เอกสารประกอบการสอน

434422 Surface Excavation and Design

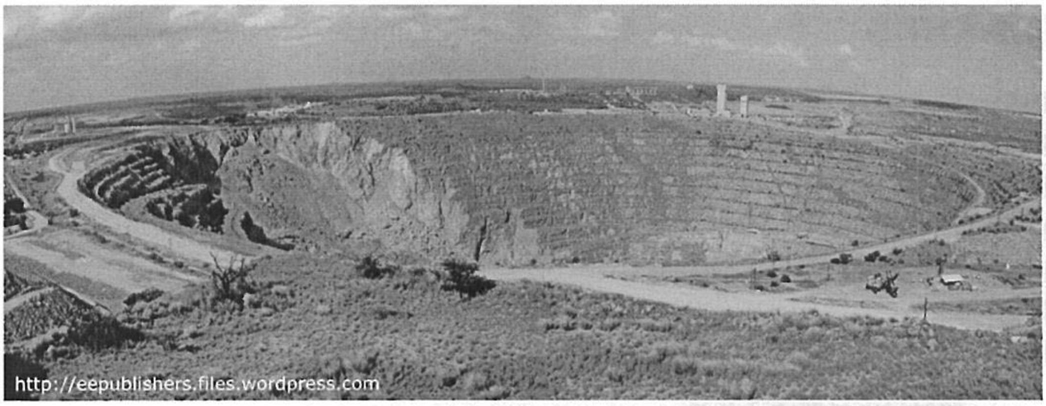
prepared by

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

Geological Engineering Program
Suranaree University of Technology

Disclaimer

This document has been prepared for use as a lecture note for the subject indicated above. The contents have been complied from relevant text books and technical papers, with a main emphasis on the teaching methodology and learning step on the subject. The author does not claim the originality of the presented materials (e.g., theories, formula, illustrations & tables). The document is not intended to be a technical publication. It serves as an internal document, and hence should not be distributed nor sold to publics.



434422 Surface Excavation & Design
4 credits

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

434422 Surface Excavation and Design

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

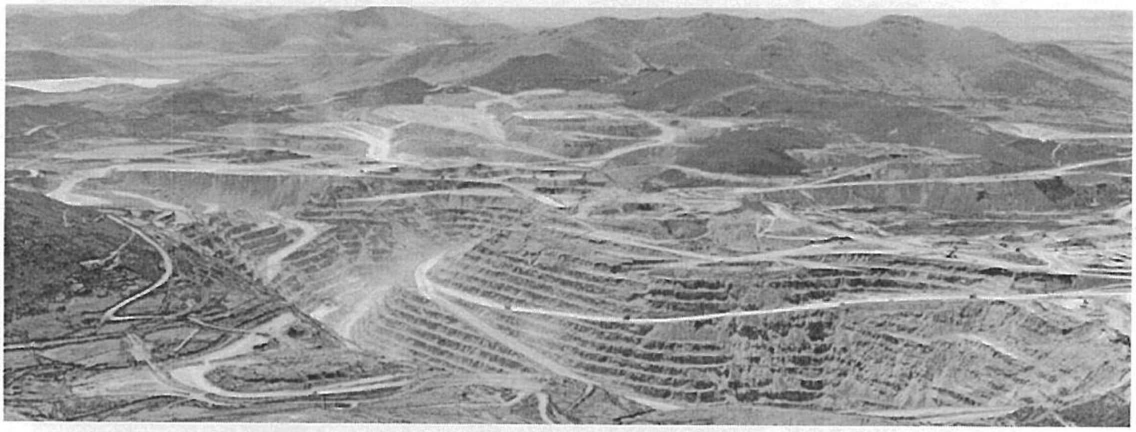
Instructor: Prachya Tepnarong, Ph.D.

SYLLABUS

Topic 1:	Introduction
Topic 2:	Basic Mechanics of Slope Failure
Topic 3:	Structural Geology and Data Interpretation
Topic 4:	Site Investigation and Geological Data Collection
Topic 5:	Rock Strength Properties and their Measurement
Topic 6:	Groundwater Flow and Pressure
Topic 7:	Plane Failure
	MIDTERM EXAM
Topic 8:	Wedge Failure
Topic 9:	Circular Failure
Topic 10:	Toppling Failure
Topic 11:	Numerical Analysis
Topic 12:	Slope Excavation Methods
Topic 13:	Stabilization of Rock Slopes
Topic 14:	Slope Movement Monitoring
	FINAL EXAM

Scoring

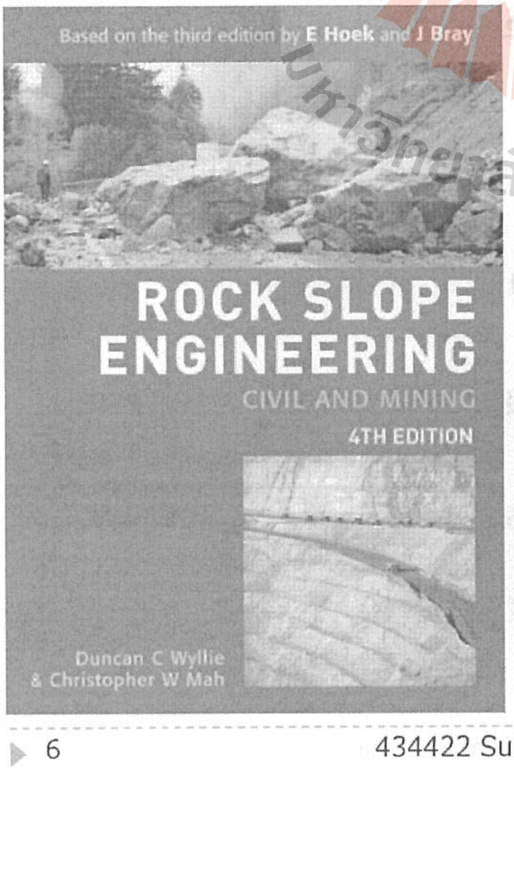
- Homework 20%
- Quiz 10%
- Term Project 20%
- Mid-term Exam 25%
- Final Exam 25%



References:

- Hoek, E. and Bray J.W., 1980, *Rock slope engineering*, 3rd ed., Institute of Mining and Metallurgy, London, 358 p.
- Brady, B.H.G. and Brown E.T., 1985, *Rock mechanics for underground mining*, George Allen and Unwin, London, 527 p.
- Duncan, C.W. and Christopher W.M., 2004, *Rock slope engineering: civil and mining (Base on Rock slope engineering, 3rd ed., 1981, by Dr Evert Hoek and Dr John Bray)*, Spon Press, London, 431p.
- Hartman, H.L. (ed.), 1992, *SME mining engineering handbooks*, Vol. 1 & 2, Society for Mining, Metallurgy and Exploration, Littleton, CO., 2260 p.

Text Book



Economic and Planning Considerations

Factors in Rock Slope Design and Analysis

1. Geologic Conditions (Rock types, structural geology, GW, etc.)

2. Excavation Technique (Soft rock / Hard rock)

3. Shape of Slope (Dip angle/dip direction)

4. Cost

Economic Consequence of Instability

Cost Considerations used to Design Rock Slope Excavations

1. Basic Excavation Cost

(The basic cost unit is taken as the cost per ton mined from the face)

2. Clean-up Cost

(The cost of clearing up a slope failure)

3. Drainage Installation Cost

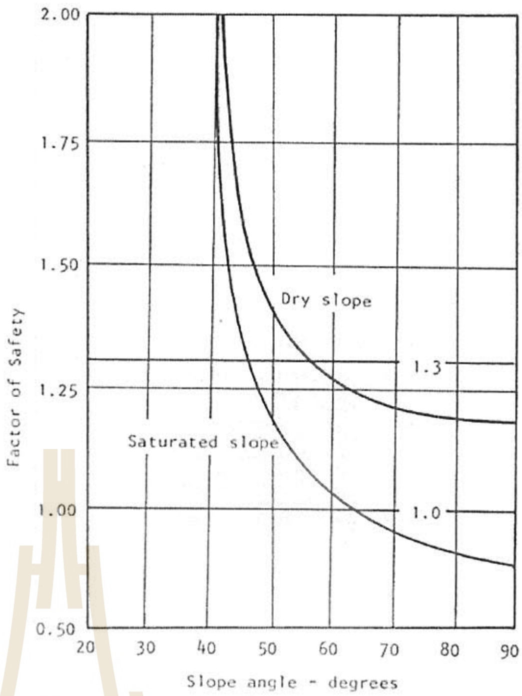
(The design and installation of a drainage system involves a fixed cost)

4. Rock Support Cost

(The cost of rock bolt, tensioned cables, etc., installed by a specialist contractor)

Variation of Factor of Safety with slope angle

Figure 2 : Variation of Factor of Safety with slope angle.



Excavation Tonnages and Cable Loads

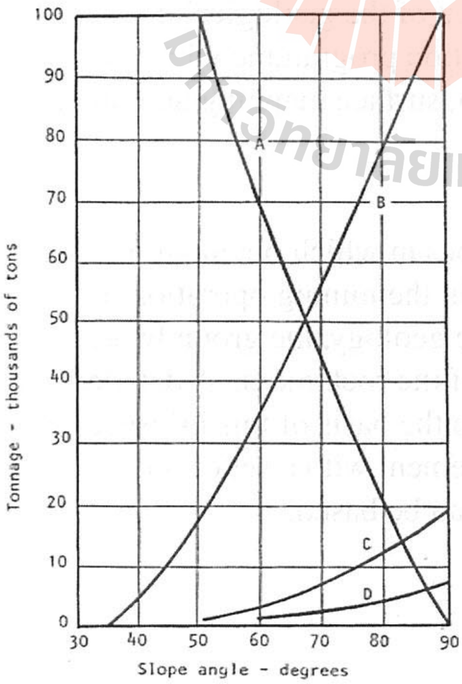


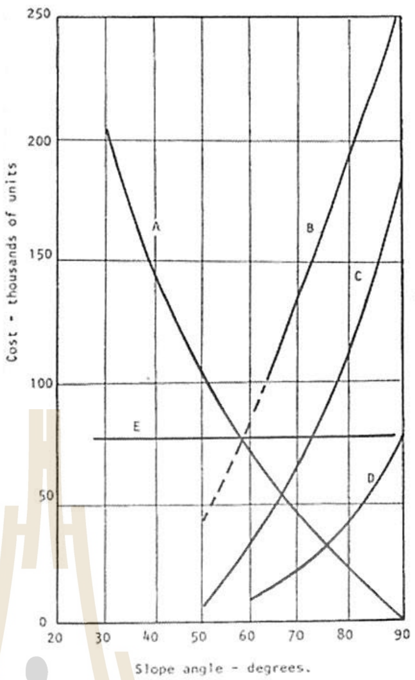
Figure 3 : Excavation tonnages and cable loads.

Line A - *Tonnages excavated in flattening slope 100 ft. high × 300 ft. long.*
Line B - *Tonnage to be cleared up if wedge failure occurs.*
Line C - *Cable load required for a factor of safety of 1.3 for a saturated slope.*
Line D - *Cable load required for a factor of safety of 1.3 for a dry slope.*

Comparative Cost Options

Figure 4 : Comparative cost options.

Line A - Cost per ton mined from face - from line A in Figure 3.
Line B - Cost of clearing up a slope failure.
Line C - Cost of installing cables in a saturated slope.
Line D - Cost of installing cables in a dry slope.
Line E - Cost of draining slope.

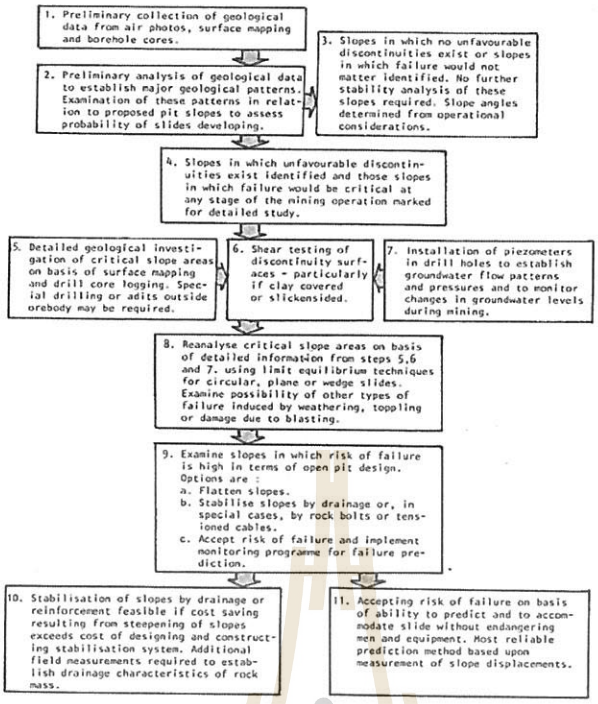


Planning of Slope Stability Investigation

Stage 1 involves a preliminary evaluation of the geological data available from the prospecting or exploration programme which normally includes air photo interpretation, surface mapping and diamond drilling.

