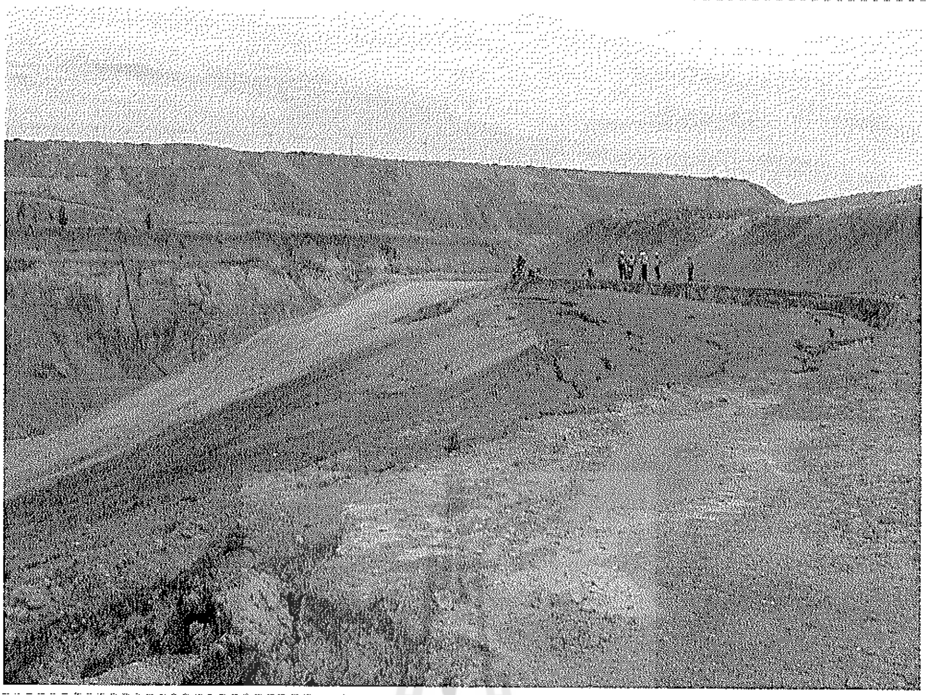
$R_1 = P_1 \tan\phi_d + T \sin(\psi_p + \psi_T) + W_1 \cos\psi_p$

$S_1 = P_1 - T \cos(\psi_p + \psi_T) + W_1 \sin\psi_p$

(c)



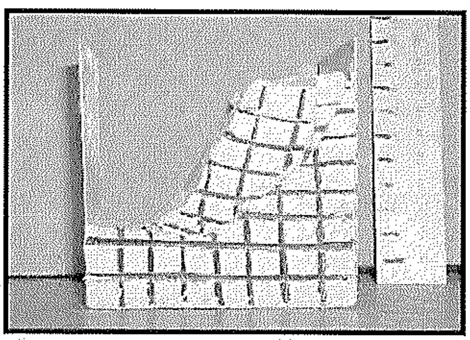
Circular Failure



Conditions for Circular Failure and Methods of Analysis

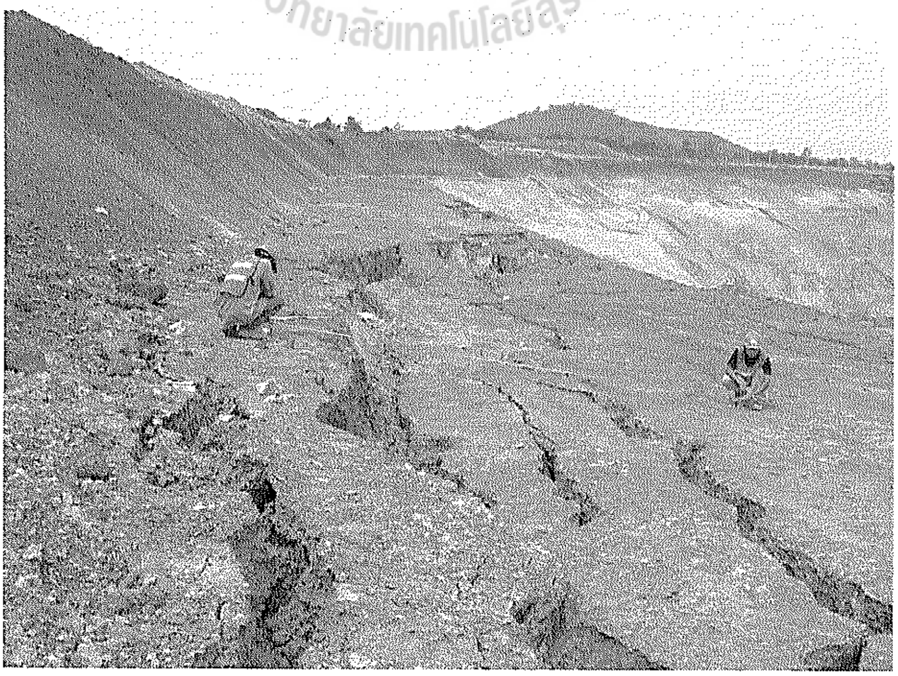
- The individual particles in a soil or rock mass are very small when compare with slope height
- The particles are not interblock

For examples:
- Soil slope
- Rock filled / waste rock slope
- Heavily-fractured rock
- Highly altered and weathered rocks

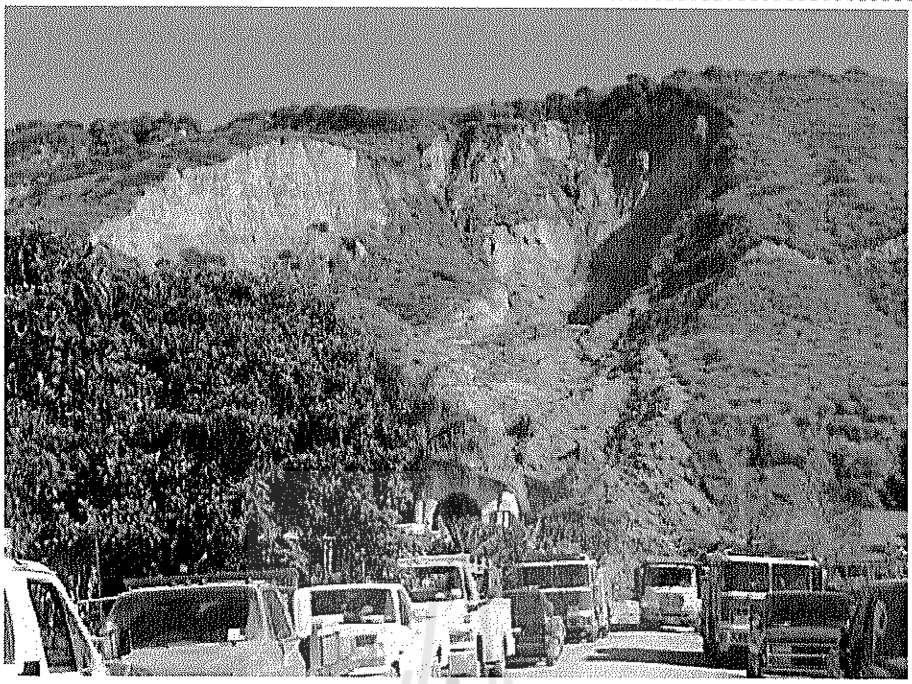


Circular Failure

Circular Failure



Circular Failure



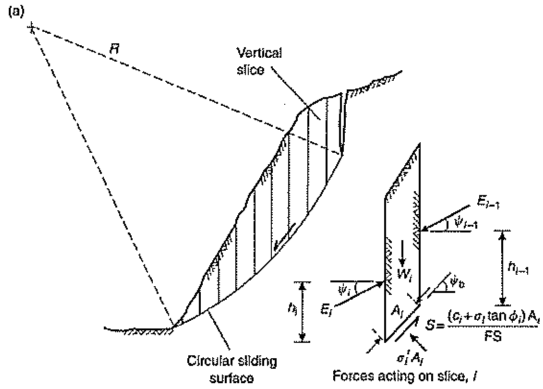
Stability Analysis Procedure

Defining the factor of safety of the slope as

$$F.S. = \frac{\text{Shear strength available to resist sliding } (c + \sigma\tan\phi)}{\text{Shear stress required for equilibrium on slipe surface } (\tau_e)}$$

and rearranging this equation, we get

$$\tau_e = \frac{c + \sigma\tan\phi}{F.S}$$



Derivation of Circular Failure Charts

Assumptions

- Homogeneous material
- Coulomb criterion shear strength ($\tau = c + \sigma \, . \, \mathrm{Tan}\, \phi$)
- Circular failure surface passes slope toe
- Vertical tension crack exist
- Locations of tension crack and of failure surface are critical (minimum F.S.)
- Groundwater conditions, varying from a dry slope to a fully saturated slope

Defining the factor of safety of the slope as

$$\mathrm{F.S.} = \frac{\text{Shear strength available to resist sliding } (c + \sigma \tan\phi)}{\text{Shear stress required for equilibrium on slipe surface } (\tau_e)}$$

and rearranging this equation, we get

$$\tau_e = \frac{c + \sigma \, . \, \mathrm{Tan}\,\phi}{\mathrm{F.S}}$$

Groundwater Flow Assumptions

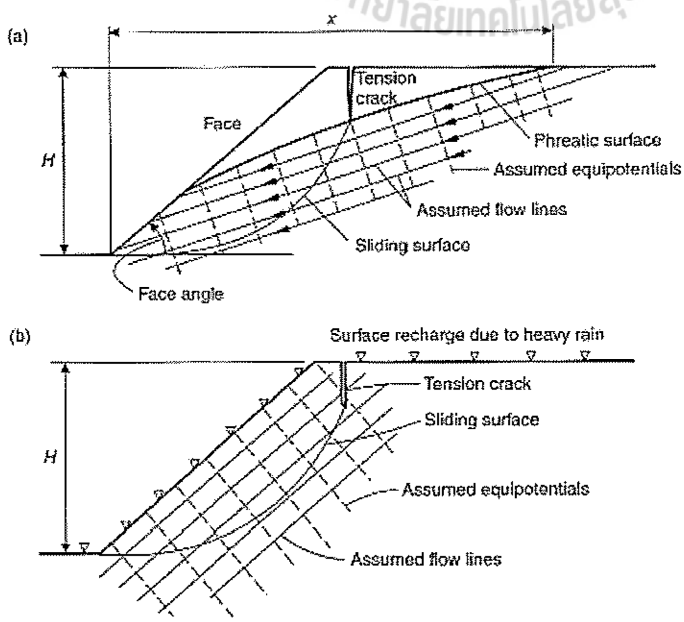
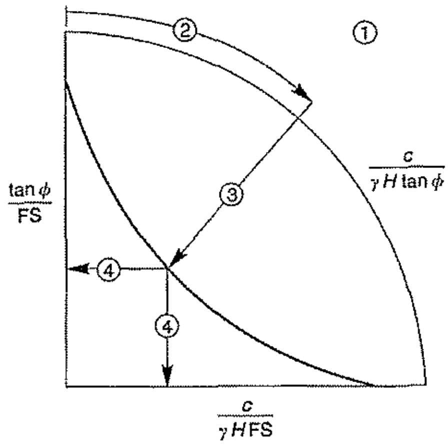


Figure 8.3 Definition of ground water flow patterns used in circular failure analysis of slopes in weak and closely fractured rock: (a) ground water flow pattern under steady state drawdown conditions where the phreatic surface coincides with the ground surface at a distance *x* behind the toe of the slope. The distance *x* is measured in multiples of the slope height *H*; (b) ground water flow pattern in a saturated slope subjected to surface recharge by heavy rain.

Use of the Circular Failure Charts



Step 1 : Decide upon the groundwater conditions (chart no. 1-5)

Step 2 : Calculate the value of the dimensionless ratio

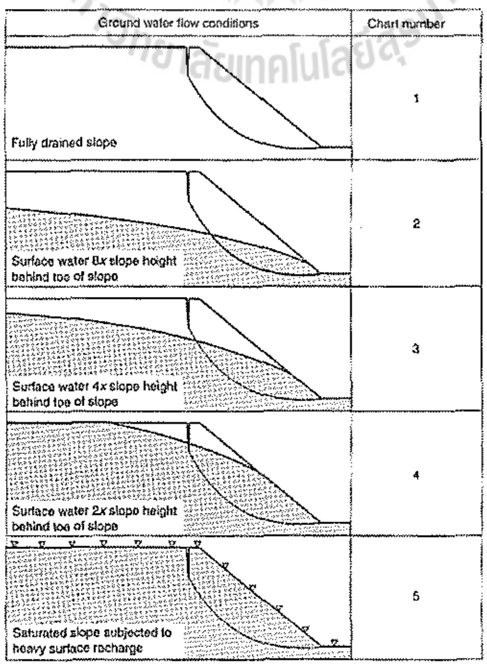
$$\frac{c}{\gamma H \tan\phi}$$

Find this value on the outer circular scale of the chart.

Step 3 : Follow the radial line from the value found in step 2 to its intersection with the curve which corresponds to the slope angle under consideration.

Step 4 : Find the corresponding value of **tanϕ/FS** or **c/γHFS**, depending upon which is more convenient, and calculate the F.S.

Groundwater Flow Conditions



Circular Failure Charts No.1

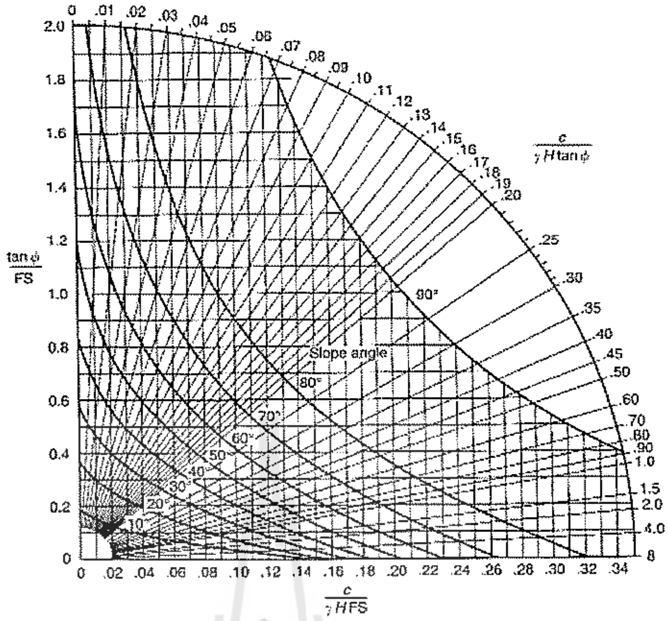


Figure 8.6 Circular failure chart number 1—fully drained slope.