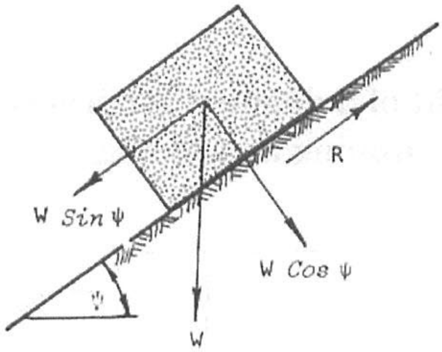
Stage 2 which applies only to those slopes in which potential instability could prove dangerous at some stage in the mining operation, involves a much more detailed study of the geology, the groundwater conditions and the mechanical properties of the rock mass. A detailed analysis of stability is then carried out on the basis of this information and this should provide the mine management with a set of quantative data upon which rational decisions can be based.

Planning a Slope Stability Program



Greek Letter

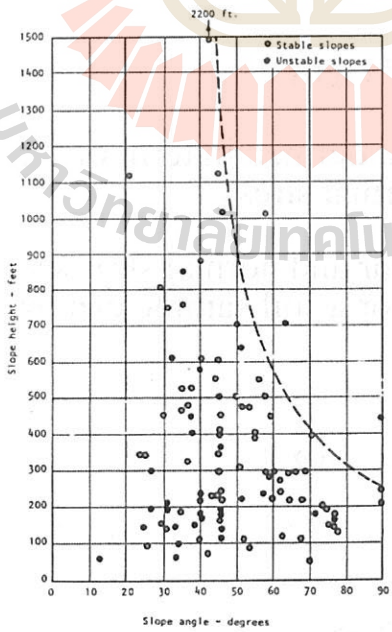
Greek Letter	Name	Equivalent	Sound When Spoken
Α α	Alpha	A	al-fah
Β β	Beta	B	bay-tah
Γ γ	Gamma	G	gam-ah
Δ δ	Delta	D	del-tah
Ε ε	Epsilon	E	ep-si-lon
Ζ ζ	Zeta	Z	zay-tah
Η η	Eta	E	ay-tay
Θ θ	Theta	Th	thay-tah
Ι ι	Iota	I	eye-o-tah
Κ κ	Kappa	K	cap-ah
Λ λ	Lambda	L	lamb-dah
Μ μ	Mu	M	mew
Ν ν	Nu	N	new
Ξ ξ	Xi	X	zzEye
Ο ο	Omicron	O	om-ah-cron
Π π	Pi	P	pie
Ρ ρ	Rho	R	row
Σ σ	Sigma	S	sig-ma
Τ τ	Tau	T	tawh
Υ υ	Upsilon	U	oop-si-lon
Φ ϕ	Phi	Ph	figh or fie
Χ χ	Chi	Ch	kigh
Ψ ψ	Psi	Ps	sigh
Ω ω	Omega	O	o-may-gah



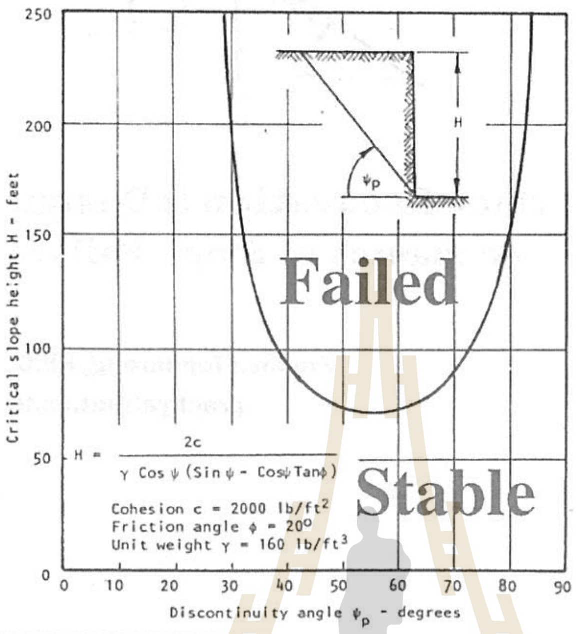
434422 Surface Excavation & Design
Topic 2 Basic Mechanics of Slope Failure

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

Slope Height vs. Slope Angle (Hard Rock Slope)



Critical height of a drained vertical slope containing a planar discontinuity dipping at and angle ψ_p



Friction, Cohesion and Unit Weight

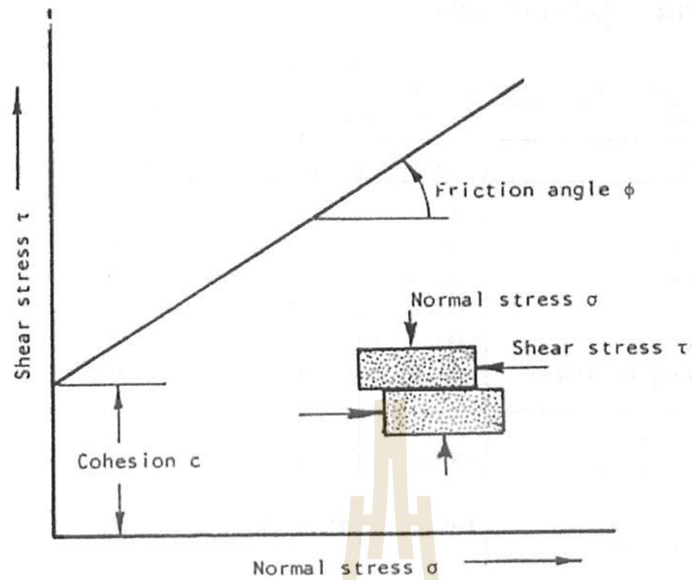
- Friction and cohesion are best defined in term of the plot of shear stress versus normal stress
- The relationship between shear and normal stresses for a typical rock surface or for a soil sample can be expressed as:

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

where
τ = shear stress
σ = normal stress
c = cohesion
ϕ = basic friction angle

} **from direct shear test** (c and ϕ)

Shear stress-normal stress relationship



Typical soil and rock properties

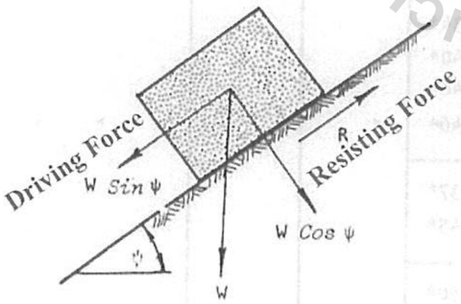
TABLE 1 - TYPICAL SOIL AND ROCK PROPERTIES

Description: Type		Description: Material	Unit weight (Saturated/dry) lb/ft^3	Unit weight (Saturated/dry) kN/m^3	Friction angle degrees	Cohesion lb/ft^2	Cohesion kPa
Cohesionless	Sand	Loose sand , uniform grain size	118/90	19/14	28-34*		
		Dense sand, uniform grain size	130/109	21/17	32-40*		
		Loose sand, mixed grain size	124/99	20/16	34-40*		
		Dense sand, mixed grain size	135/116	21/18	38-46*		
	Gravel	Gravel, uniform grain size	140/130	22/20	34-37*		
		Sand and gravel, mixed grain size	120/110	19/17	48-45*		
	Blasted/broken rock	Basalt	140/110	22/17	40-50*		
		Chalk	80/62	13/10	30-40*		
		Granite	125/110	20/17	45-50*		
		Limestone	120/100	19/16	35-40*		
		Sandstone	110/80	17/13	35-45*		
		Shale	125/100	20/16	30-35*		

Typical soil and rock properties

TABLE 1 - TYPICAL SOIL AND ROCK PROPERTIES							
Description			Unit weight (Saturated/dry)		Friction angle	Cohesion	
Type		Material	lb/ft^3	kN/m^3	degrees	lb/ft^2	kPa
Cohesive	Clay	Soft bentonite	80/30	13/6	7-13	200-400	10-20
		Very soft organic clay	90/40	14/6	12-16	200-600	10-30
		Soft, slightly organic clay	100/60	16/10	22-27	400-1000	20-50
		Soft glacial clay	110/76	17/12	27-32	600-1500	30-70
		Stiff glacial clay	130/105	20/17	30-32	1500-3000	70-150
		Glacial till, mixed grain size	145/130	23/20	32-35	3000-5000	150-250
	Rock	Hard igneous rocks - granite, basalt, porphyry	** 160 to 190	25 to 30	35-45	720000-1150000	35000-55000
		Metamorphic rocks - quartzite, gneiss, slate	160 to 180	25 to 28	30-40	400000-800000	20000-40000
		Hard sedimentary rocks - limestone, dolomite, sandstone	150 to 180	23 to 28	35-45	200000-600000	10000-30000
		Soft sedimentary rock - sandstone, coal, chalk, shale	110 to 150	17 to 23	25-35	20000 - 400000	1000-20000

Sliding due to gravitational Loading



- Coulomb Criterion:

$$\tau = c + \sigma \tan \phi \qquad (1)$$

- The normal stress σ which acts across the potential sliding surface is given by

$$\sigma = (W \cos \psi) / A \qquad (2)$$

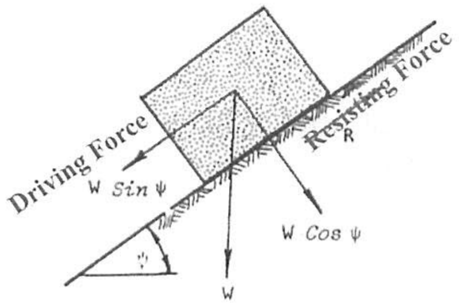
where A is the base area of the block

- Sub (2) into (1); and Shear Force, $R = \tau A$

$$\tau = c + \frac{W \cos \psi}{A} . \tan \phi$$

$$R = cA + W \cos \psi . \tan \phi \qquad (3)$$

Sliding due to gravitational Loading



- Condition of Limiting Equilibrium

Driving Force = Resisting Force

$$W \sin\psi = cA + W\cos\psi \,.\, \tan\phi \quad (4)$$

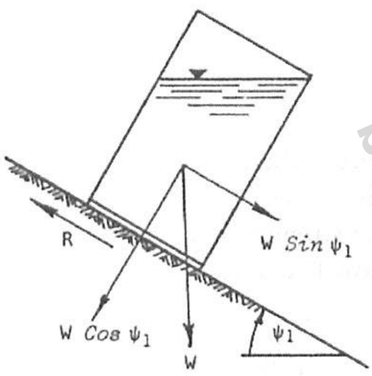
- If the cohesion c = 0, the condition of limiting equilibrium defined by equation (4) simplifies to

$$\cancel{W}\sin\psi = \cancel{cA}^{\,0} + \cancel{W}\cos\psi \,.\, \tan\phi$$

$$\sin\psi = \cos\psi \,.\, \tan\phi$$

$$\psi = \phi \quad (5)$$

Influence of Water Pressure on Shear Strength



- The influence of water pressure upon the shear strength of two surfaces in contact can most effectively be demonstrated by the beer can experiment.
- An opened beer can filled with water rests on an inclined piece of wood as shown in sketch.
- For simplicity the cohesion between the beer can base and the rood is assumed to be zero. According to equation (5) the can with its contents of water will slide down the plank when $\psi_1 = \phi$.
- The base of the can is now punctured sot that water can enter the gap between the base and the plank, giving rise to a water pressure u or to an uplift force

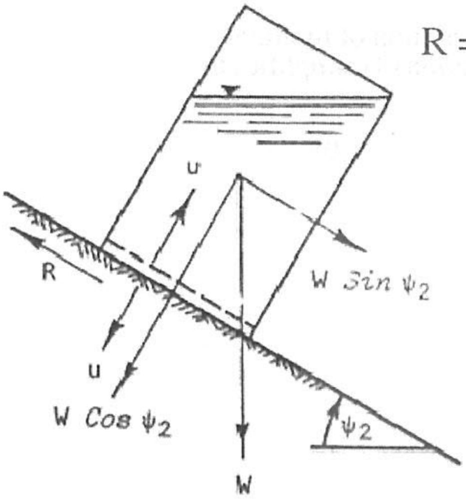
$$U = uA$$

where A is the base area of the can.

Influence of Water Pressure on Shear Strength

- The normal force W cos ψ_2 is now reduced by this uplift force U and the resistance to sliding is now

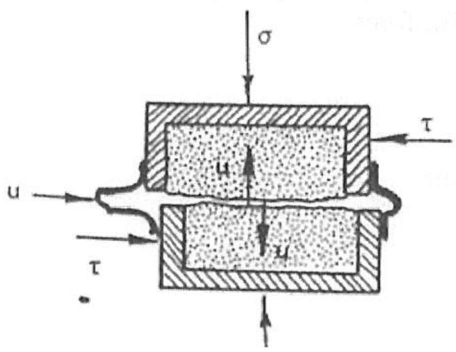
$$R = (W \cos \psi_2 - U) \tan \phi \qquad (6)$$



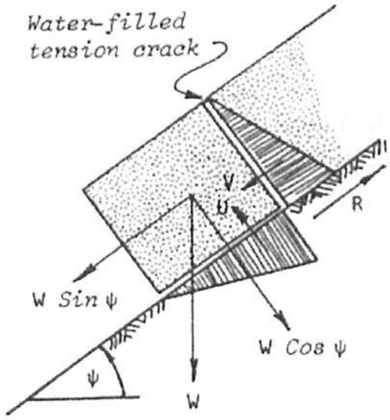
Effective Stress Law

- The normal stress σ acting across the failure surface is reduced to the effective stress (σ - u) by the water pressure u. The relationship between shear strength and normal strength defined by equation (1) now becomes

$$\tau = c + (\sigma - u) \tan \phi \qquad (10)$$



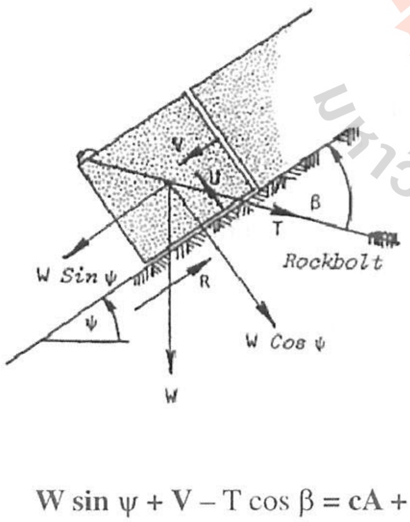
The Effect of Water Pressure in a tension Crack



- The condition of limiting equilibrium for this case of a block acted upon by water forces V and U in addition to its own weight W is defined by
- From this equation it will be seen that the disturbing force tending to induce sliding down the plane is increased and the frictional force resisting sliding is decreased and hence, both V and U result in decreases in stability.

$$W \sin \psi + V = cA + (W \cos \psi - U) \tan \phi \qquad (11)$$

Reinforcement to Prevent Sliding



- Consider the block resting on the inclined plane and acted upon by the uplift force U and the force V due to water pressure in the tension crack.
- A rockbolt, tensioned to a load T is installed at an angle β to the plane. The resolved component of the bolt tension T acting parallel to the plane is T Cos β while the component acting across the surface upon which the block rests is T sin β. The condition of limiting equilibrium for the case is defined by

$$W \sin \psi + V - T \cos \beta = cA + (W \cos \psi - U + T \sin \beta) \tan \phi \qquad (12)$$

Factor of Safety of Slope

- In order to compare the stability of slopes under conditions other than those of limiting equilibrium, some form of index is required and the most commonly used index is the *factor of Safety (F.S or F)*

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

- Considering the case of the block acted upon by water forces and stabilised by a tensioned rockbolt the factor of safety is given by

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

Factor of Safety of Slope

- The bolt tension required to provide a specified factor of safety of F is a minimum when the angle β satisfies the equation

$$\tan\beta = \tan\phi / F.S. \qquad (14)$$

- This result is obtained by differentiating equation (13) with respect to β, and setting

$$\frac{dT}{d\beta} = 0 \text{ and } \frac{dF}{d\beta} = 0.$$

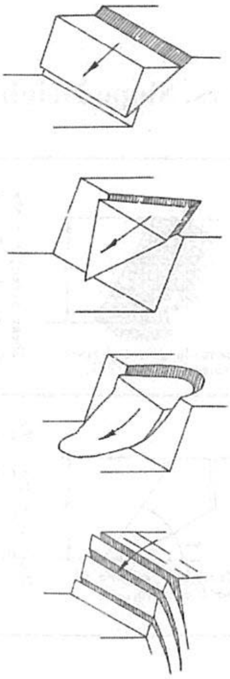
Minimum F.S.

Mining Slope (Shot Life Slope)	F.S. = 1.1-1.3
Civil (Long Term Slope)	F.S. = 1.5
Natural Slope	F.S. = 1.1-1.3

Type of Slope Failure

Failure Modes:

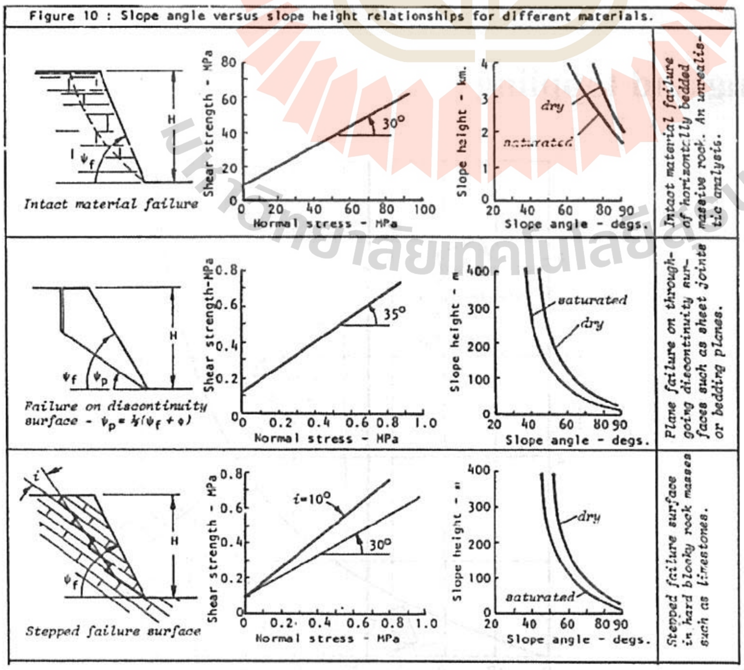
1. Plane Failure
2. Wedge Failure
3. Circular Failure
4. Toppling Failure
5. Ravelling Slope (Weathering, Freeze & Thawing)



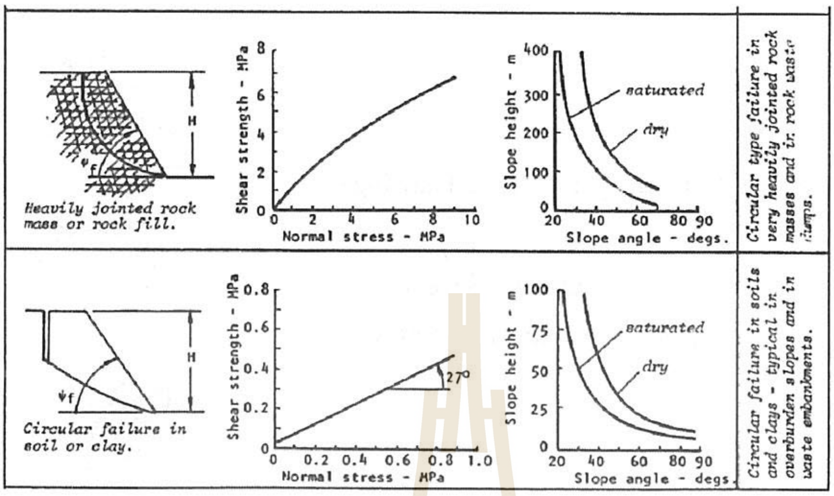
Modes of 1-3 can be Calculated Factor of Safety

Modes of 4-5 cannot be Calculated Factor of Safety

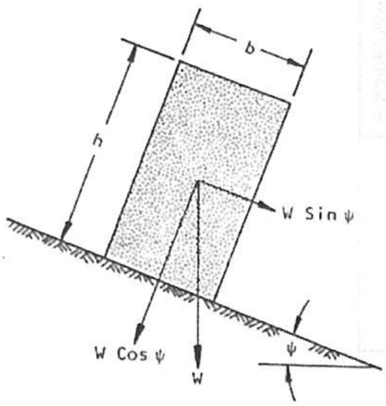
Slope Angle vs. Slope Height Relationships for Different Material



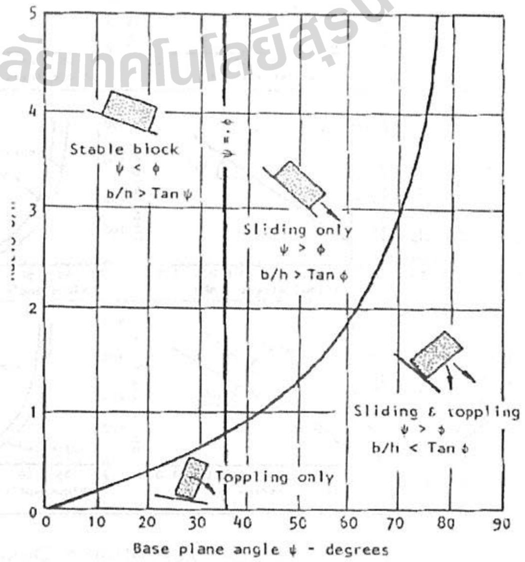
Slope Angle vs. Slope Height Relationships for Different Material



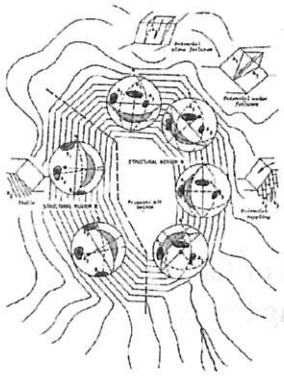
Conditions for sliding and toppling

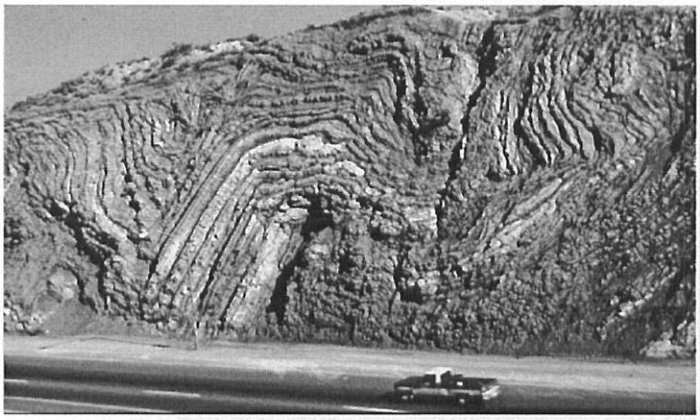


Geometry of block on inclined plane.



Conditions for sliding and toppling of a block on an inclined plane.