Circular Failure Charts No.2

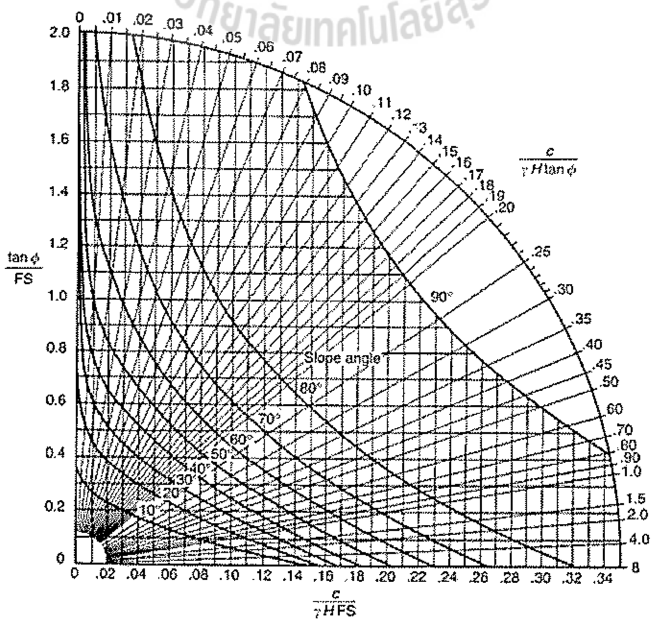


Figure 8.7 Circular failure chart number 2—ground water condition 2 (Figure 8.5).

Circular Failure Charts No.3

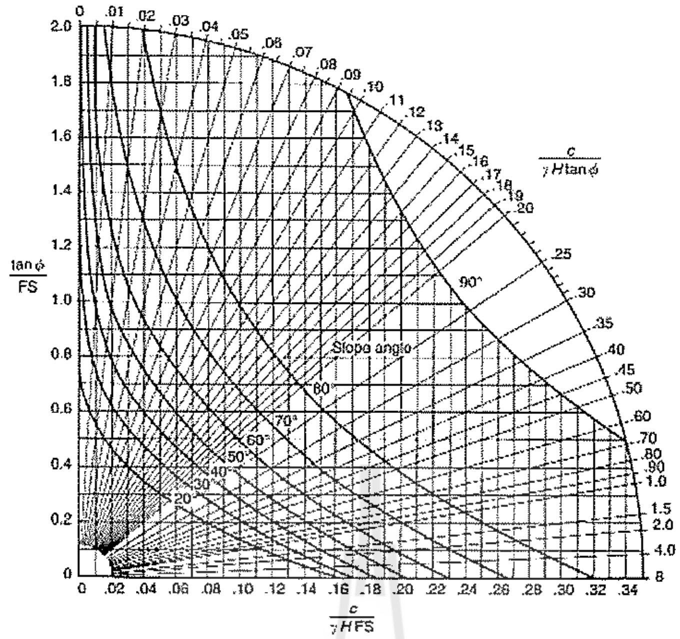


Figure 8.8 Circular failure chart number 3—ground water condition 3 (Figure 8.4).

Circular Failure Charts No.4

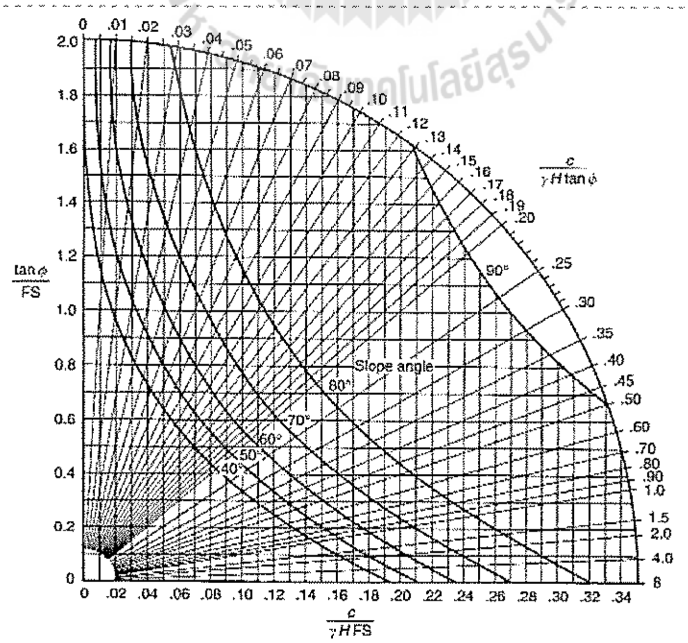


Figure 8.9 Circular failure chart number 4—ground water condition 4 (Figure 8.4).

Circular Failure Charts No.5

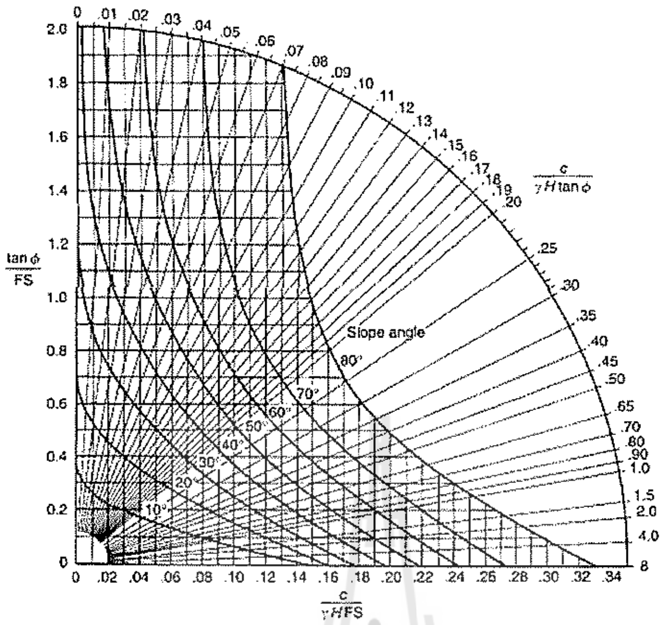


Figure 8.10 Circular failure chart number 5—fully saturated slope.

Example of Circular Failure Analysis using Chart

Given:

Slope height, H = 15.2 m.

Slope angle, ψ_f = 40 degrees

Soil density, γ_r = 15.7 kN/m^3

Cohesion, c = 38 kPa

Friction angle, ϕ = 30 degrees

Surface water source 61 m behind toe

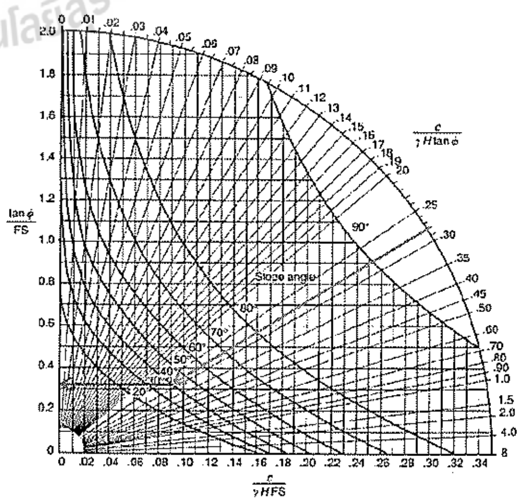
Step 1 : Decide upon the groundwater conditions
(61/15.5) ~ 4 → Chart no. 3

Step 2 : Calculate the value of the ratio

$$\frac{c}{\gamma H \tan\phi} = 0.28$$

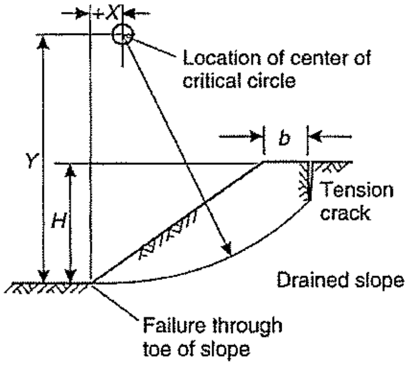
Step 3 : Corresponding value of
tan ϕ / F.S. = 0.32 (for ψ_f = 40 degrees)

Step 4 : Calculate the F.S.
F.S. = (0.32/tan 30) = 1.80

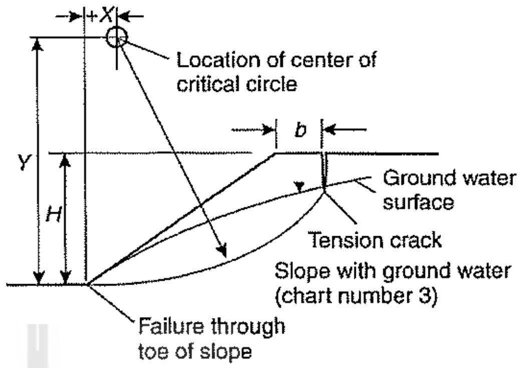


Location of Critical Slide Surface and Tension Crack

- Locations of both the critical failure circle and the critical tension crack for limiting equilibrium (F.S. = 1).

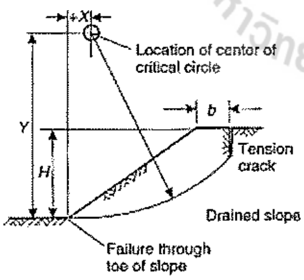


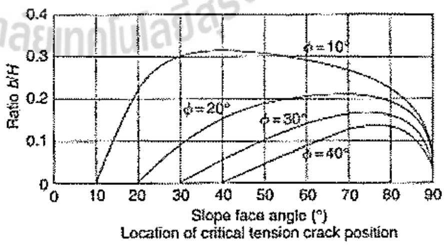
Drained Slope

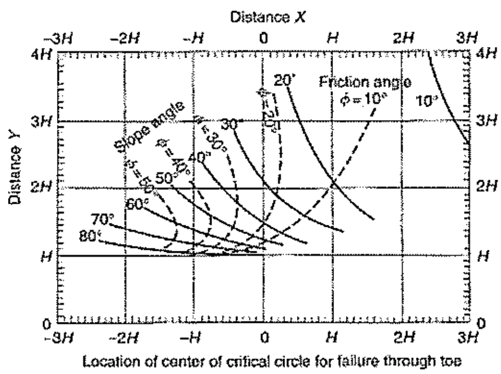


Slope with Groundwater

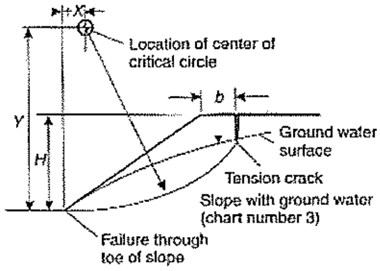
Drained Slope

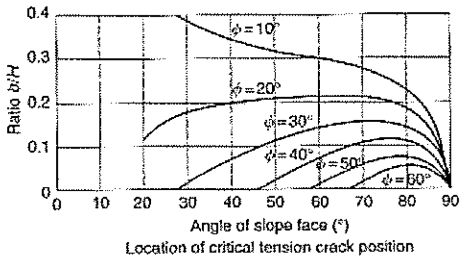


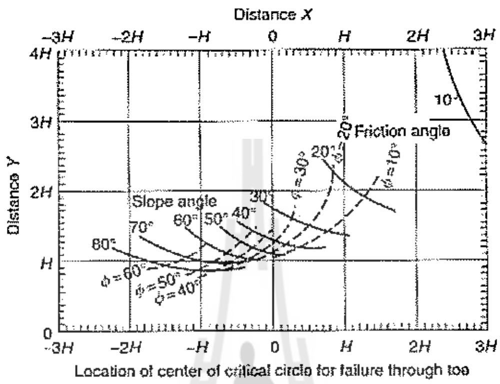




Slope with Groundwater (chart no.3)







Example of Find the

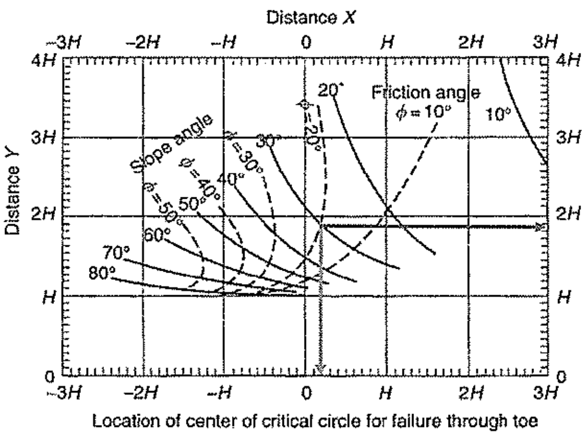
Given:

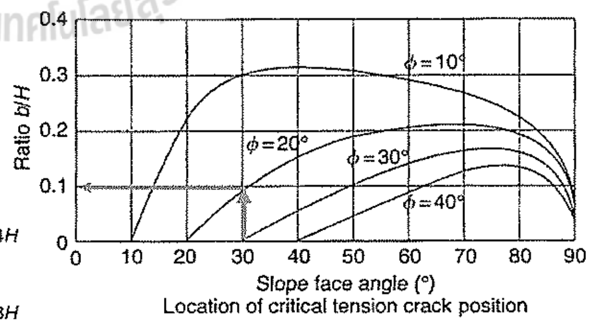
Drained Slope

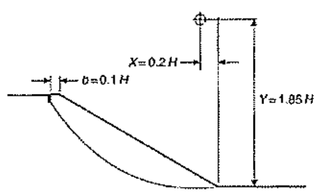
Slope height, H = 15.2 m.