434422 Surface Excavation & Design

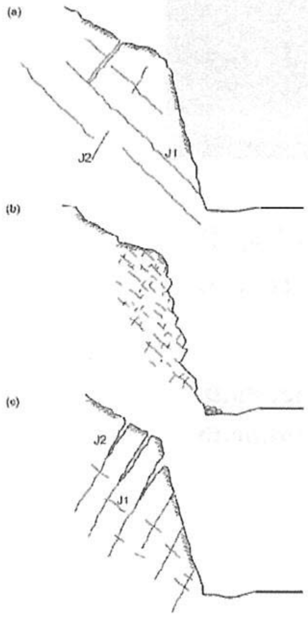
Topic 3 Structural Geology and Data Interpretation

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

Definition of Geological Terms

Rock Material = Intact Rock
Rock Mass = In-situ Rock
Waste Rock = Broken Rock (Angular)
Sand & Gravel (Rounded)
Discontinuities = Weak Plan (fault, joint, bedding, cleavage, crack, dykes)
Major Discontinuities = Domination of a particular outcrop
Discontinuities Set = Systems of discontinuities (approximately same inclination and orientation)
Continuity = Persistence
Gouge = Infilling (Material between two faces of a structural discontinuity)
Roughness = Surface roughness on discontinuities in rock

Effective of Discontinuities on Slope Stability

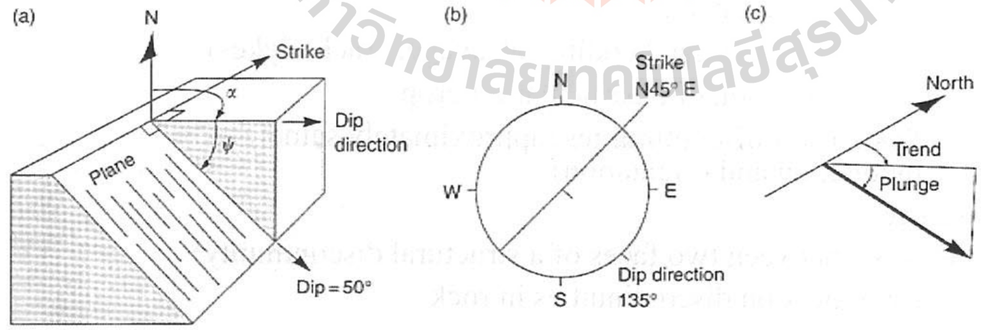


(a) Persistence J1 joint dipping out of face forms potentially unstable sliding block;

(b) Closely spaced, low persistence joints cause reveling of small block;

(c) Persistence J2 joints dipping into face form potential toppling slabs.

Orientation of Discontinuities



Definition of Geometrical Terms

Dip = maximum inclination of a discontinuity to horizontal (angle ψ)

Dip Direction = direction of horizontal trace of line dip, measured clockwise from north (angle α)

Strike = trace of intersection of an obliquely inclined plane with horizontal reference plane and dip direction of oblique plane

Plunge = dip of line, such as line of intersection of two plane or axis of borehole or tunnel

Trend = direction of horizontal projection of a line, measured clockwise from north

Stereographic Analysis of Structural Geology

- -Stereographic projection
- -Pole Plots and Contour Plots
- -Pole Density
- -Great Circles
- -Line of Intersection

Stereographic Projection

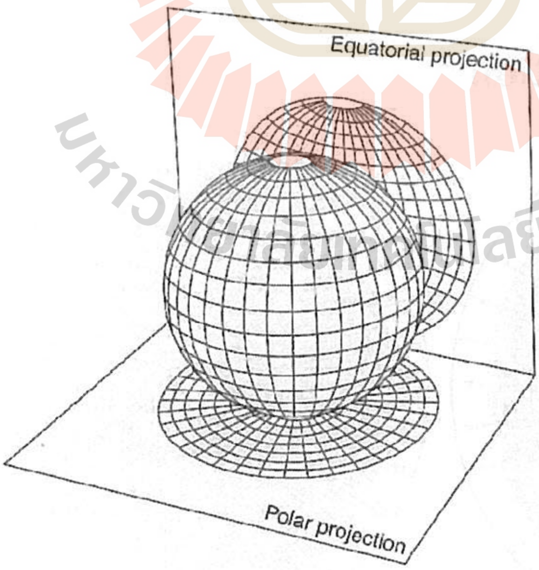


Figure 2.7 Polar and equatorial projections of a sphere.

Stereographic Projection

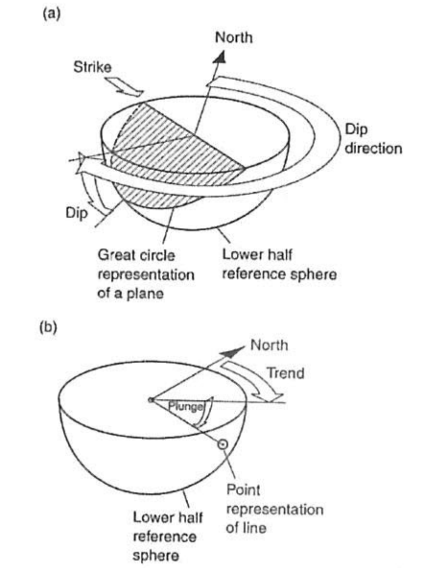


Figure 2.5 Stereographic representation of plane and line on lower hemisphere of reference sphere: (a) plane projected as great circle; (b) isometric view of line (plunge and trend).

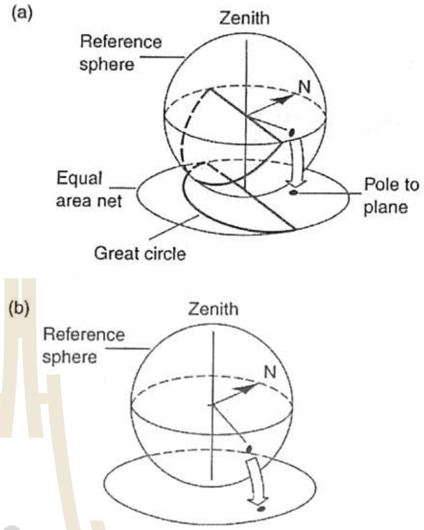
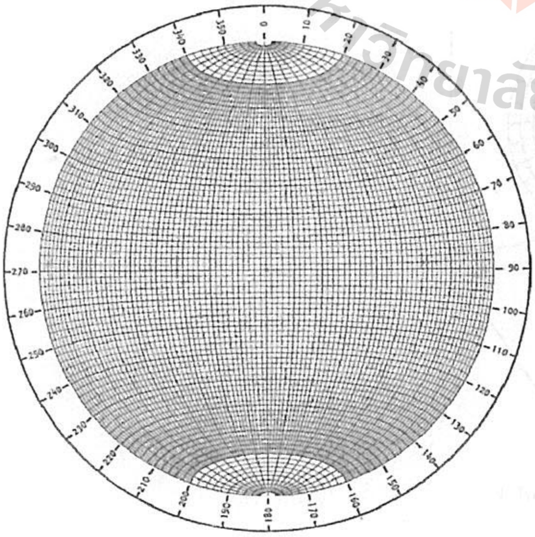
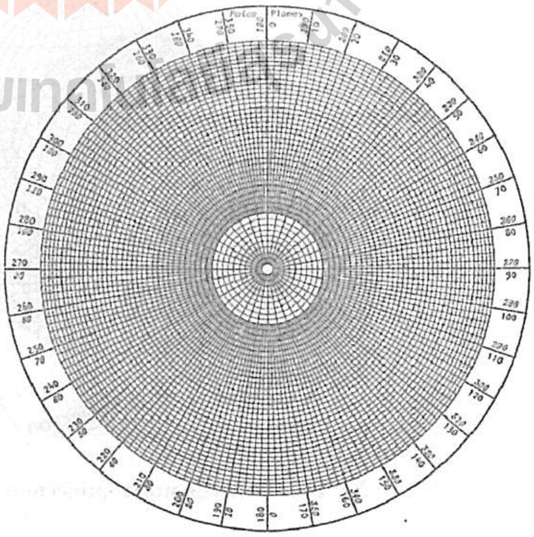


Figure 2.6 Equal area projections of plane and line: (a) plane projected as great circle and corresponding pole; (b) line projected as pole.

Stereographic Projection



Equatorial Equal-Area Stereonet



Polar Equal-Area Stereonet

Stereographic Projection

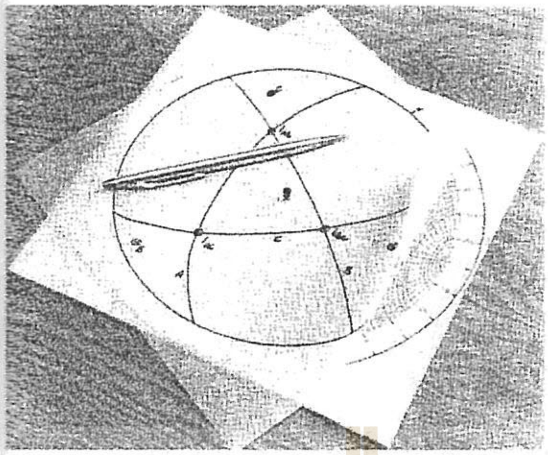


Figure 2.8 Geological data plotted and analyzed on a piece of tracing paper that is located over the center of the stereonet with a pin to allow the paper to be rotated.

Pole Plots and Contour Plots

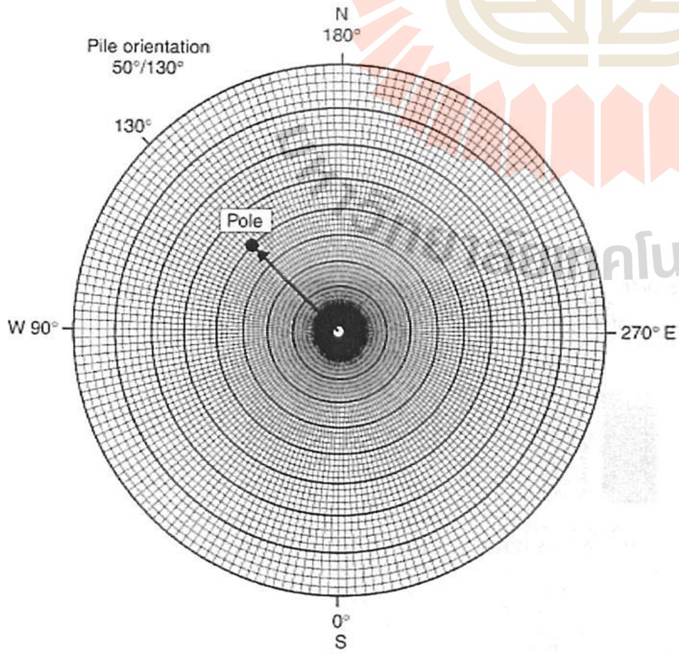


Figure 2.9 Plotting poles on a polar net. Plot pole of plane oriented at 50/130—locate dip direction of 130° clockwise around the circumference of a circle starting at the lower end of the vertical axis. At 130° radial line, count 50° out from the center of the net, and plot a point at the intersection between 130° radial line and 50° circle.

Pole Plots and Contour Plots

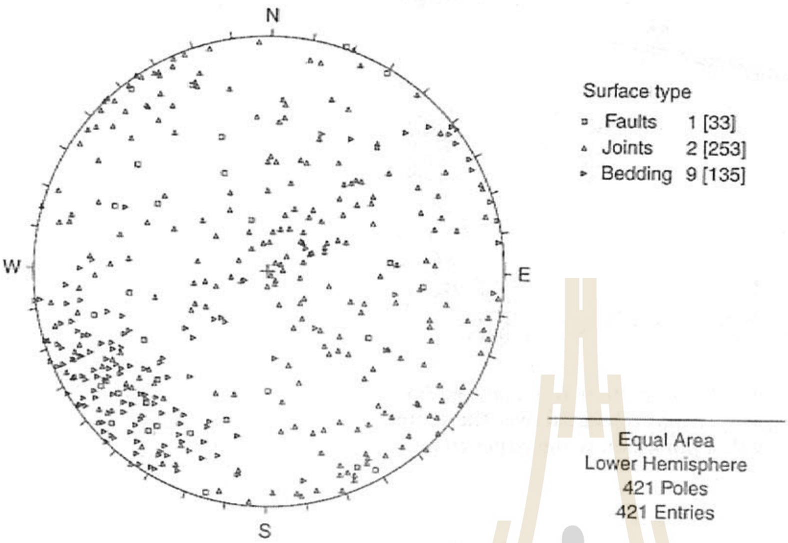


Figure 2.10 Example of pole plot of 421 planes comprising bedding, joints and faults.

Pole Density

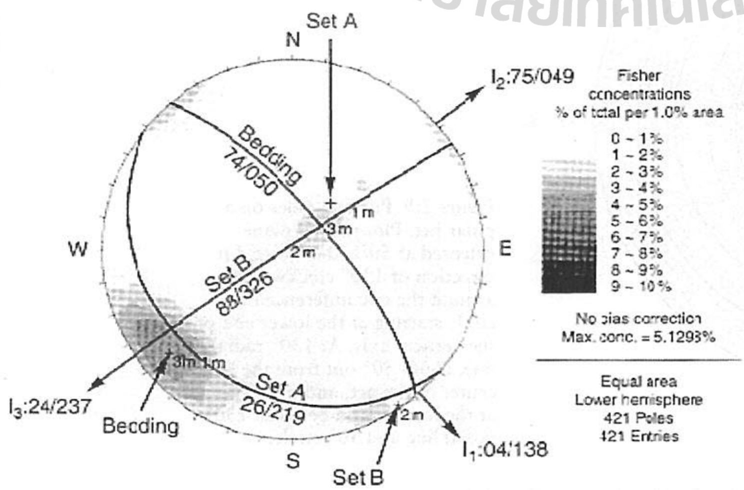
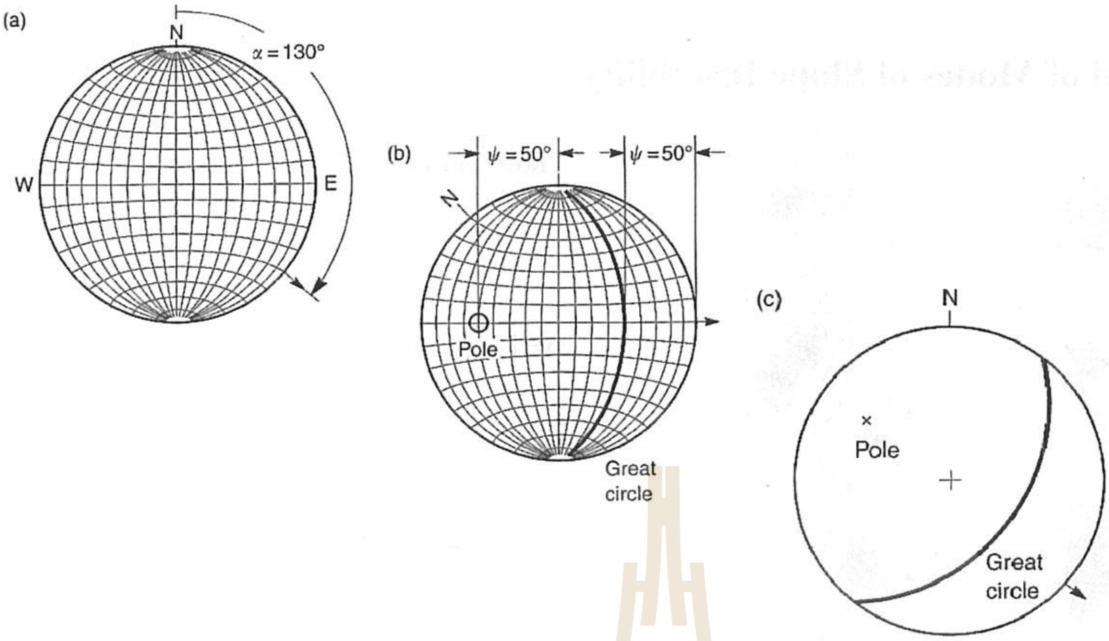
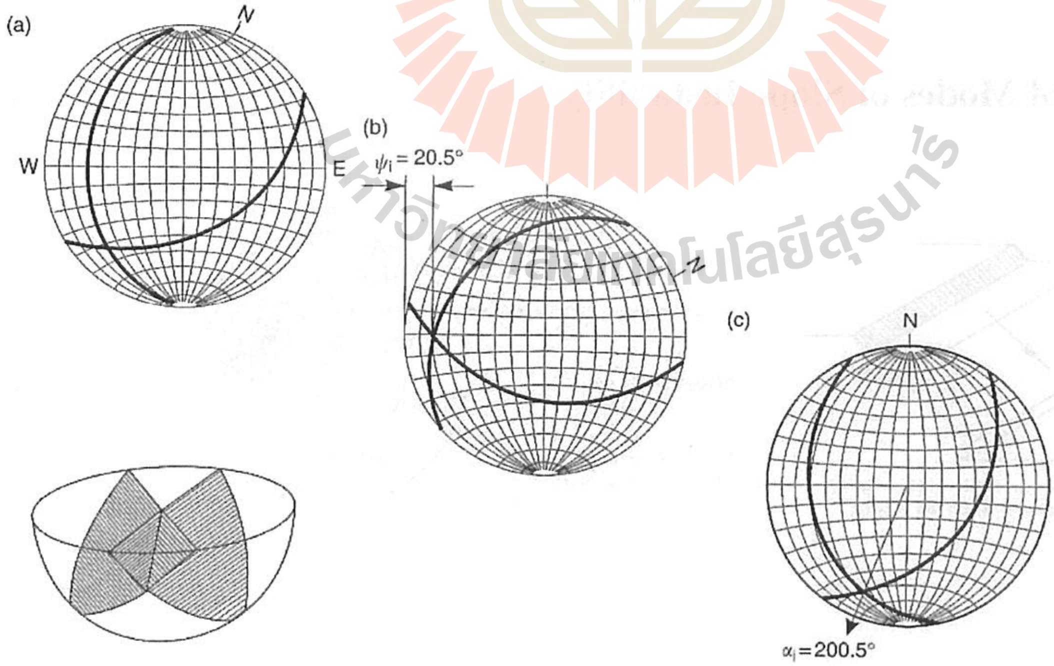


Figure 2.11 Contoured plot of data shown in Figure 2.10, with great circles corresponding to mean orientation of bedding and two orthogonal joint sets, and lines of intersection between planes.

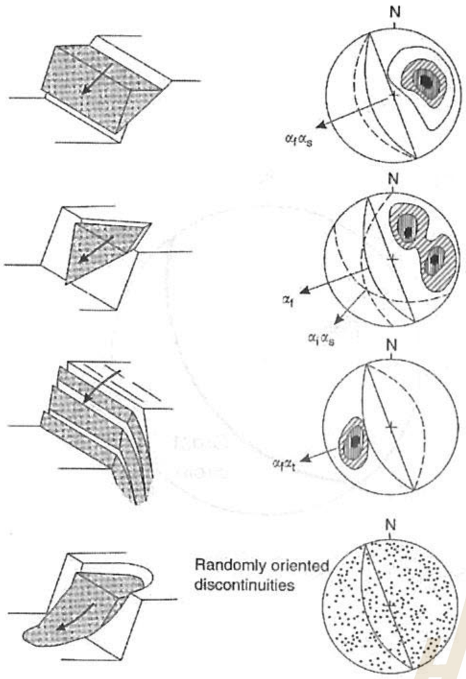
Great Circles



Line of Intersection



Identified of Modes of Slope Instability



Plane Failure

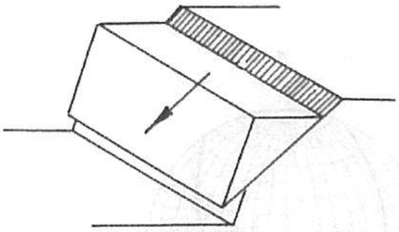
Wedge Failure

Toppling Failure

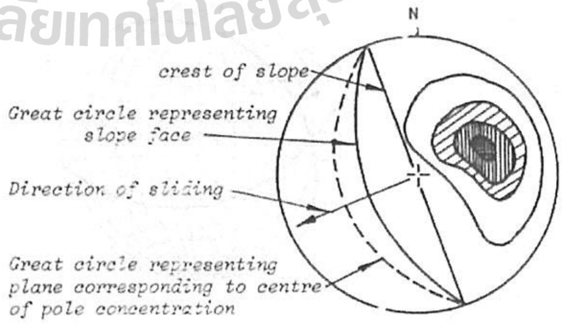
Circular Failure

Identified of Modes of Slope Instability

Plane Failure

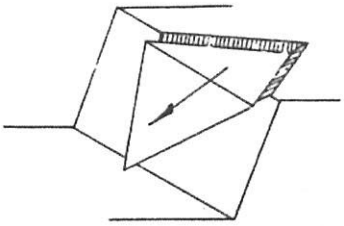


Plane failure in rock with highly ordered structure such as slate.

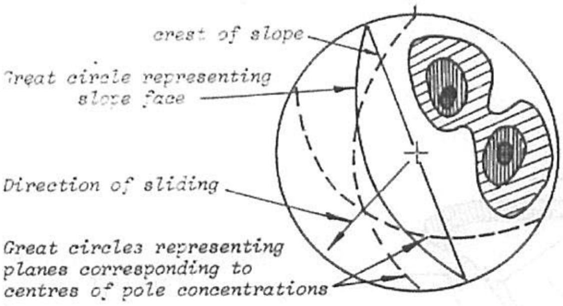


Identified of Modes of Slope Instability

Wedge Failure

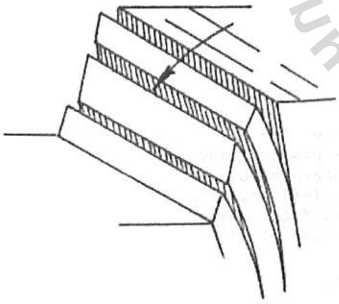


Wedge failure on two intersecting discontinuities.

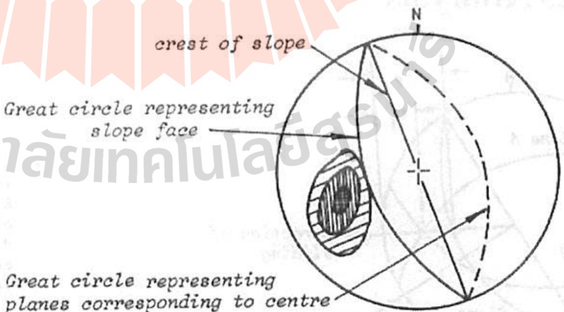


Identified of Modes of Slope Instability

Toppling Failure

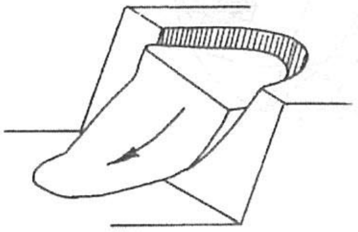


Toppling failure in hard rock which can form columnar structure separated by steeply dipping discontinuities.

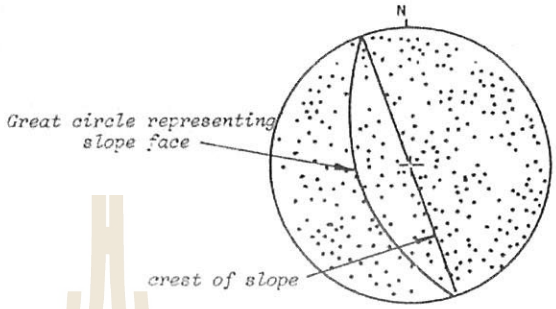


Identified of Modes of Slope Instability

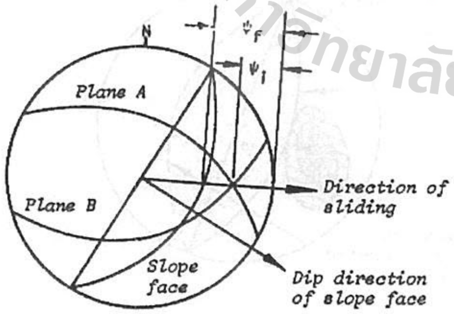
Circular Failure



Circular failure in overburden soil, waste rock or heavily fractured rock with no identifiable structural pattern.