Slope angle, ψ_f = 30 degrees

Friction angle, ϕ = 20 degrees

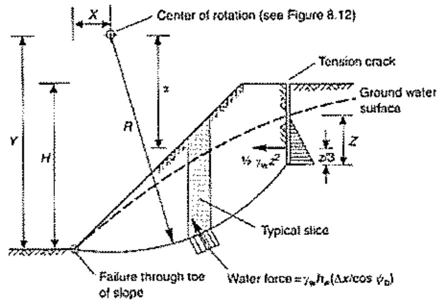


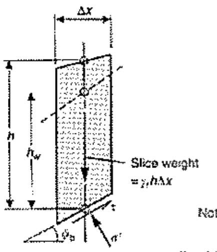




X = 0.2 H
Y = 1.85 H
b = 0.1 H

Bishop's Simplified Method of Slices (Mohr-Coulomb)





Factor of safety:

$$FS = \frac{\Sigma X/(1+Y/FS)}{\Sigma Z + Q} \qquad (8.3)$$

where

$$X = [c + (\gamma_r h - \gamma_w h_w)\tan\phi](\Delta x/\cos\psi_b) \qquad (8.4)$$

$$Y = \tan\psi_b \tan\phi \qquad (8.5)$$

$$Z = \gamma_r h \Delta x \sin\psi_b \qquad (8.6)$$

$$Q = \tfrac{1}{2}\gamma_w z^2 (z/R) \qquad (8.7)$$

Note: angle ψ_b is negative when sliding uphill

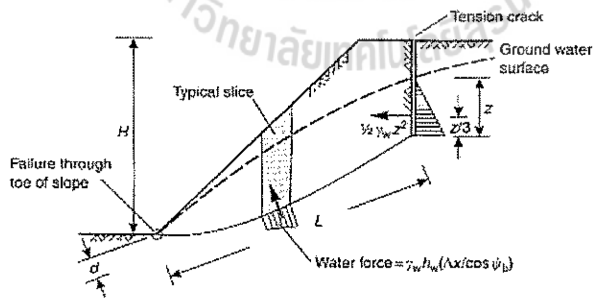
The following conditions must be satisfied for each slice:

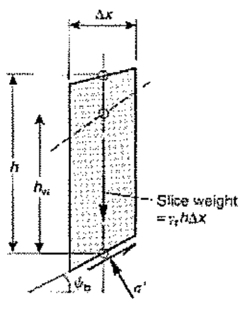
(1) $$\sigma' = \frac{\gamma_r h - \gamma_w h_w - c(\tan\psi_b/FS)}{1+Y/FS} \qquad (8.8)$$

(2) $$\cos\psi_b(1+Y/FS) > 0.2 \qquad (8.9)$$

Figure 8.16 Bishop's simplified method of slices for the analysis of non-circular failure in slopes cut into materials in which failure is defined by the Mohr–Coulomb failure criterion.

Janbu's Modified Method of Slices (Mohr-Coulomb)





Factor of safety:

$$FS = \frac{f_0 \Sigma X/(1+Y/FS)}{\Sigma Z + Q} \qquad (8.10)$$

where

$$X = [c + (\gamma_r h - \gamma_w h_w)\tan\phi](1+\tan^2\psi_b)\Delta x \qquad (8.11)$$

$$Y = \tan\psi_b \tan\phi \qquad (8.12)$$

$$Z = \gamma_r h \Delta x \tan\psi_b \qquad (8.13)$$

$$Q = \tfrac{1}{2}\gamma_w z^2 \qquad (8.14)$$

Note: angle ψ_b is negative when sliding uphill

Approximate correction factor f_0

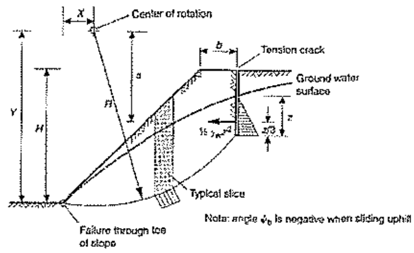
$$f_0 = 1 + K(d/L - 1.4(d/L)^2) \qquad (8.15)$$

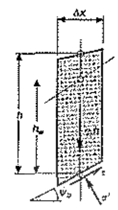
for $c' = 0$; $K = 0.31$

$c' > 0$, $\phi' > 0$; $K = 0.50$

Figure 8.17 Janbu's modified method of slices for the analysis of non-circular failure in slopes cut into materials in which failure is defined by the Mohr–Coulomb failure criterion.

Janbu's Modified Method of Slices (non-liner shear strength)





Factor of safety:

$$FS = \frac{\sum (c_i' + \sigma' \tan\phi_i')(\Delta x / \cos\psi_b)}{\sum \gamma_r h \Delta x \sin\psi_b + \frac{1}{2}\gamma_w z^2 a / R} \qquad (8.16)$$

where

$$\sigma' = \gamma_r h \cos^2\psi_b - \gamma_w h_w \qquad \text{(Fellenius solution)} \qquad (8.17)$$

and

$$\sigma' = \frac{\gamma_r h - \gamma_w h_w - (c_i' \tan\psi_b / FS)}{1 + (\tan\phi_i' \tan\psi_b / FS)} \qquad \text{(Bishop solution)} \qquad (8.18)$$

$$\phi' = \sin^{-1}\left[\frac{6 a m_b (s + m_b \sigma_{3n}')^{a-1}}{2(1+a)(2+a) + 6 a m_b (s + m_b \sigma_{3n}')^{a-1}}\right] \qquad (4.24)$$

$$c' = \frac{\sigma_{ci}[(1+2a)s + (1-a) m_b \sigma_{3n}'](s + m_b \sigma_{3n}')^{a-1}}{(1+a)(2+a)\sqrt{1 + [6 a m_b (s + m_b \sigma_{3n}')^{a-1}]/[(1+a)(2+a)]}} \qquad (4.25)$$

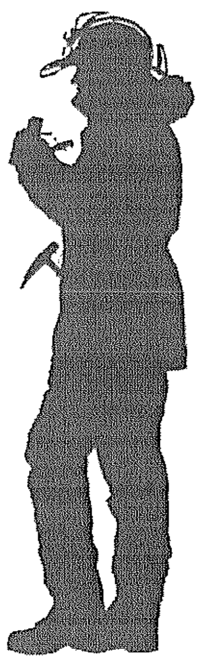
where $\sigma_{3n} = \sigma_{3max}'/\sigma_{ci}$

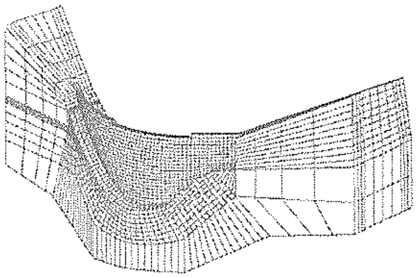
The conditions which must be satisfied for each slice are:

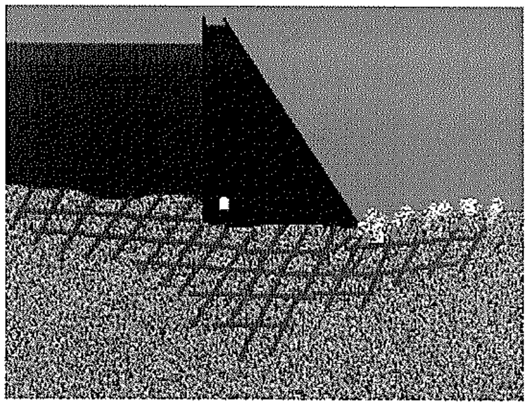
(1) $\sigma' > 0$, where σ' is calculated by Bishop's method

(2) $\cos\psi_b [1 + (\tan\psi_b \tan\phi_i')/FS] > 0.2$

Figure 8.18 Bishop's simplified method of slices for the analysis of circular failure in slope in material in which strength is defined by non-linear criterion given in Section 4.5.







434636 Foundations on Rock

Topic 7 Foundations of Gravity & Embankment Dams

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

General Requirements

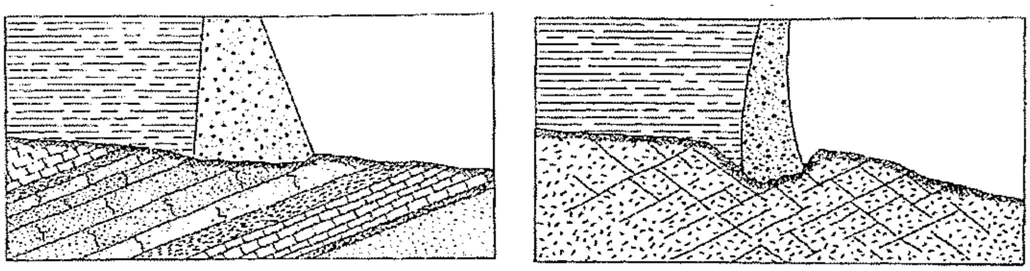
1. Stability against sliding
2. Stability against overturning
3. Stability under differential deformation
4. Control of seepage and erosion

Loads on Dams

1. Dead weight
   - Dam structure + intake + gates + bridge
   - Unit weight of concrete about 23 kN/m^3
2. External water forces (upstream)
   - Water + silt :horizontal → 13.5 kN/m^3
     :vertical → 19 kN/m^3
3. Internal water force
   - Uplift pressure in foundation and abument
4. Thermal expansion
   - Concrete Gravity Dams
5. Seismic force
   - Static acceleration

Sliding Stability

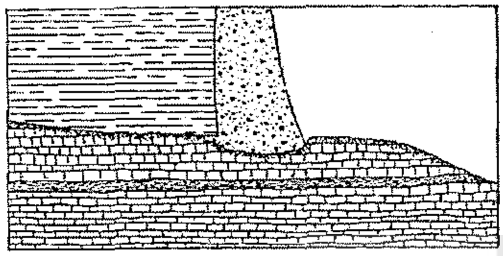
- Geological Conditions



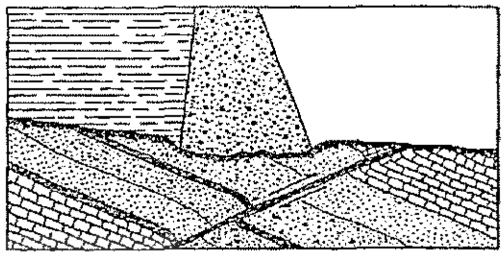
(a) (d)

Sliding Stability

- Geological Conditions



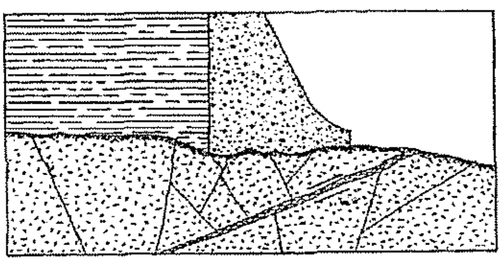
(b)



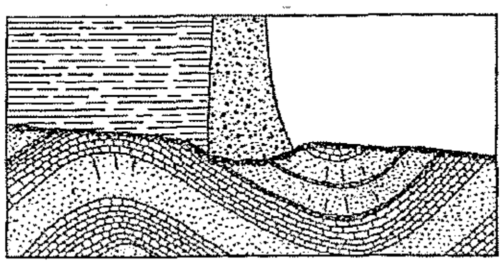
(e)

Sliding Stability

- Geological Conditions



(c)



(f)

Sliding Stability

- Shear strength
  - Rock shear strength

    ϕ, c (normally c = 0) → joint

    ϕ, c → infilling material
  - Rock-concrete shear strength

    in Earth dam

tension	0-450 kPa
cohesion	0-900 kPa
friction angle	32-54 degrees

Sliding Stability

- Water pressure distributions

  under dam foundation

  -uplift pressure

  -effect of drain

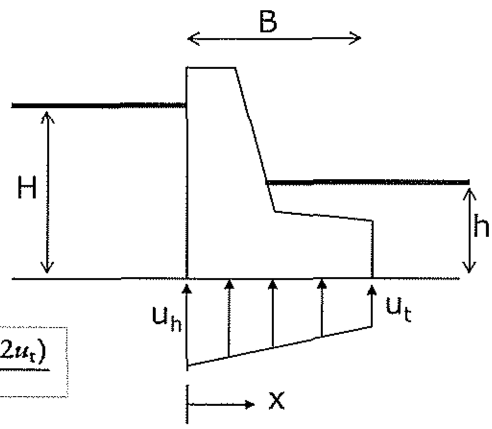
Uplift pressure

$$u_x = u_t + \frac{R(B-x)}{B}(u_h - u_t)$$

R = proportional reduction in head at drain

Uplift force

$$U = (u_t B) + \frac{(u_x - u_t)(B-x)}{2} + \frac{x(u_h + u_x - 2u_t)}{2}$$



Sliding Stability Analysis

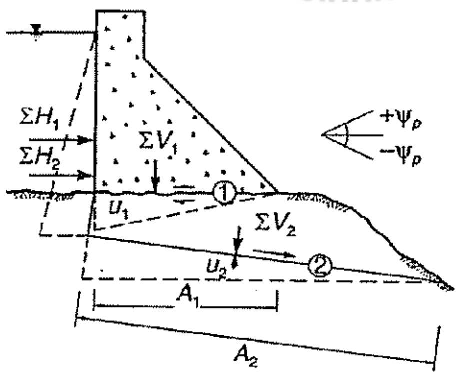
- For Horizontal Sliding

$$FS = \frac{cA_1 + (\Sigma V_1 - u_1)\tan\phi}{\Sigma H_1}$$

ΣH_1 = Sum of horizontal force
(reservoir + tailing water + ice + wind)
ΣV_1 = Sum of weight of dam structure

Sliding Stability Analysis

- For Non-horizontal Sliding



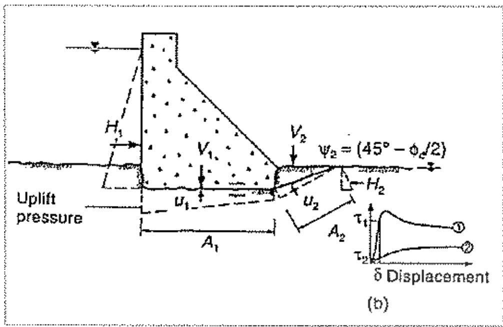
(a)

$$FS = \frac{cA_2 + [\Sigma V_2 \cos\psi_p - u_2 + \Sigma H_2 \sin\psi_p]\tan\phi}{\Sigma H_2 \cos\psi_p - \Sigma V_2 \sin\psi_p}$$

ψ_p = dip angle of sliding plane

Sliding Stability Analysis

- For Recessed Dam



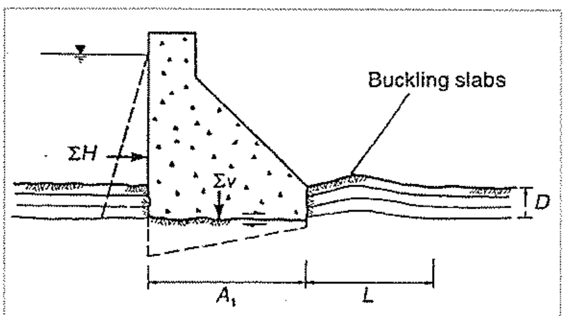
$$FS = \frac{\sum_{i=1}^{n} [c_i A_i \cos\psi_i + (V_i - u_i \cos\psi_i)\tan\phi_i]/\eta_{\psi i}}{\sum_{i=1}^{n} (H_i - V_i \tan\psi_i)}$$

i = Subscript related to n plane segment

$$\eta_{\psi i} = \frac{1 - \dfrac{\tan\phi_i \tan\psi_i}{FS}}{1 + \tan^2\psi_i}$$