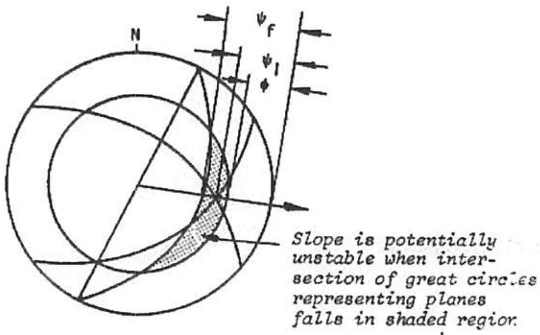


Kinematics Analysis



a: Sliding along the line of intersection of planes A and B is possible when the plunge of this line is less than the dip of the slope face, measured in the direction of sliding, ie

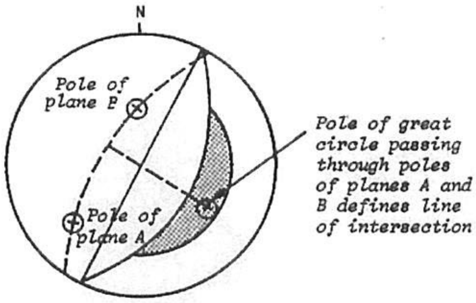
$$\psi_f > \psi_i$$



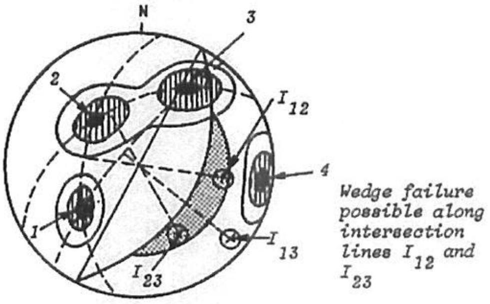
b : Sliding is assumed to occur when the plunge of the line of intersection exceeds the angle of friction, ie

$$\psi_f > \psi_i > \phi$$

Kinematics Analysis



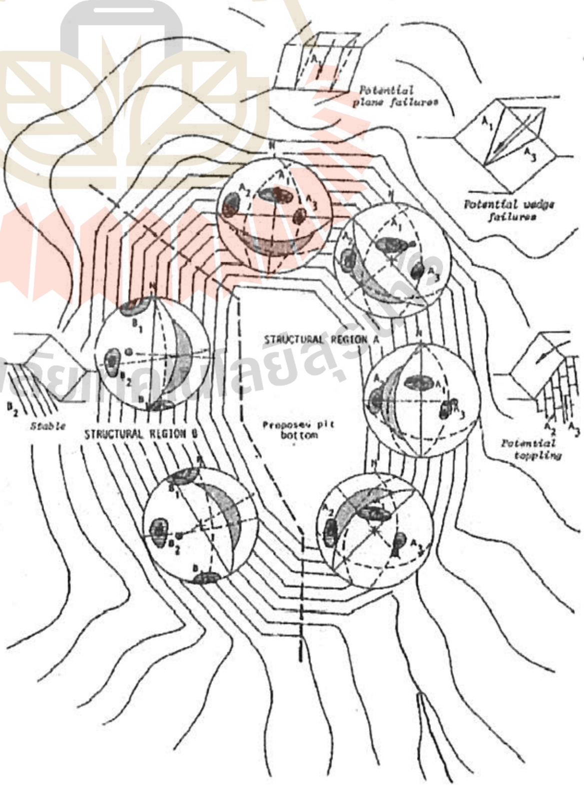
c : Representation of planes by their poles and determination of the line of intersection of the planes by the pole of the great circle which passes through their poles.

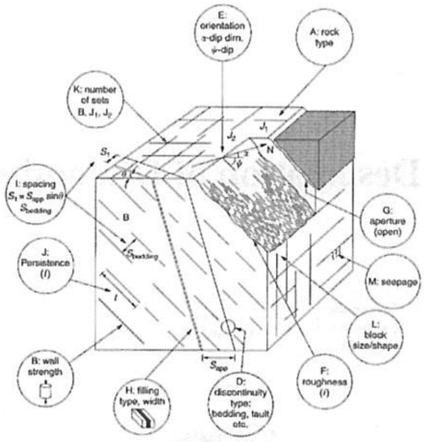


d : Preliminary evaluation of the stability of a 50° slope in a rock mass with 4 sets of structural discontinuities.

Kinematics Analysis

- Presentation of structural geology on stereonets, and preliminary evaluation of slope stability of proposed open pit mine.





434422 Surface Excavation & Design

Topic 4 Site Investigation and Geological Data Collection

Prachya Tepnarong, Ph.D.

prachya@sut.ac.th

Investigation and Collection Processes

1. Regional Geology Investigations

-Air Photograph

-Contour Map

-Geologic Map

2. Surface Mapping (detailed mapping)

-Rock Type

-Structure (discontinuity)

-Groundwater

3. Core Logging

-Confirm Rock Types

-Confirm Structure

-GW Level, Water Table, Permeability

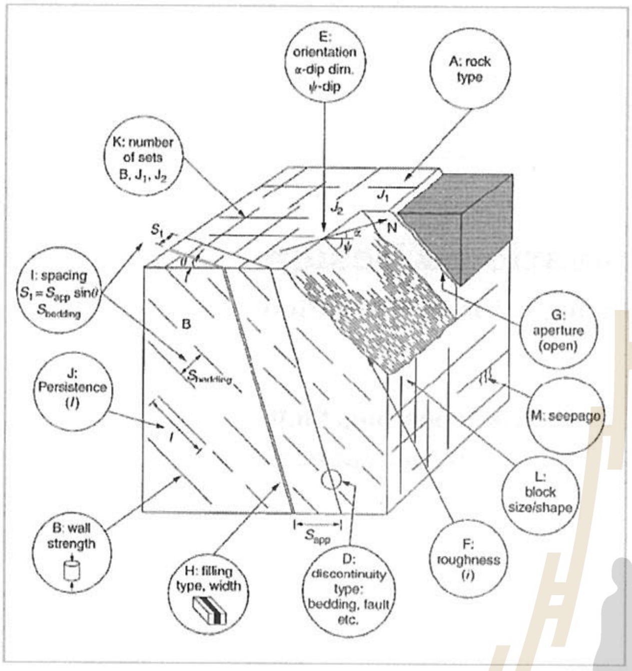
-Discontinuity (RQD)

4. Laboratory Testing

-Joint Shear Strength Test

-Uniaxial Compression Test, Point Load Index Test

Quantitative Description of Discontinuities in Rock Masses (ISRM)

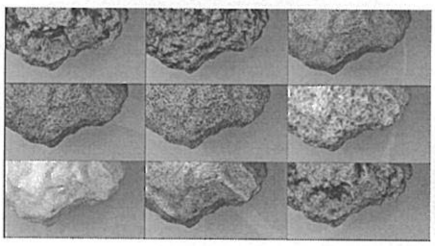


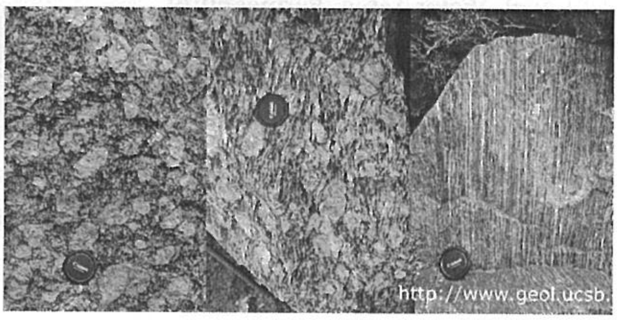
- A - Rock type
- B - Rock strength
- C - Weathering
- D - Discontinuity description
- E - Discontinuity orientation
- F - Roughness
- G - Aperture
- H - Infilling type and width
- I - Spacing
- J - Persistence
- K - Number of sets
- L - Block size and shape
- M - Seepage

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

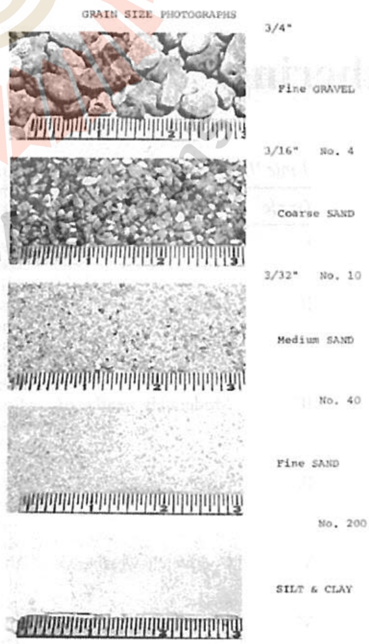
Genetic Group			Detrital Sedimentary			Pyroclastic	Chemical Organic	Metamorphic		Igneous			
Usual Structure			BEDDED			BEDDED		FOLIATED	MASSIVE	MASSIVE			
COMPOSITION										Light coloured minerals are quartz, feldspar, mica and feldspar-like minerals			Dark minerals
	Grain size (mm)		Grains of rock, 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 / Coarse grained	60 / 2	RUDACEOUS	Grains are of rock fragments Rounded grains: CONGLOMERATE / Angular grains: BRECCIA	LIMESTONE (undifferentiated)	CALCIRUDITE	Rounded grains AGGLOMERATE Angular gains VOLCANIC BRECCIA	SALINE ROCKS Halite Anhydrite Gypsum	MIGMATITE / GNEISS Alternate layers of granular and flakey minerals	HORNFELS MARBLE GRANULITE	PEGMATITE			PYROXENITE and PERIDOTITE
										GRANITE	DIORITE	GABBRO	
Medium grained	0.06	ARENACEOUS	SANDSTONE: Grains are mainly mineral fragments QUARTZ SANDSTONE: 95% quarts, voids empty or cemented ARKOSE: 75% quartz, up to 25% feldspar; voids empty of cemented ARGILLARCEOUS SANDSTONE: 75% quartz, 15% • fine detrital material		CALCARENITE	TUFF	CHERT FLINT	SCHIST PHYLLITE SLATE MYLONITE	QUARTZITE AMPHIBOLITE	MICRO-GRANITE	MICRO-DIORITE	DOLERITE	SERPENTINE
Fine grained	0.002	AREGILLACEOUS 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										OBSIDEIAN 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

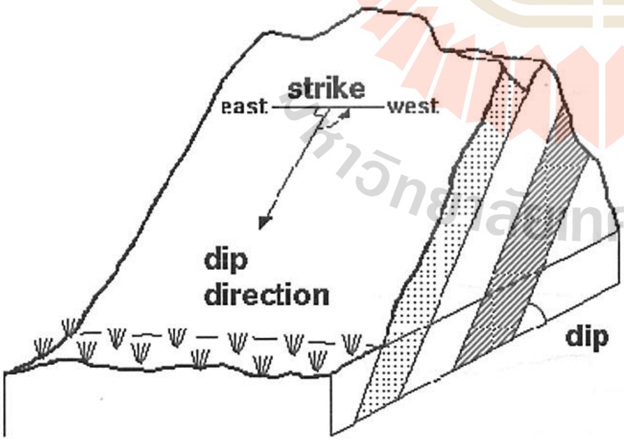
Foliation – parallel orientation of platy minerals, or mineral banding in metamorphic rocks

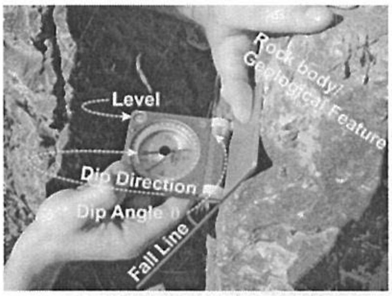
Joint – discontinuity in which there has been no observable relative moment

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

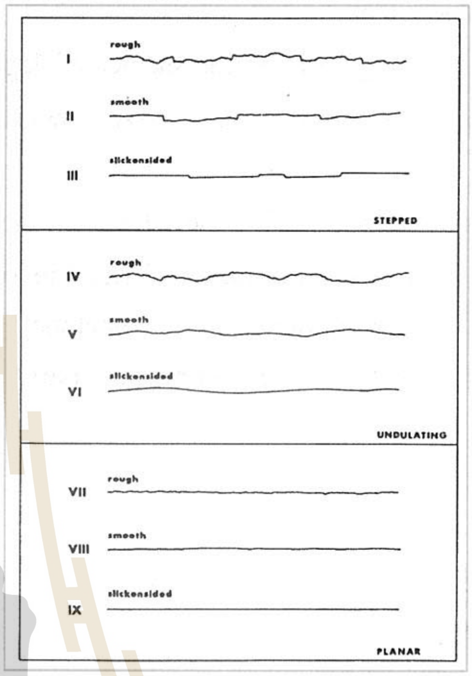




F-Roughness

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

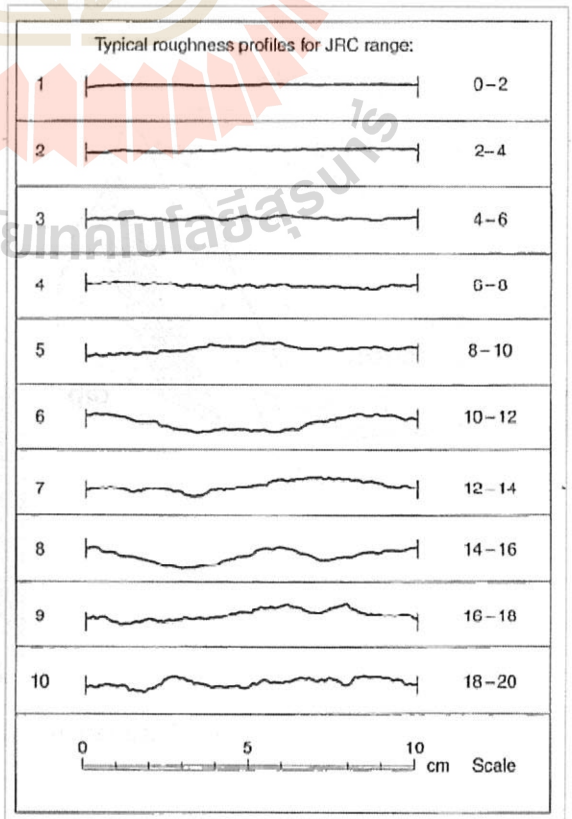
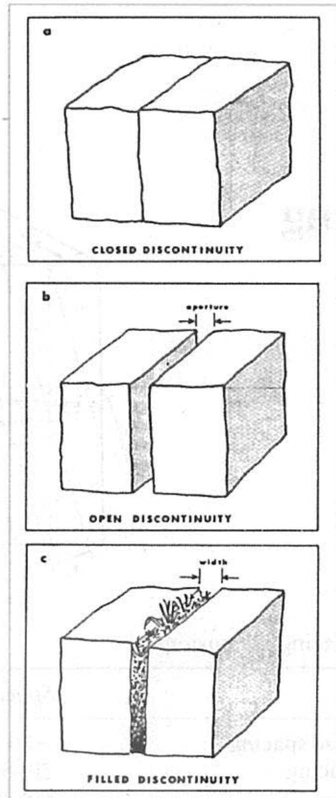


Figure II.3 Roughness profiles and corresponding range of JRC (joint roughness coefficient) values (ISRM, 1981a).