Sliding Stability Analysis

- For sliding due to bucking



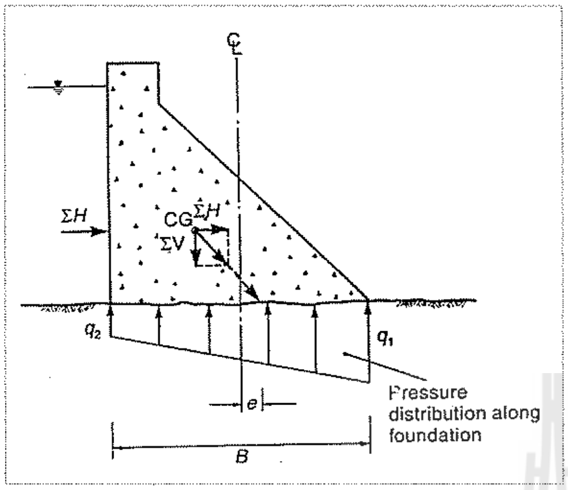
Bucking Resistance

$$f_r = \frac{\pi^2 EA}{\left(\dfrac{L}{D/2}\right)^2}$$

E = Deformation Modulus

$$FS = \frac{cA_1 + \Sigma V \tan\phi + f_r}{\Sigma H}$$

Overturning and stress distributions in foundation

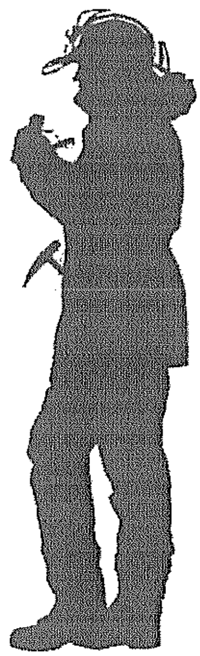


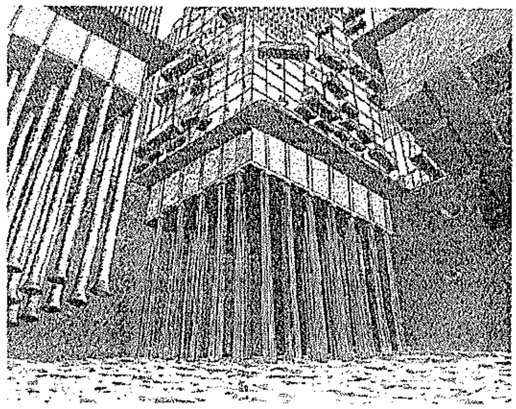
$$e = \frac{M}{\Sigma V}$$

$$q_1 = \Sigma\frac{V}{B}\left(1 + 6\frac{e}{B}\right)$$

and

$$q_2 = \Sigma\frac{V}{B}\left(1 - 6\frac{e}{B}\right)$$





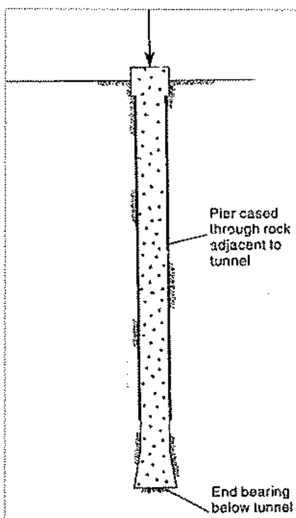
434636 Foundations on Rock

Topic 8 Rock Socket Piers

Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Load Capacity of Socket Piers

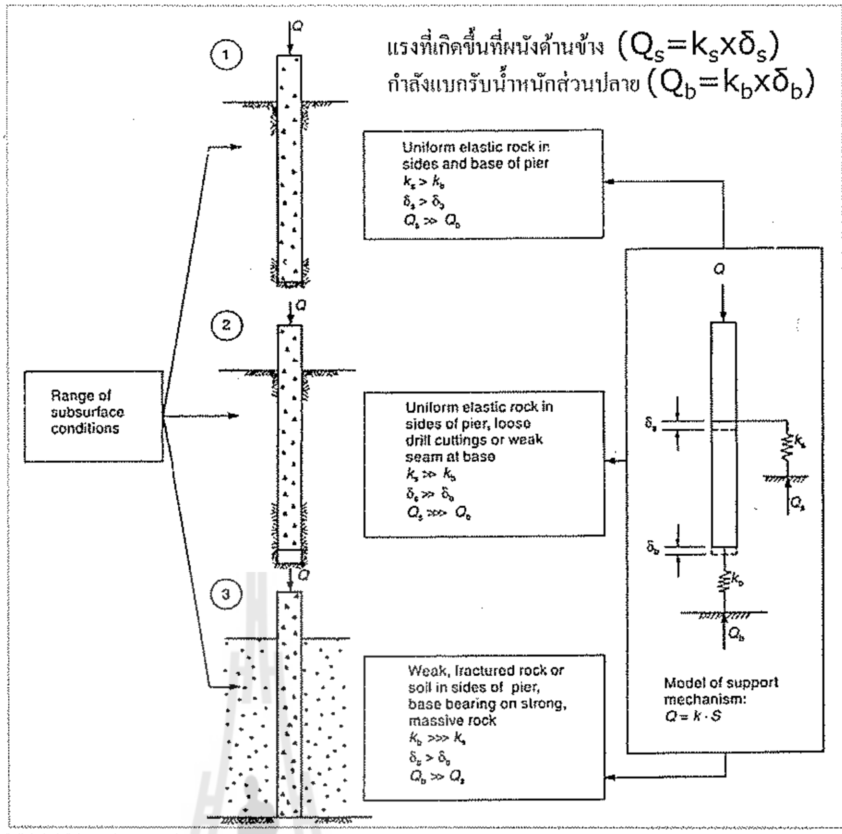
- Side wall shear strength adhesion or skin friction
- End bearing
- Combination of both



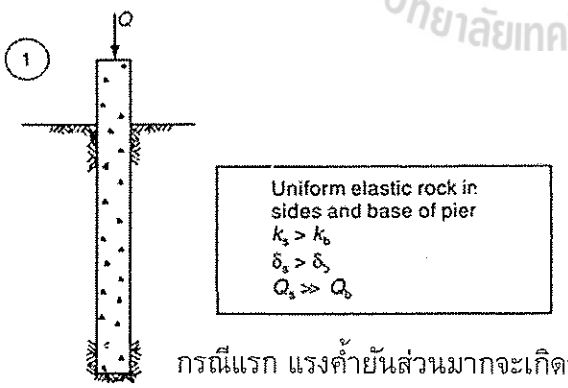
Mechanism of Load Transfer

ขนาดของแรงค้ำยันที่เกิดแรงเฉือนด้านข้างของผนังและกำลังแบกรับน้ำหนักบรรทุกที่ปลาย ขึ้นอยู่กับ

1) ค่าสัมประสิทธิ์ความยืดหยุ่นของวัสดุที่ pier นั้นฝังอยู่และค่าความยืดหยุ่นของตัว pier เอง
2) ขนาดของแรงที่มีความสัมพันธ์กับกำลังรับแรงเฉือนที่ด้านข้างของผนัง และ
3) วิธีที่ใช้ในการก่อสร้างกลไกของการถ่ายแรงและการหลุดตัวของ pier รวมทั้งการกระจายตัวของแรงระหว่างผนังด้านข้างของหลุมเจาะและค่ากำลังแบกรับน้ำหนักบรรทุกที่ส่วนปลาย

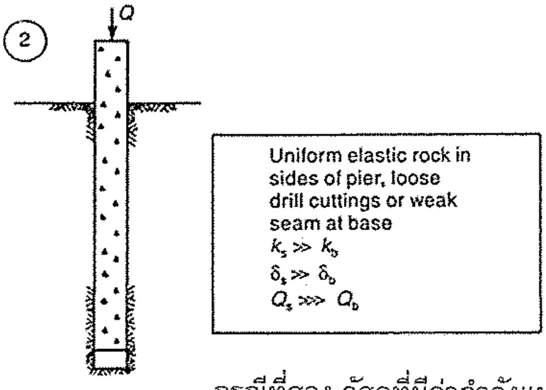


Mechanism of Load Transfer



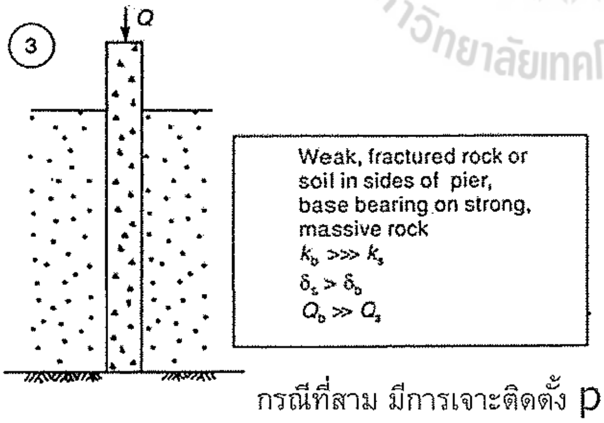
กรณีแรก แรงค้ำยันส่วนมากจะเกิดขึ้นที่ส่วนบนของ pier กล่าวคือ แรงต้านด้านข้างของผนังต่อหนึ่งหน่วยของการเคลื่อนตัวมีค่ามากกว่าแรงต้านที่เกิดขึ้นบริเวณส่วนปลายซึ่งมีค่าการเคลื่อนตัวที่เท่ากัน ดังนั้น ค่าคงที่ของความยืดหยุ่น k_s มีความเหนียวมากกว่าค่าคงที่ความยืดหยุ่นที่ฐาน k_b การโค้งงอของ pier เกิดจากความยืดหยุ่นที่ไม่เหมาะสมของ pier และการโค้งงอที่ส่วนปลาย เนื่องจากการโค้งงอส่วนมากเกิดขึ้นที่ส่วนด้านบนของ pier นั่นคือ δ_s มีค่ามากกว่า δ_b แรงเฉือนที่เกิดขึ้นที่ผนังด้านข้างมีค่ามากกว่าแรงต้านที่ส่วนปลายของ pier

Mechanism of Load Transfer



กรณีที่สอง วัสดุที่มีค่ากำลังแบกรับน้ำหนักต่ำเกิดขึ้นที่ฐานของ pier ดังนั้น ค่าความยืดหยุ่นคงที่ k_b จึงมีความเหนียวน้อยกว่าค่าความยืดหยุ่นคงที่ k_s มาก นั่นคือ แรงที่กระทำมีค่าไม่เกินกำลังรับแรงเฉือนของผนังด้านข้าง การเคลื่อนตัวส่วนมากเกิดขึ้นที่ส่วนบนของ pier และส่วนหลักของแรงเกิดขึ้นในแรงเฉือนของผนังด้านข้าง

Mechanism of Load Transfer



กรณีที่สาม มีการเจาะติดตั้ง pier ผ่านวัสดุที่มีค่า modulus ต่ำลงไปอยู่ในชั้นที่มีค่า modulus สูงกว่า ดังนั้น ค่าคงที่ความยืดหยุ่น k_b มีค่ามากกว่าค่าคงที่ยืดหยุ่น k_s มาก ในกรณีนี้จะเกิดการเคลื่อนที่มาก เนื่องจากค่าความยืดหยุ่นที่ไม่เหมาะสมของ pier และเกิดเล็กน้อยเนื่องจากการเบี่ยงเบนในวัสดุที่มีค่าความยืดหยุ่นสูงกว่า ที่อยู่ใต้ฐานของ pier ในสภาวะเช่นนี้จะเกิดแรงมากที่ส่วนปลายของ pier

Shear behavior of rock sockets

- Morh-Coulomb criterion

  $\tau = c + \sigma_n \tan \phi$

- If displacement of pier exceeds the elastic limit of the interface -> c=0 & $\phi = \phi_{residaul}$

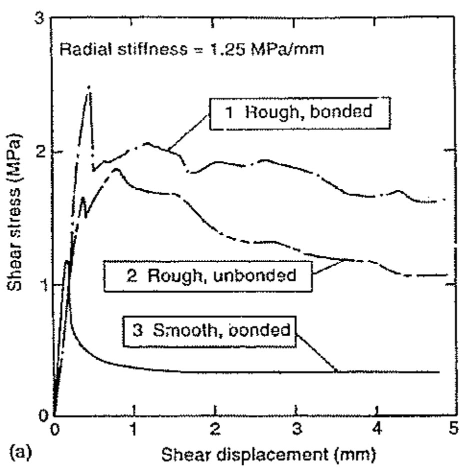
  $\tau = \sigma_n \tan \phi_{res}$

- Normal stress at rock-concrete interface is induced by two mechanism.
  1. Application of compressive load on top of pier results in elastic dilation of concrete
  2. Shear displacement at rough surface of dill hole results in mechanical dilation of the interface

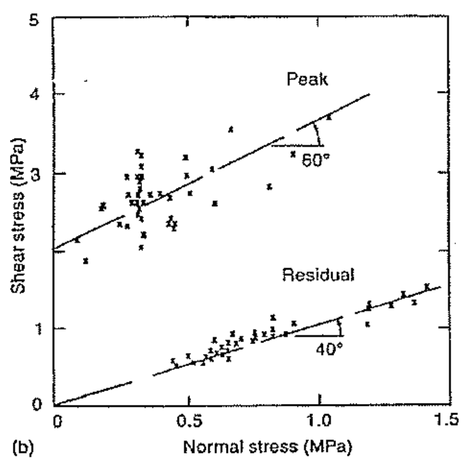
Shear behavior of rock sockets

การทดสอบพฤติกรรมแรงเฉือนของรอยสัมผัสของหิน – คอนกรีตในเครื่องมือการทดสอบ

Constant normal stiffness (Ooi และ Carter, 1987)