G-Aperture

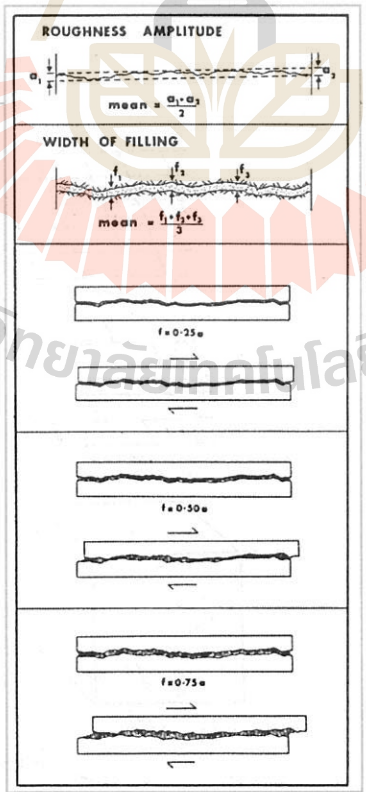
Table II.6 Aperture dimensions

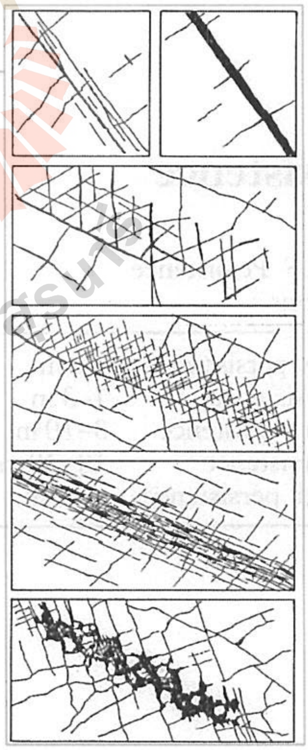
Aperture	*Description*	
<0.1 mm	Very tight	
0.1–0.25 mm	Tight	"Closed" features
0.25–0.5 mm	Partly open	
0.5–2.5 mm	Open	
2.5–10 mm	Moderately wide	"Gapped" features
>10 mm	Wide	
1–10 cm	Very wide	
10–100 cm	Extremely wide	"Open" features
>1 m	Cavernous	



H-Infilling type and width

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





I-Spacing

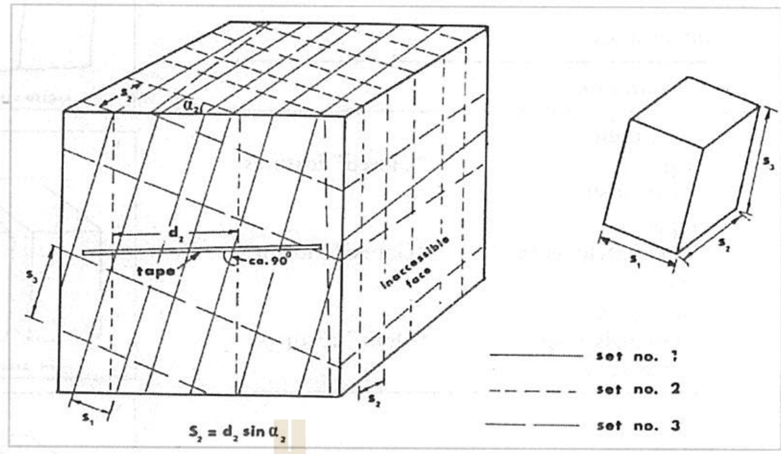


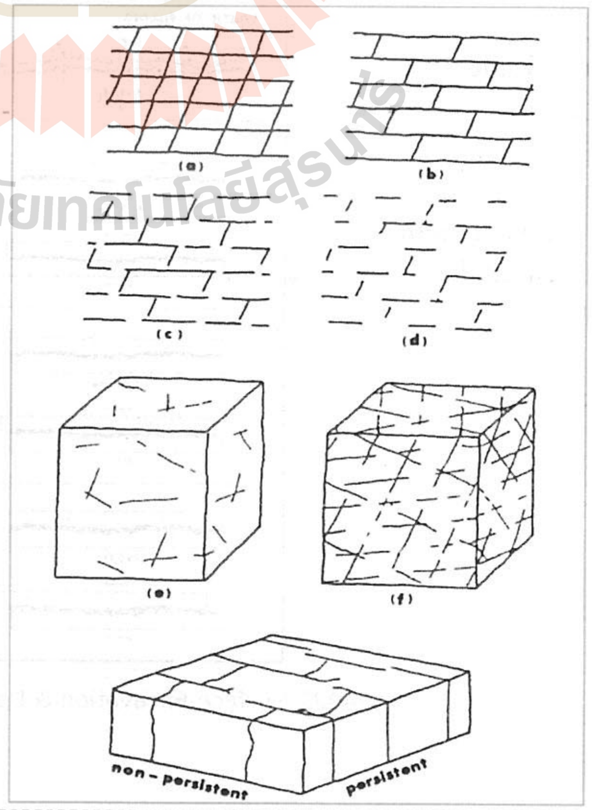
Table II.7 Spacing dimensions

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

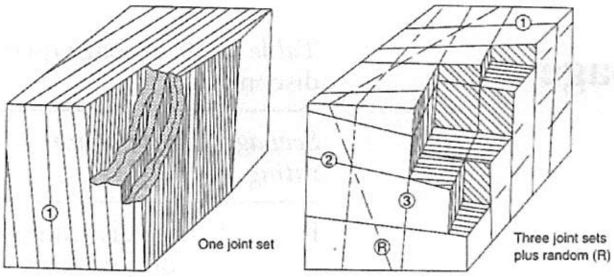


Figure II.4 Examples illustrating the effect of the number of joint sets on the mechanical behavior and appearance of rock masses (ISRM, 1981a).

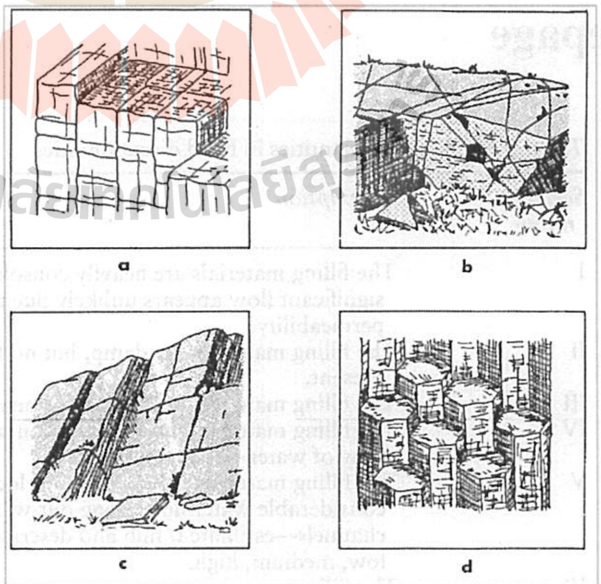
I	massive, occasional random joints
II	one joint set
III	one joint set plus random
IV	two joint sets
V	two joint sets plus random
VI	three joint sets
VII	three joint sets plus random
VIII	four or more joint sets
IX	crushed rock, earth-like

L-Block size and shape

Table II.9 Block dimensions

Description	J_V *(joints/m³)*
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

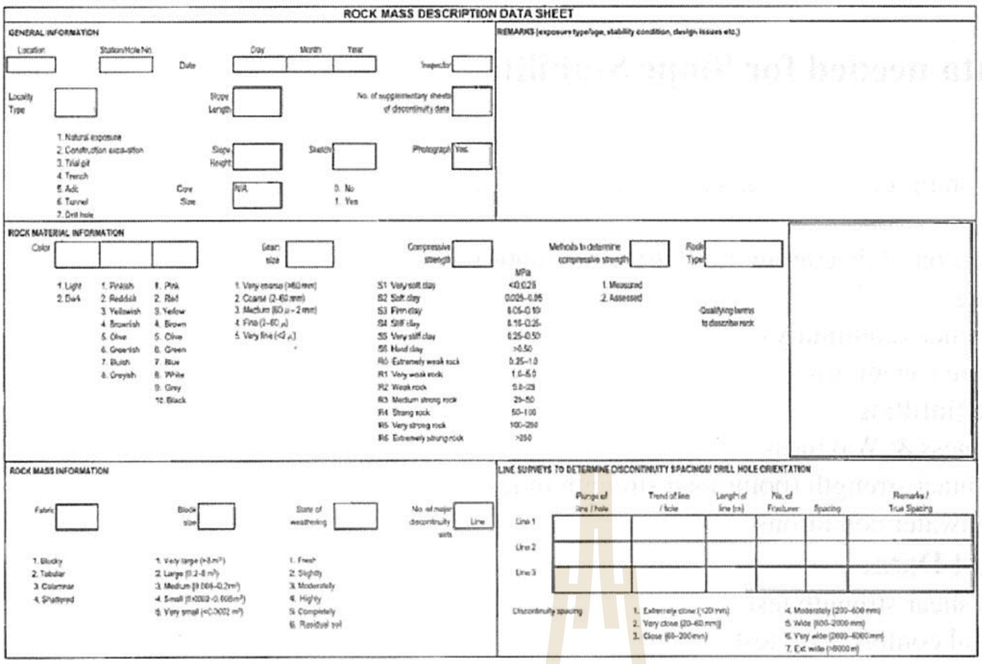
Seepage rating	*Description*
I	The discontinuity is very tight and dry, water flow along it does not appear possible.
II	The discontinuity is dry with no evidence of water flow.
III	The discontinuity flow is dry but shows evidence of water flow, that is, rust staining.
IV	The discontinuity is damp but 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 Description Data Sheet



ROCK MASS DESCRIPTION DATA SHEET

GENERAL INFORMATION
Location | Station/Hole No. | Date: Day, Month, Year | Inspector
Locality Type | Slope Length | No. of supplementary sheets of discontinuity data
1. Natural exposure
2. Construction excavation
3. Trial pit
4. Trench
5. Adit
6. Tunnel
7. Drill hole
Slope Height | Sketch | Photograph: Yes
Core Size: N/A
0. No
1. Yes

REMARKS (exposure type/age, stability condition, design issues etc.)