Shear stress – displacement curves

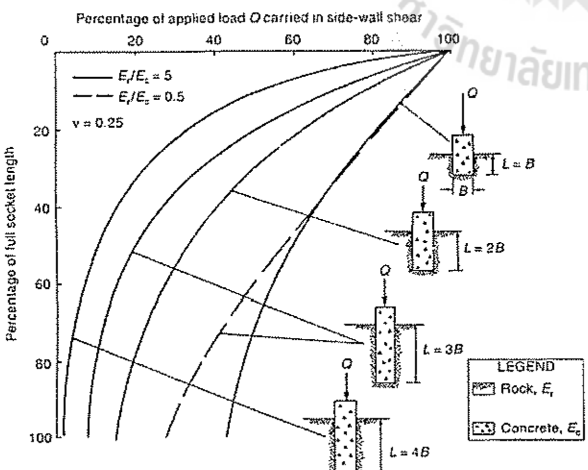


Peak and residual strength envelopes

Factors affecting Load Capacity

- Geometry of piers
- Elastic modulus of rock around pier and below pier
- Strength around pier and below pier
- Condition of side wall
- Condition of end of pier
- Layering rock
- Settlement of pier
- Creep

1) Effect of socket geometry



Recommended
L/B ratio ≥ 4

อัตราส่วนระหว่างความยาวต่อเส้นผ่าศูนย์กลางมีผลกระทบต่อกำลังรับน้ำหนักของ pier อย่างมีนัยสำคัญ โดยอัตราส่วนนี้มีการเพิ่มขึ้นจากศูนย์ โดยแรงในส่วน end bearing จะมีค่าลดน้อยลงไป และมีการเพิ่มขึ้นในส่วนของแรงเฉือนทางด้านข้างของผนัง
ในสภาวะที่หินมีค่า modulus มากกว่า pier แรงเฉือนทั้งหมดจะเกิดขึ้นที่ผนังด้านข้างด้วยอัตราส่วน L/B=4 ในขณะที่มีแรงเฉือนด้านข้างของผนังเกิดขึ้นเพียงร้อยละ 50 เมื่อมีอัตราส่วน L/B=1 นั่นหมายความว่า pier สั้นที่วางอยู่บนหินแข็งบนฐานของ pier มีความมั่นคงในการรับแรง ส่วนใน pier ยาวจะมีแรงเพียงเล็กน้อย

Distribution of side-wall shear stress (after Osterberg and Gill, 1973)

2) Effect of rock modulus

Increase in normal load

(Seidel and Haberfield, 1994)

$$\Delta\sigma = \frac{E_{(m)}}{(1+\nu_{(m)})}\frac{\Delta r}{r}$$

where:

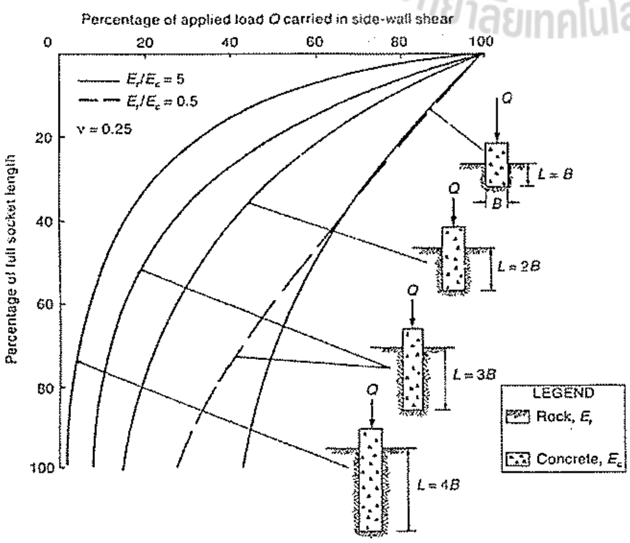
$E_{(m)}$ = rock mass modulus

$\nu_{(m)}$ = rock mass Poisson's ratio

R =radius of pier

Δr = change in radius of pier

2) Effect of rock modulus



การกระจายตัวของแรงเฉือนตามผิวของผนังด้านข้างของ pier เมื่อหินมีค่าความยืดหยุ่นสูงกว่าคอนกรีต (E_r/E_c =5) pier จะถูกกักและแรงกดจะมีค่ามากที่ผนังด้านข้าง ซึ่งส่งผลให้มีแรงกระทำมากในส่วนบนของ pier ในทางตรงกันข้ามถ้าหินมีค่าความยืดหยุ่นต่ำกว่าคอนกรีต (E_r/E_c =0.5) ค่าแรงกดจะมีค่าลดน้อยลง และมีแรงเฉือนเพียงเล็กน้อยเกิดขึ้นที่ผนังด้านข้างของ pier ผลของการลดค่าความยืดหยุ่นด้วยการลดขนาดอย่างเป็นสัดส่วน ทำให้ แรงเฉือนมีการกระจายตัวลงไปใน pier ที่สม่ำเสมอมากขึ้น และแรงที่ฐานจะเพิ่มขึ้นร้อยละ 8 ถึง 30 ของแรงที่กระทำ

2) Effect of rock modulus

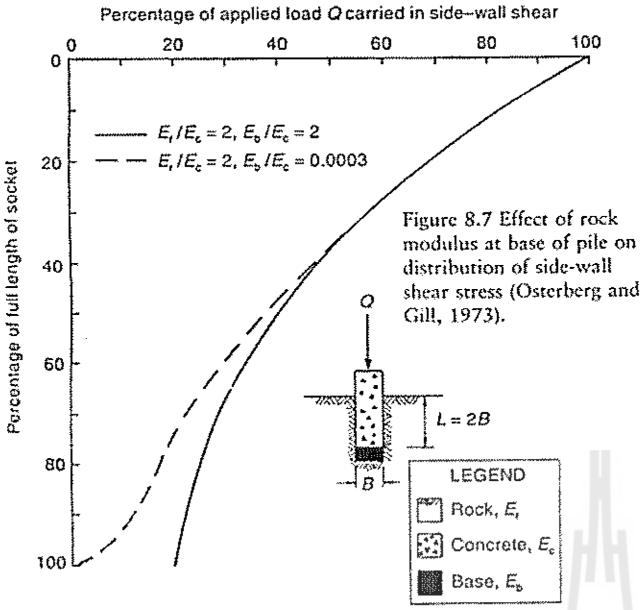
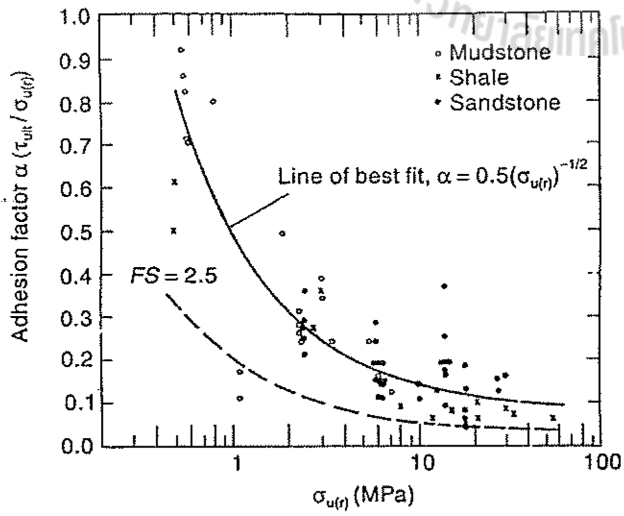


Figure 8.7 Effect of rock modulus at base of pile on distribution of side-wall shear stress (Osterberg and Gill, 1973).

การกระจายตัวของแรงเค้นลงตาม pier ก็ได้รับอิทธิพลจากค่า Deformation modulus ของหินที่ฐานของ pier เช่นกัน ถ้าหินมีค่าความยืดหยุ่นน้อยจะสามารถรับแรงที่ฐานได้เพียงเล็กน้อย

ความแตกต่างการกระจายตัวของแรงเค้นสองแบบซึ่งขึ้นอยู่กับ relative modulus ของหินใน pier และที่ต่ำกว่าฐาน pier ในหินที่มีค่า modulus ต่ำจะมีค่ากำลังแบกรับน้ำหนักลดลง เมื่อเปรียบเทียบกับ pier ที่มีหินแข็งที่ฐาน

3) Effect of rock strength



$$(\tau_{ult}/\sigma_{u(r)}) = 0.5\ (\sigma_{u(r)})^{-0.5}$$

τ_{ult} = side wall shear strength

$\sigma_{u(r)}$ = Uniaxial compressive strength

$$\sigma_{u(r)} \uparrow \rightarrow \tau_{ult} \uparrow$$

(Williams and Pell, 1981)

4) Condition of side walls

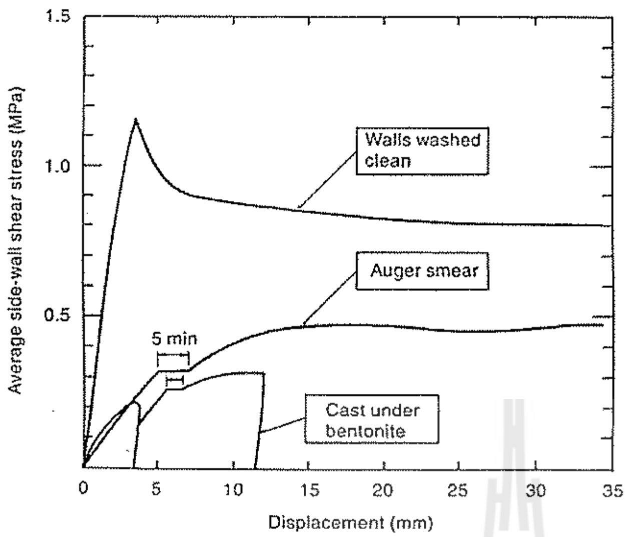


Figure 8.11 Influence of side-wall condition on socket shear strength (Williams and Pells, 1981, courtesy of Research Journals. National Research Council Canada).

5) Condition of end of socket

- Must be cleaned of all drill cutting and loose rock
- If not possible to clean and inspect, it may be necessary to assume that there is no end bearing (fully load inside-wall shear)

6) Layering in the rock

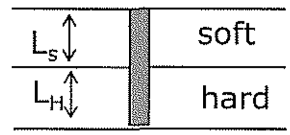
Effective side-wall shear resistance

$$\tau^* = p\tau_s + (1-p)\tau_r$$

Effective side-wall modulus

$$E^* = pE_s + (1-p)E_r$$

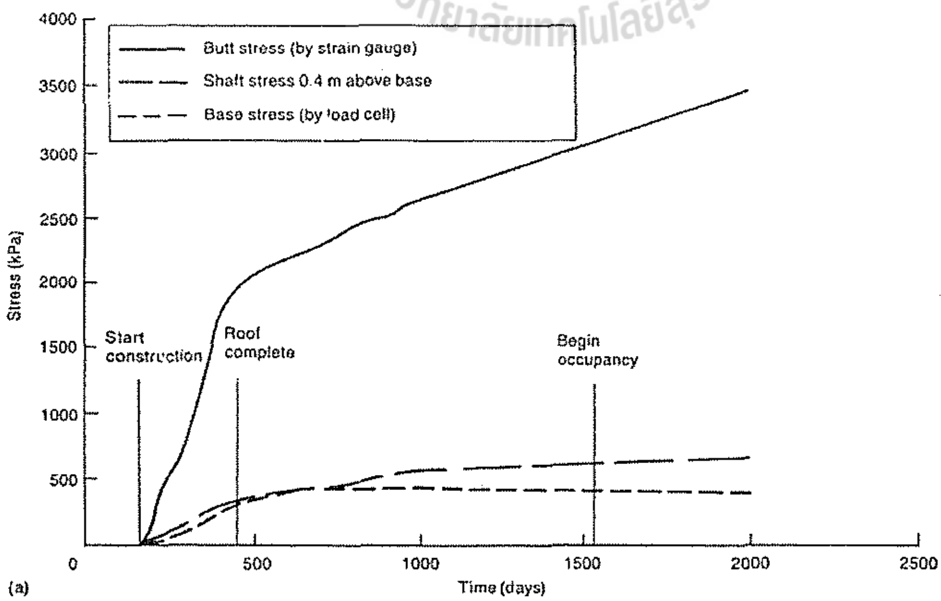
$p = L_s/L_H$



p = proportion of the shaft which consists of low strength material
Subscript s = low strength material of side-wall
Subscript r = high strength material of side-wall

(Rowe and Armitage, 1978; Thorne, 1980)

7) Creep



Design of side-wall resistance

$$Q = \tau_a \pi B L$$

Q = total applied load
τ_a = allowable side wall shear stress
B= diameter of socket
L = length of socket

For clean socket

Empirical equation

Side-wall undulation (ลอนคลื่น) b/w 1-10 mm deep and <10 mm wide

$$\tau_a = \frac{0.6(\sigma_{u(r)})^{0.5}}{FS}$$

Side-wall undulation >10 mm deep and >10 mm wide

$$\tau_a = \frac{0.75(\sigma_{u(r)})^{0.5}}{FS}$$

(Rowe and Armitage, 1978)

Design of end bearing

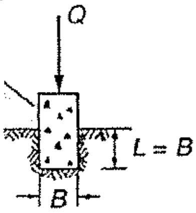
$$Q_a = \sigma_{u(r)} \frac{\pi B^2}{4}$$

Empirical equation

Q_a = Allowable load capacity with includes a F.S. 2-3
$\sigma_{u(r)}$ = Uniaxial compressive strength

Conditions:

1. Base of socket is at least one diameter below the ground surface
2. Rock to depth of at least one diameter below the base of socket is either intact or tightly jointed (no gouge filled seems).
3. There no solution cavities or voids below base of pier.



(Rowe and Armitage, 1978)

Design of end bearing

For conditions where
Rock below pier contains horizontal/near horizontal seams infilling with low strength material

$$Q_a = K'\omega\sigma_{u(r)}$$

where

$$\text{Empirical factor, } K' = \frac{\left(3 + \frac{S}{B}\right)}{10\left(1 + 300\frac{t}{S}\right)^{1/2}}$$

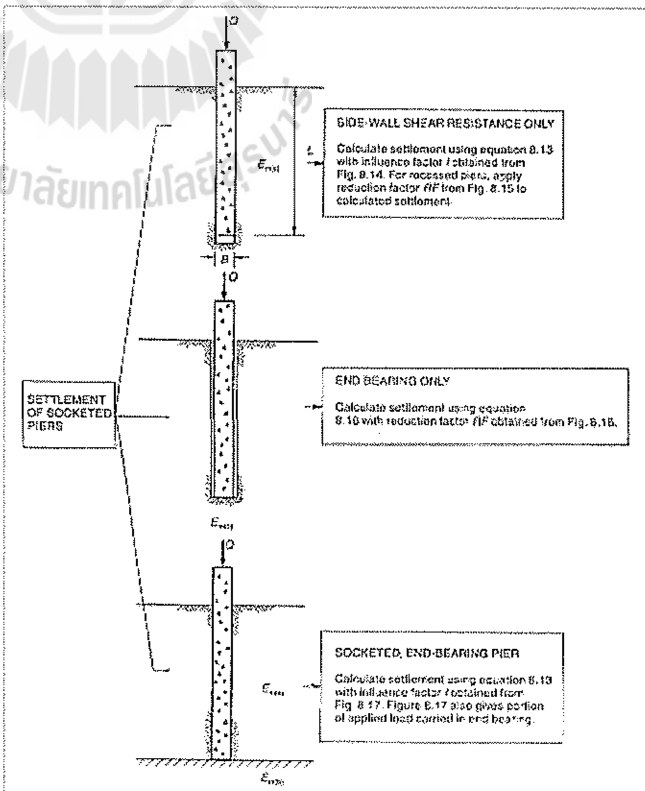
$$\text{Depth factor, } \omega = 1 + \frac{0.4L}{B}$$

Empirical equation

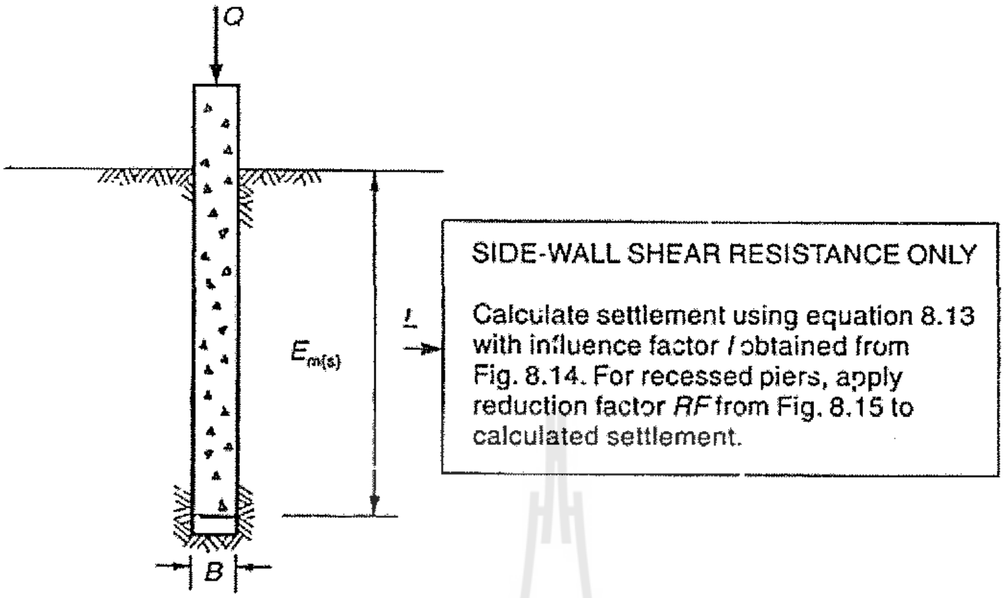
L = socket length
B = socket diameter
S = seam spacing
t = seam thickness

(Canadian Geotechnical Society, 1992)

Axial Deformation



Axial Deformation

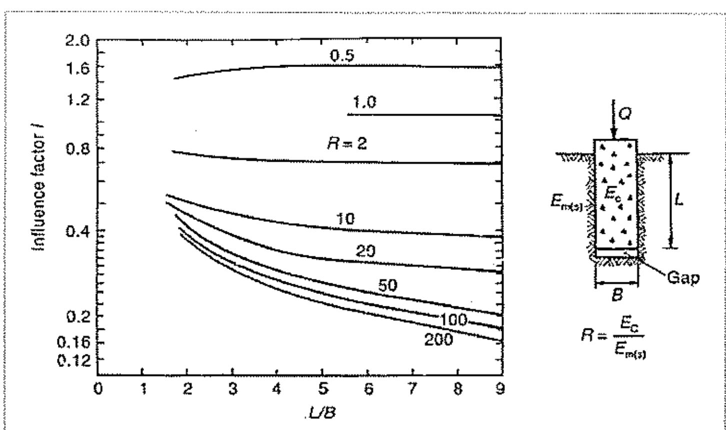


Axial Deformation

1) Settlement of side-wall resistance sockets

$$\delta = \frac{QI}{BE_{m(s)}}$$

Q = applied load
B = diameter of socket
$E_{m(s)}$ = rock mass deformation modulus
I = settlement influence factor



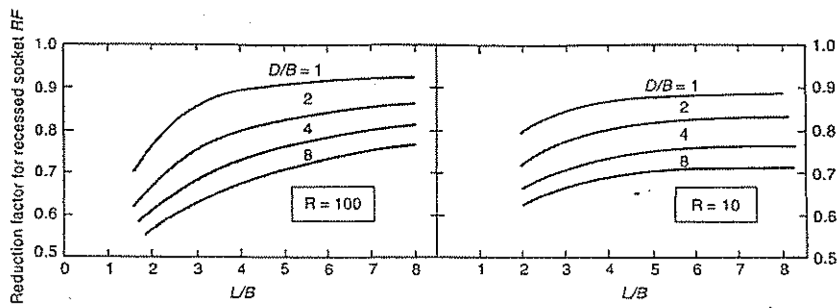
$$E_{m(s)} = 110\sqrt{\sigma_{u(r)}}$$

(Rowe and Armitage, 1978)

(Pells and Turner, 1979)

Axial Deformation

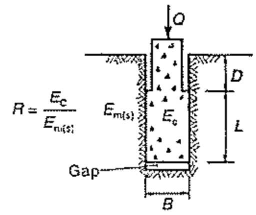
1) Settlement of side-wall resistance sockets



For recessed socket

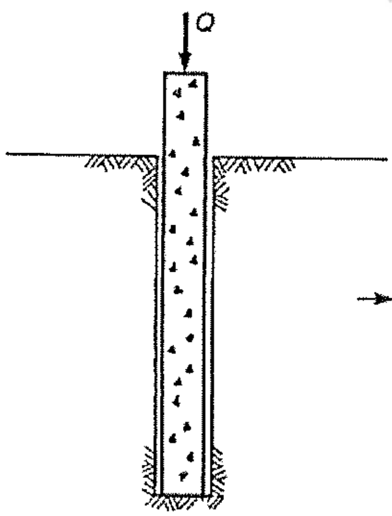
$$\delta = RF\frac{QI}{BE_{m(s)}}$$

where RF is a reduction factor



(Pells and Turner, 1979)

Axial Deformation



END BEARING ONLY

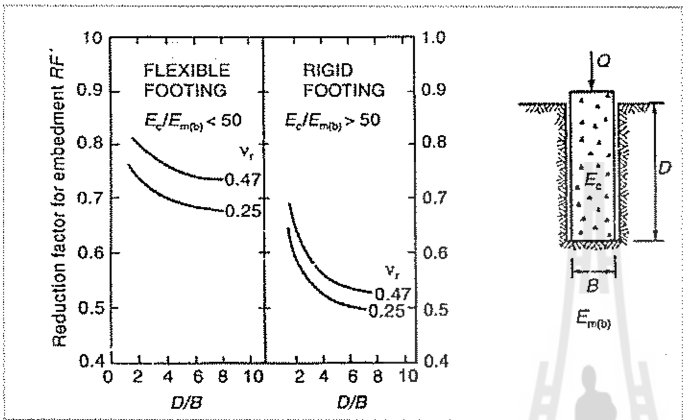
Calculate settlement using equation 8.16 with reduction factor *RF* obtained from Fig. 8.16.

Axial Deformation

2) Settlement of end bearing

$$\delta = \frac{4Q}{\pi B^2}\left[\frac{D}{E_c} + \frac{RF' C_d B(1-\nu^2)}{E_{m(b)}}\right]$$

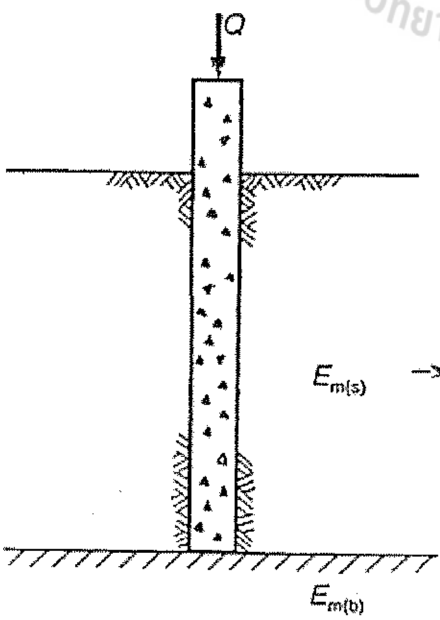
E_c = concrete modulus
RF′ = reduction factor
D = depth of pier
C_d = shape and rigidity factor
Q = foundation load
B = pier diameter
$E_{m(b)}$ = deformation modulus of rock mass



(pier มักมีรูปร่างเป็นวงกลม ค่าเฉลี่ยของการทรุดตัวสำหรับ flexible footing C_d คือ 0.85 และ rigid footing C_d เท่ากับ 0.79)

(Pells and Turner, 1979)

Axial Deformation



SOCKETED, END-BEARING PIER

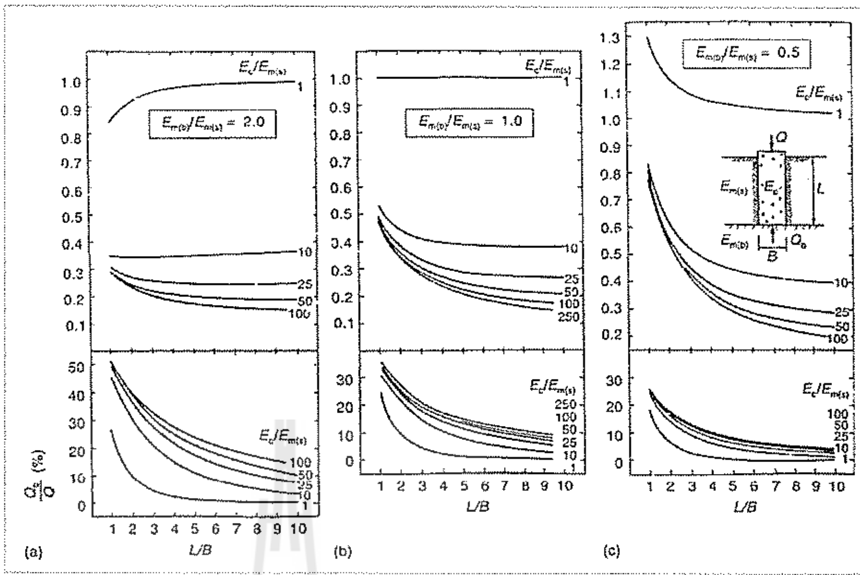
Calculate settlement using equation 8.13 with influence factor *I* obtained from Fig. 8.17. Figure 8.17 also gives portion of applied load carried in end bearing.

Axial Deformation

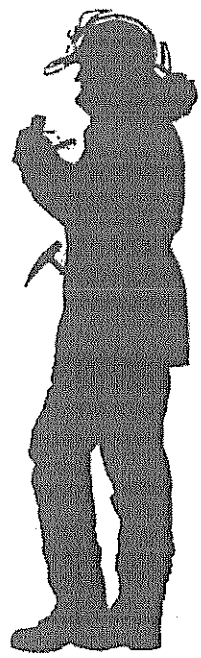
3) Settlement of socketed, end bearing pire

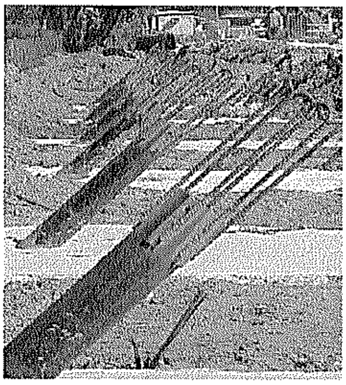
$$\delta = \frac{QI}{BE_{m(s)}}$$

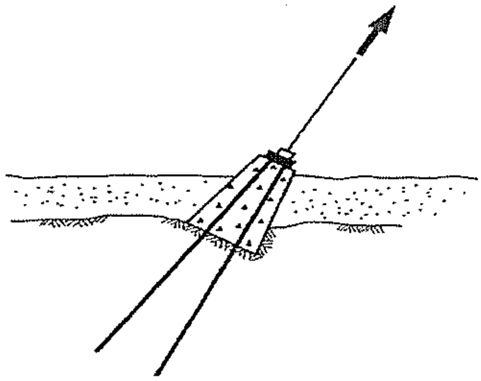
Q = applied load
B = diameter of socket
$E_{m(s)}$ = rock mass deformation modulus
I = settlement influence factor



(after Rowe and Armitage, 1978)







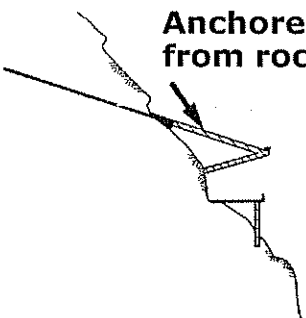
434636 Foundations on Rock

Topic 9 Tension Foundations

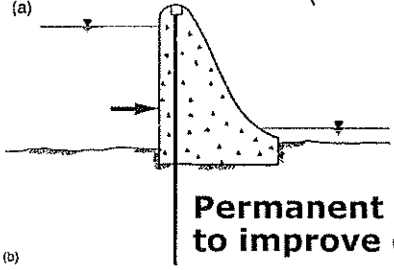
Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

Tension Foundations

Anchored roof to protect roadway from rock fall

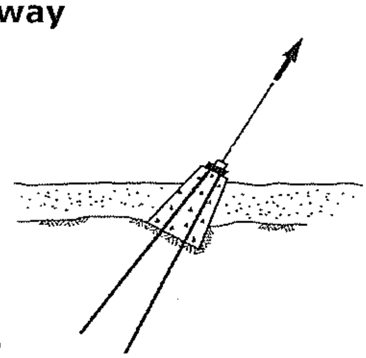


(a)



(b)

Permanent tie-down install to improve overturning of dam



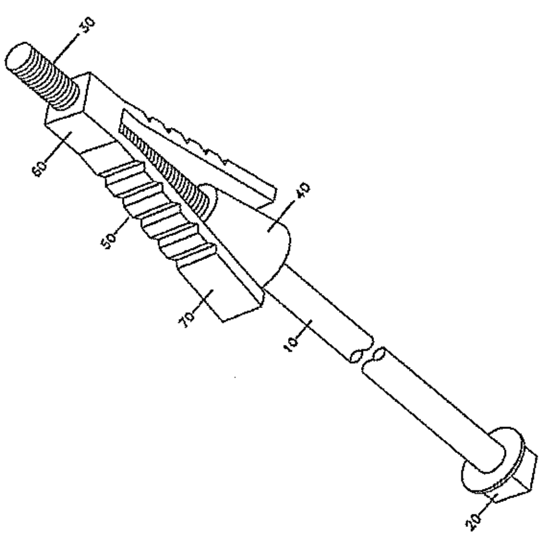
(c)

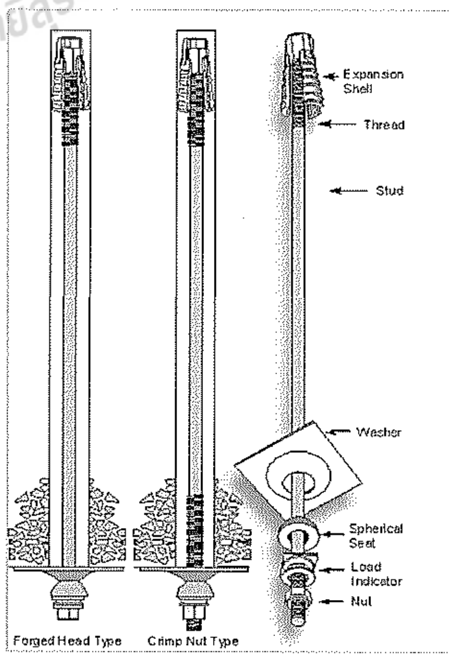
Rock anchor providing support for tensioned cable

Anchorage (Rock Bolt)

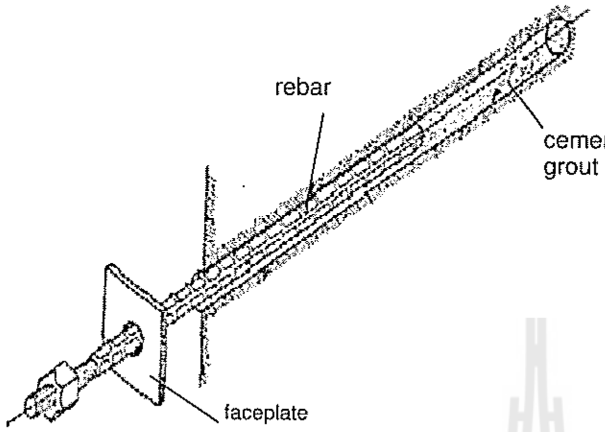
- Mechanical Anchorage
- Cement Grout Anchorage
- Resin Grout Anchorage

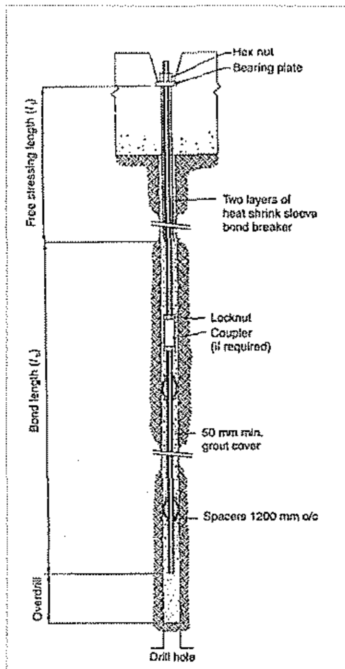
Mechanical Anchorage



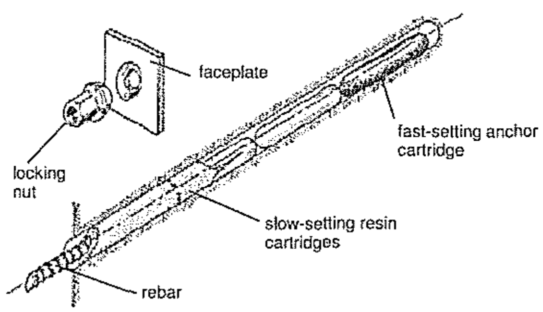


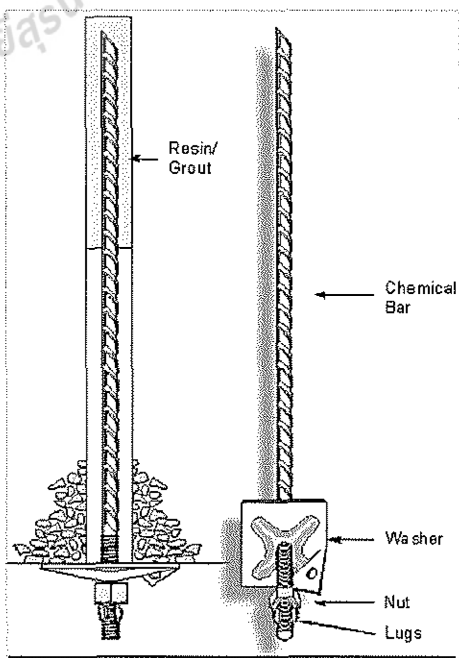
Cement Grout Anchorage



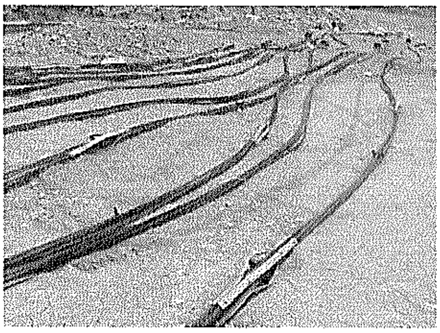


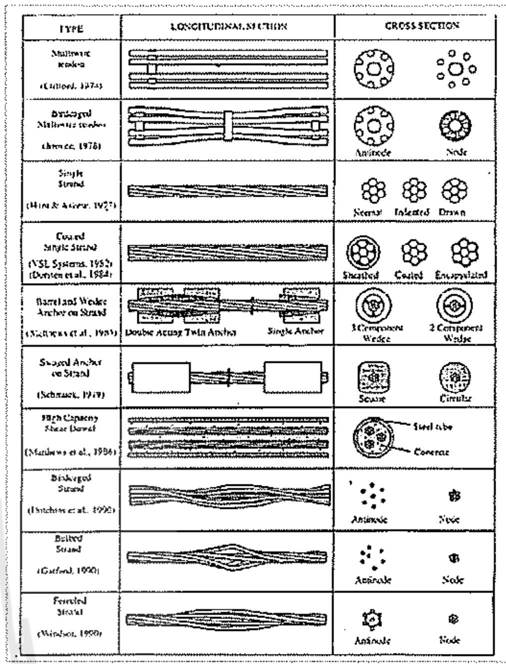
Resin Grout Anchorage





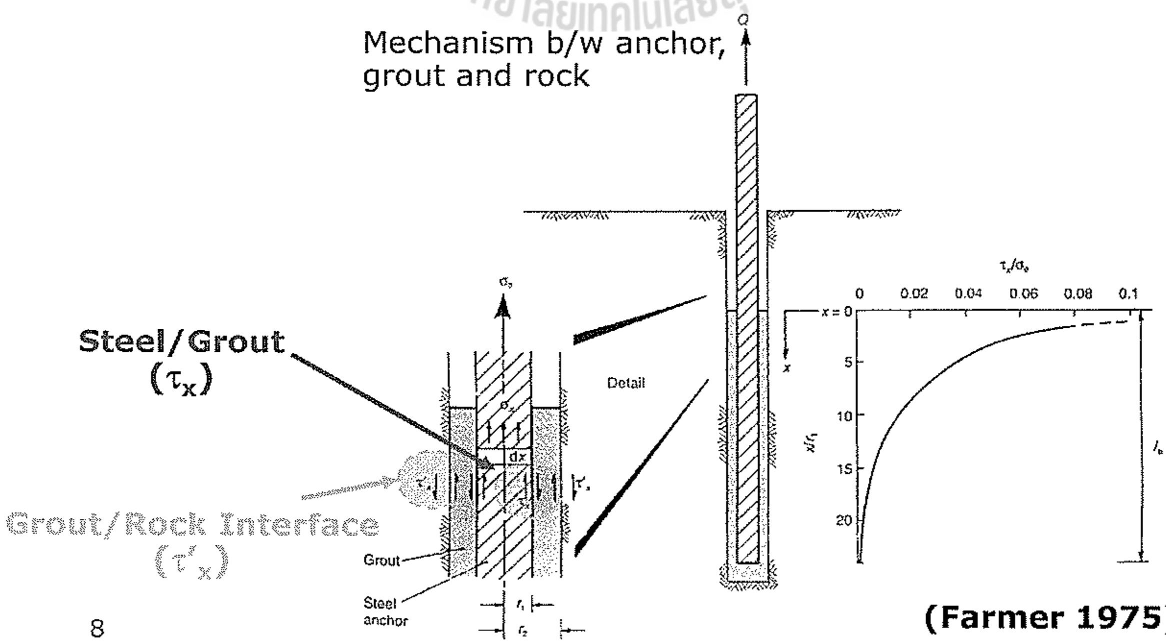
Cable Bolt





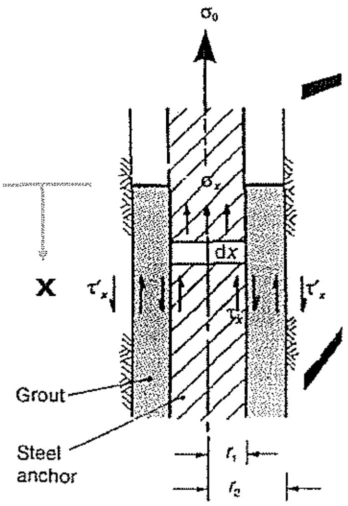
Mechanics of Load Transfer

Mechanism b/w anchor, grout and rock



(Farmer 1975)

Mechanics of Load Transfer



Shear Stress distribution b/w Steel & Grout

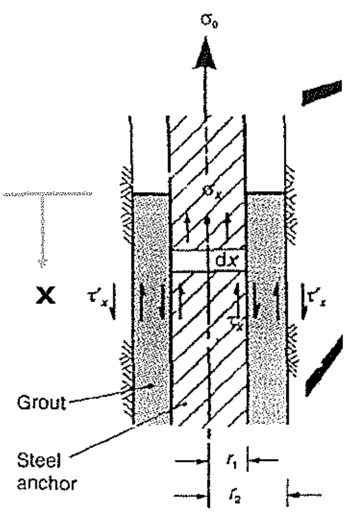
$$\tau_x = \frac{1}{2} r_1 \Omega \sigma_0 e^{-\Omega x}$$

Assumptions:
-Elastic behavior (steel, grout and rock)
-No slippage at the interface

where:
r_1 = radius of bolt
σ_0 = applied normal stress
x = distance from proximal end of bond length

(Farmer 1975)

Mechanics of Load Transfer



Shear Stress b/w Steel & Grout

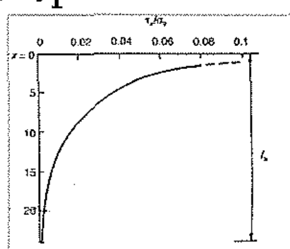
$$\tau_x = \frac{1}{2} r_1 \Omega \sigma_0 e^{-\Omega x}$$

Thin grout annulus, (r_2-r_1) < r_1

$$\Omega = \left[\frac{R}{r_1(r_2 - r_1)}\right]^{1/2}$$

Thick grout annulus, (r_2-r_1) > r_1

$$\Omega = \left[\frac{R}{r_1^2 \ln(r_2/r_1)}\right]^{1/2}$$



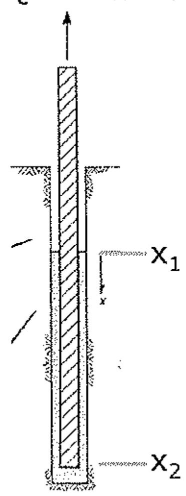
where:
$R = E_g/E_b$
E_g = Elastic Modulus of Grout
E_b = Elactic Modulus of Bolt
r_2 = radius of drill hole

(Farmer 1975)

Mechanics of Load Transfer

Total load Q carried by anchorage b/w any point (x_1 and x_2)

Q = total load

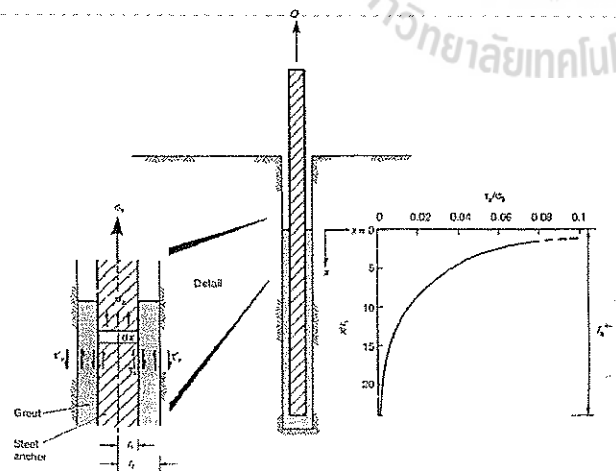


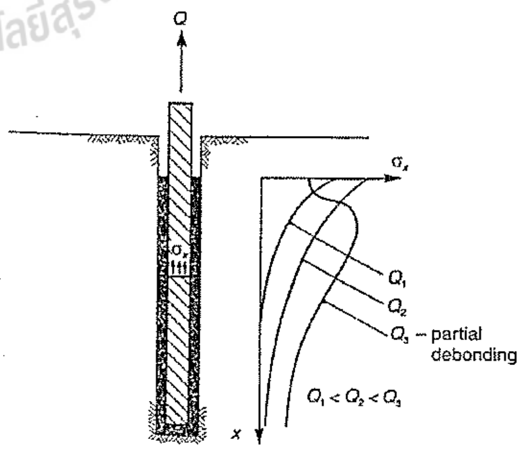
Integration of equation $\tau_x = \frac{1}{2} r_1 \Omega \sigma_0 e^{-\Omega x}$

$$
\begin{aligned}
Q &= 2\pi r_1 \int_{x_1}^{x_2} \tau_x \, dx \\
&= \pi r_1^2 \Omega \sigma_0 \int_{x_1}^{x_2} e^{-\Omega x} \, dx \\
&= -\pi r_1^2 \sigma_0 \left[e^{-\Omega x} \right]_{x_1}^{x_2} \\
&= \pi r_1^2 \sigma_0 \left[e^{-\Omega x_1} - e^{-\Omega x_2} \right]
\end{aligned}
$$

(Farmer 1975)

Allowable bond stresses and anchor design





The typical distributions of shear stress along anchor length demonstrate **non-linear** nature of distribution.

Allowable bond stresses and anchor design

- Exact form of distribution (non-linear) is difficult to predict for wide range of conditions.

- To simplify assumption for design proposes:
  - Uniform shear stress distribution along bond length
  - Magnitude of average shear stress (both rock-grout and grout-steel interfaces) has been established empirically from results of tests on full-scale and laboratory anchors.

- $\tau_{rock\text{-}grout} \leq \tau_{grout\text{-}steel}$

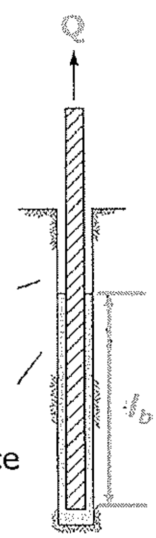
Allowable bond stresses and anchor design

- Assuming that shear stress is uniformly distribution

Bond Length (l_b)

$$l_b = \frac{Q}{\pi d \tau_a}$$

where:
Q = applied tensile load
d= diameter of drill hole
τ_a = working bond strength of rock-grout interface



Allowable bond stresses and anchor design

- Approximation relationship b/w rock-grout bond strength and UCS (Littlejohn and Bruce, 1977)

Working bond strength (design value with F.S. =3)

$$\tau_a \approx \frac{\sigma_{u(r)}}{30}$$

Ultimate bond strength

$$\tau_u \approx \frac{\sigma_{u(r)}}{10}$$

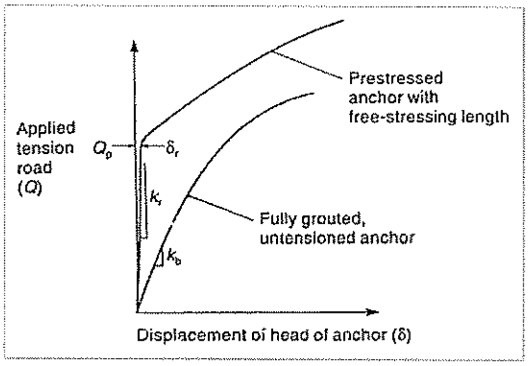
where: $\sigma_{u(r)}$ = Uniaxial compressive strength of rock in bond zone

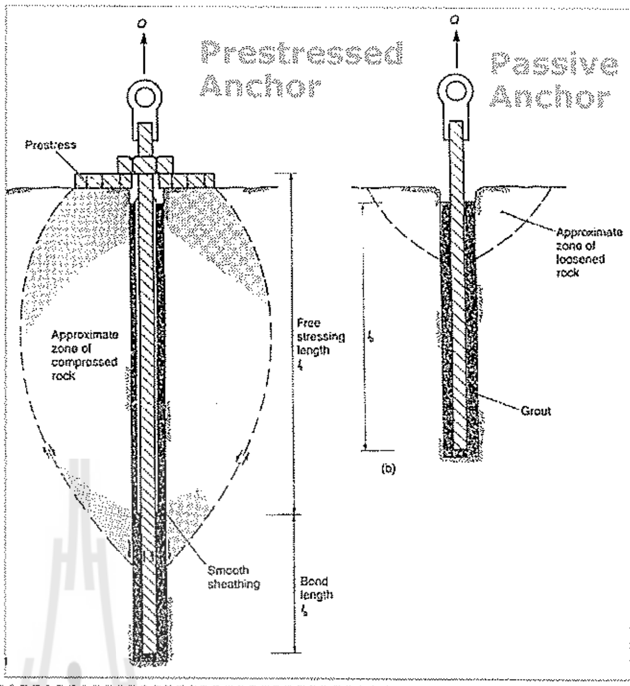
Allowable bond stresses and anchor design

Table 9.2 Approximate relationship between rock type and working bond shear strength for cement grout anchorages

Rock type	*Working bond stress τ_a at rock–grout interface*	
	MPa	p.s.i.
Granite, basalt	0.55–1.0	80–150
Dolomitic limestone	0.45–0.70	70–100
Soft limestone	0.35–0.50	50–70
Slates, strong shales	0.30–0.45	40–70
Weak shales	0.05–0.30	10–40
Sandstone	0.30–0.60	40–80
Concrete	0.45–0.90	70–130
Weak rock	0.35–0.70	50–100
Medium rock	0.70–1.05	100–150
Strong rock	1.05–1.40	150–200

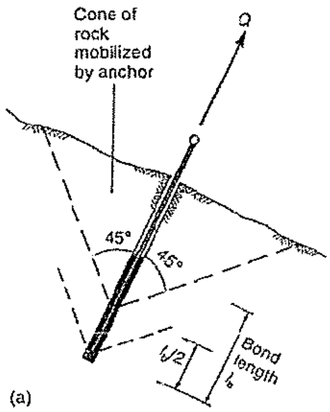
Prestressed and passive anchors



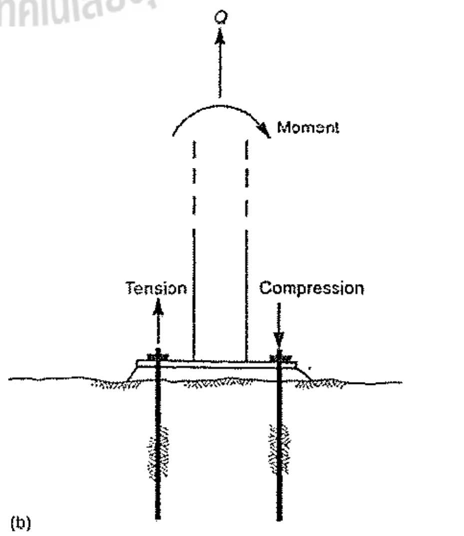


Uplift capacity of rock anchors

Failure Modes

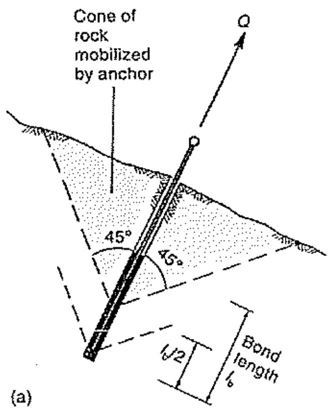


Pure Tension Loading



Combined Moment and Tension Loading

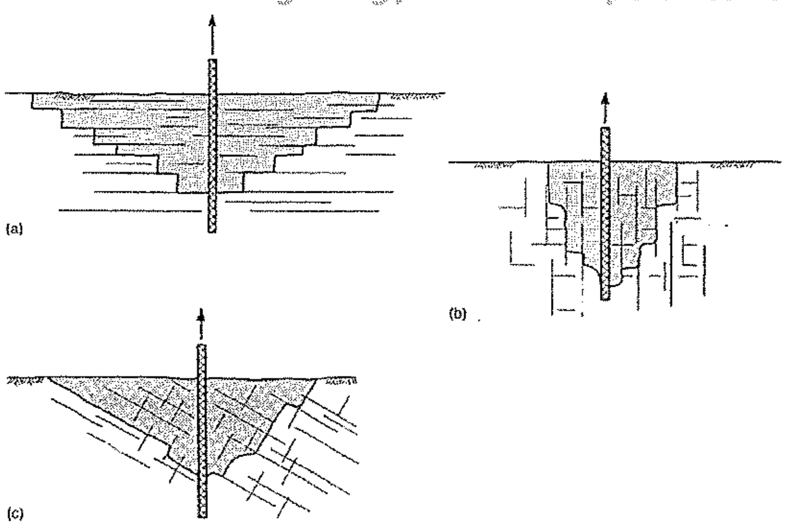
Pure Tension Loading



<u>Several possible failure modes</u>:
-failure in steel
-failure in rock-grout bond
-failure in grout-steel interface
-failure in **cone of rock**

Pure Tension Loading

Influence of structural geology on the shape of cone

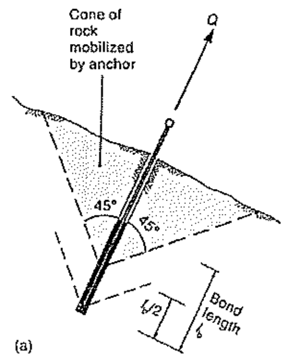


Pure Tension Loading

Estimate tensile strength of fracture rock:
(Hoek and Brown Criterion)

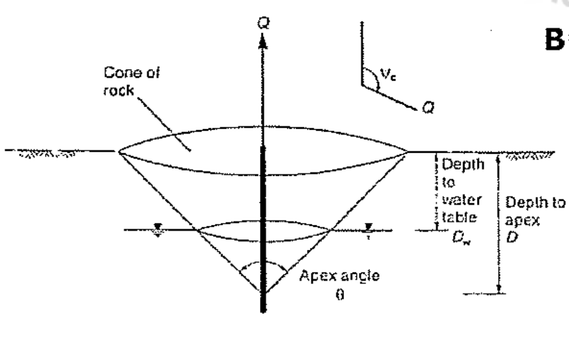
$$\sigma_t = \frac{\sigma_{u(r)}}{2}\left[m - (m^2 + 4s)^{1/2}\right]\frac{1}{FS}$$

where:
σ_t = working tensile strength on surface of cone
$\sigma_{u(r)}$ = UCS of rock
m, s = rock mass constant



Pure Tension Loading

Cone of rock mobilized by tie-down anchor to resist uplift load



Buoyant Weight, W_c

$$W_c = \frac{\pi}{3}\tan^2\left(\frac{\theta}{2}\right)\left[D^3\gamma_r - (D - D_w)^3\gamma_w\right]$$

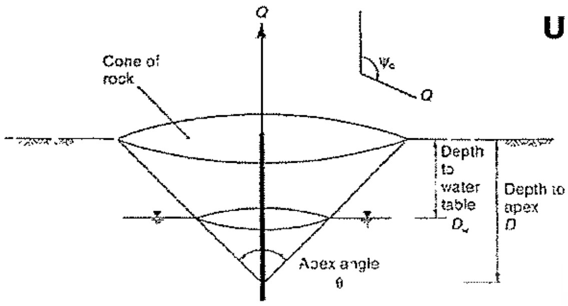
where:
D = depth of apex below ground surface
D_w = depth of water table
γ_r = rock unit weight
γ_w = water unit weight
θ = apex angle of cone (assume = 90 deg.)

Resisting force developed on curve surface area

$$f_{(r)} = \frac{\sigma_t \pi D^2 \tan(\theta/2)}{\cos(\theta/2)}$$

Pure Tension Loading

Cone of rock mobilized by tie-down anchor to resist uplift load

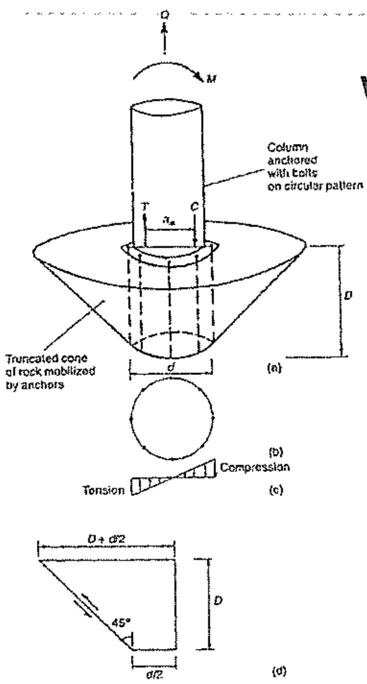


Uplift capacity, Q

$$Q = \frac{(f_{(r)} + W_c \cos \Psi_c)}{FS}$$

where:
Ψ_c = angle b/w vertical upwards directio
and load direction

Combined Moment and Tension Loading

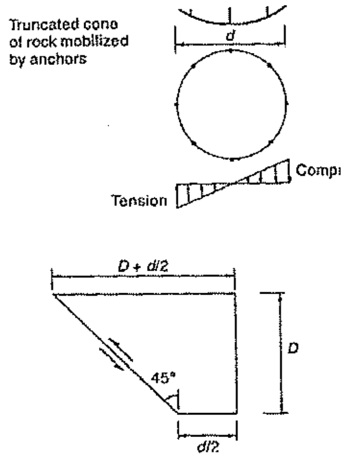


Weight of mass of rock in truncated cone, W'_c

$$W'_c = \frac{\pi}{3}\left\{\left[\left(D+\frac{d}{2}\right)^3 - \left(\frac{d}{2}\right)^3\right]\gamma_r - \left[\left(D - D_w + \frac{d}{2}\right)^3 - \left(\frac{d}{2}\right)^3\right]\gamma_w\right\}$$

where:
D = depth of truncated cone
D_w = depth of water table
d = diameter of circular of anchor bolts
γ_r = rock unit weight
γ_w = water unit weight
assume apex angle of cone, θ = 90 deg.

Combined Moment and Tension Loading



For a symmetrical distribution of tension and compression

Surface area of one half of truncated cone (ignoring horizontal base of cone)

$$A'_c = \frac{\pi}{\sqrt{2}}(D^2 + dD)$$

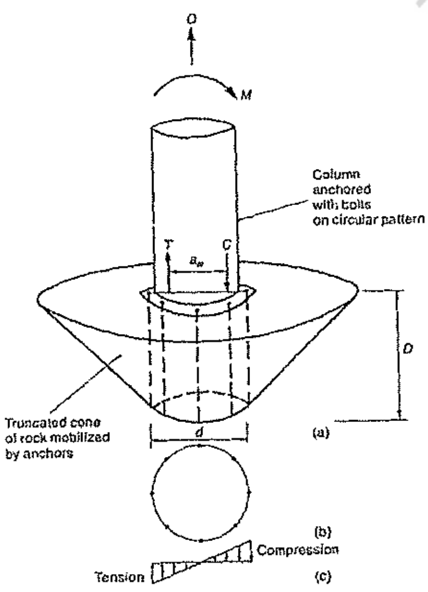
Resisting force:

$$f'_{(r)} = \sigma_t A'_c$$

Tensile strength of rock

Section through uplift position of cone

Combined Moment and Tension Loading



Magnitude of force T: (by taking moment)

$$T = \frac{M}{(a_m/2)} = \frac{3M}{d} \quad \text{when } a_m = 0.67d$$

Load capacity of tower foundation:

$$(T \pm Q) = \frac{(W'_c + f'_{(r)})}{FS}$$

$+Q$ vertical force upwards in same direction as tension force induced by the moment;
$-Q$ vertical force downwards.