ROCK MATERIAL INFORMATION
Color | Grain size | Compressive strength | Methods to determine compressive strength | Rock Type

Color:
1. Light, 2. Dark
1. Pinkish, 2. Reddish, 3. Yellowish, 4. Brownish, 5. Olive, 6. Greenish, 7. Bluish, 8. Greyish
1. Pink, 2. Red, 3. Yellow, 4. Brown, 5. Olive, 6. Green, 7. Blue, 8. White, 9. Grey, 10. Black

Grain size:
1. Very coarse (>60 mm)
2. Coarse (2–60 mm)
3. Medium (60 μ – 2 mm)
4. Fine (2–60 μ)
5. Very fine (<2 μ)

Compressive strength	MPa
S1 Very soft clay	<0.025
S2 Soft clay	0.025–0.05
S3 Firm clay	0.05–0.10
S4 Stiff clay	0.10–0.25
S5 Very stiff clay	0.25–0.50
S6 Hard clay	>0.50
R0 Extremely weak rock	0.25–1.0
R1 Very weak rock	1.0–5.0
R2 Weak rock	5.0–25
R3 Medium strong rock	25–50
R4 Strong rock	50–100
R5 Very strong rock	100–250
R6 Extremely strong rock	>250

Methods to determine compressive strength:
1. Measured
2. Assessed

Qualifying terms to describe rock

ROCK MASS INFORMATION
Fabric | Block size | State of weathering | No. of major discontinuity sets: Line

Fabric:
1. Blocky
2. Tabular
3. Columnar
4. Shattered

Block size:
1. Very large (>8 m³)
2. Large (0.2–8 m³)
3. Medium (0.008–0.2 m³)
4. Small (0.0002–0.008 m³)
5. Very small (<0.0002 m³)

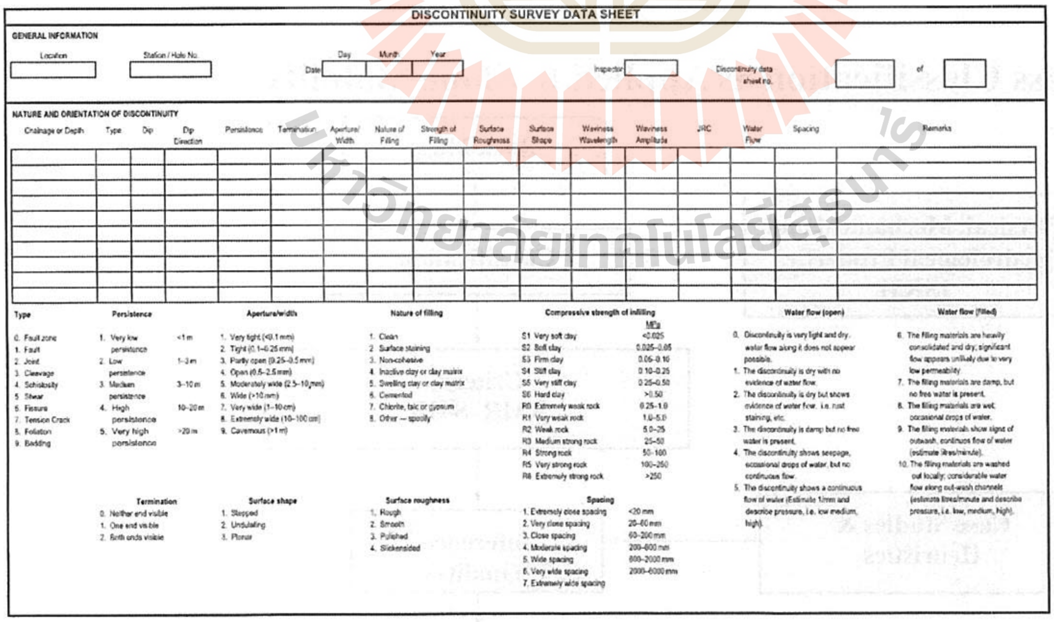
State of weathering:
1. Fresh
2. Slightly
3. Moderately
4. Highly
5. Completely
6. Residual soil

LINE SURVEYS TO DETERMINE DISCONTINUITY SPACINGS/ DRILL HOLE ORIENTATION

	Plunge of line / hole	Trend of line / hole	Length of line (m)	No. of Fracture	Spacing	Remarks / True Spacing
Line 1						
Line 2						
Line 3						

Discontinuity spacing:
1. Extremely close (<20 mm)
2. Very close (20–60 mm)
3. Close (60–200 mm)
4. Moderately (200–600 mm)
5. Wide (600–2000 mm)
6. Very wide (2000–6000 mm)
7. Ext. wide (>6000 m)

Discontinuity Survey Data Sheet



DISCONTINUITY SURVEY DATA SHEET

GENERAL INFORMATION
Location | Station / Hole No. | Date: Day, Month, Year | Inspector | Discontinuity data sheet no. ___ of ___

NATURE AND ORIENTATION OF DISCONTINUITY

Chainage or Depth	Type	Dip	Dip Direction	Persistence	Termination	Aperture/ Width	Nature of Filling	Strength of Filling	Surface Roughness	Surface Shape	Waviness Wavelength	Waviness Amplitude	JRC	Water Flow	Spacing	Remarks

Type:
0. Fault zone
1. Fault
2. Joint
3. Cleavage
4. Schistosity
5. Shear
6. Fissure
7. Tension Crack
8. Foliation
9. Bedding

Persistence:
1. Very low persistence <1 m
2. Low persistence 1–3 m
3. Medium persistence 3–10 m
4. High persistence 10–20 m
5. Very high persistence >20 m

Aperture/width:
1. Very tight (<0.1 mm)
2. Tight (0.1–0.25 mm)
3. Partly open (0.25–0.5 mm)
4. Open (0.5–2.5 mm)
5. Moderately wide (2.5–10 mm)
6. Wide (>10 mm)
7. Very wide (1–10 cm)
8. Extremely wide (10–100 cm)
9. Cavernous (>1 m)

Nature of filling:
1. Clean
2. Surface staining
3. Non-cohesive
4. Inactive clay or clay matrix
5. Swelling clay or clay matrix
6. Cemented
7. Chlorite, talc or gypsum
8. Other — specify

Compressive strength of infilling	MPa
S1 Very soft clay	<0.025
S2 Soft clay	0.025–0.05
S3 Firm clay	0.05–0.10
S4 Stiff clay	0.10–0.25
S5 Very stiff clay	0.25–0.50
S6 Hard clay	>0.50
R0 Extremely weak rock	0.25–1.0
R1 Very weak rock	1.0–5.0
R2 Weak rock	5.0–25
R3 Medium strong rock	25–50
R4 Strong rock	50–100
R5 Very strong rock	100–250
R6 Extremely strong rock	>250

Water flow (open):
0. Discontinuity is very tight and dry, water flow along it does not appear possible.
1. The discontinuity is dry with no evidence of water flow.
2. The discontinuity is dry but shows evidence of water flow, i.e. rust staining, etc.
3. The discontinuity is damp but no free water is present.
4. The discontinuity shows seepage, occasional drops of water, but no continuous flow.
5. The discontinuity shows a continuous flow of water (Estimate 1/mm and describe pressure, i.e. low medium, high).

Water flow (filled):
6. The filling materials are heavily consolidated and dry; significant flow appears unlikely due to very low permeability.
7. The filling materials are damp, but no free water is present.
8. The filling materials are wet; occasional drops of water.
9. The filling materials show signs of outwash, continuos flow of water (estimate litres/minute).
10. The filling materials are washed out locally; considerable water flow along out-wash channels (estimate litres/minute and describe pressure, i.e. low, medium, high).

Termination:
0. Neither end visible
1. One end visible
2. Both ends visible

Surface shape:
1. Stepped
2. Undulating
3. Planar

Surface roughness:
1. Rough
2. Smooth
3. Polished
4. Slickensided

Spacing:
1. Extremely close spacing <20 mm
2. Very close spacing 20–60 mm
3. Close spacing 60–200 mm
4. Moderate spacing 200–600 mm
5. Wide spacing 600–2000 mm
6. Very wide spacing 2000–6000 mm
7. Extremely wide spacing

Geologic Data needed for Slope Stability

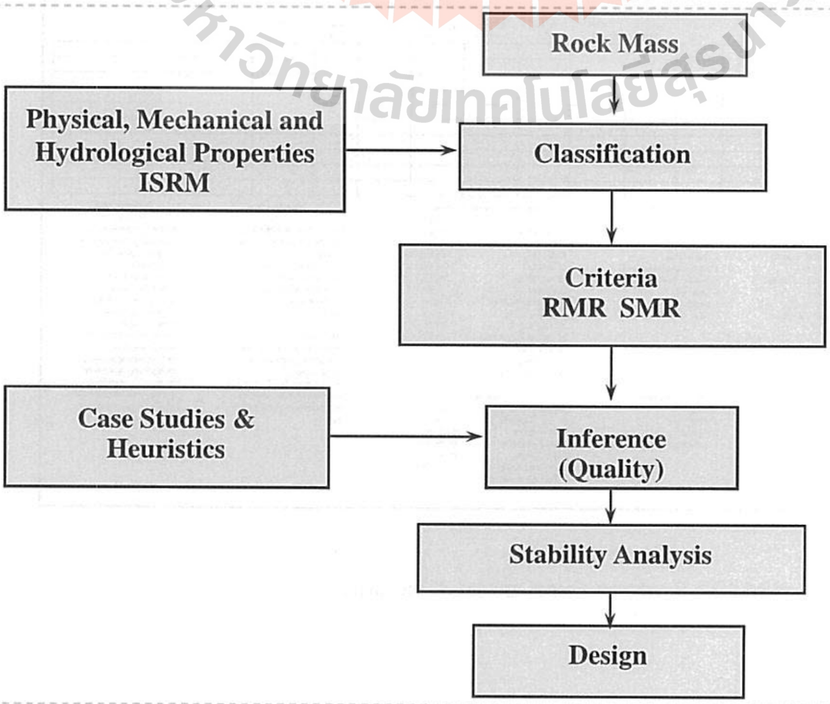
Field Data:

1. Location in relation to map references or pit plan
2. Depth
3. Orientation of discontinuities (strike/dip angle)
4. Spacing
5. Persistence (continuity)
6. Aperture (opening)
7. Gouge (infilling)
8. Roughness & Waviness
9. Field intact strength (point load strength index)
10. Groundwater conditions

Laboratory Test Data:

1. Direct shear strength test
2. Uniaxial compression test
3. Slake durability index
4. Short-term undrained shear strength of geologic materials
5. Long-term drained shear strength of geologic materials

Rock Mass Classification as Applied to Slope Stability



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	Maluta 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

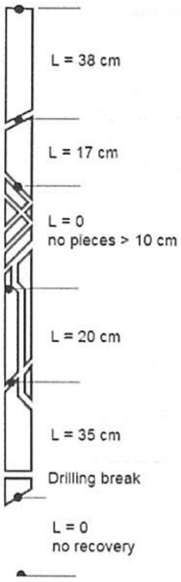
Rock Mass Classifications

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.

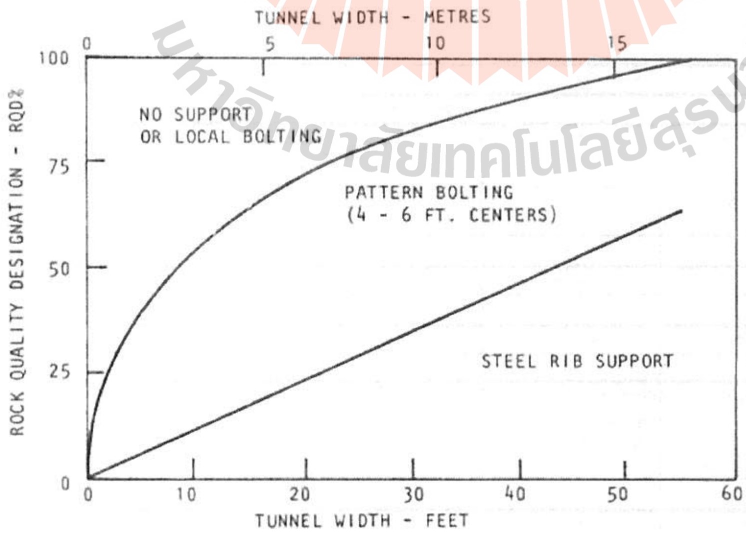
Rock Mass Classifications

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

Slope Mass Rating (SMR)

$$SMR = RMR_{basic} - (F_1.F_2.F_3) + F_4$$

- RMR_{basic} = Rock Mass Rating)
- F_1, F_2, F_3 = adjustment factor related to joint orientation respect to slope orientation and
- F_4 = correction factor for method of excavation

Slope Mass Rating (SMR)

Values of adjustment factor for different joint orientations (RAMANA, 1985)

Case of Slope Failure		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
P T W	$\lvert\alpha_j - \alpha_s\rvert$ $\lvert\alpha_j - \alpha_s - 180^o\rvert$ $\lvert\alpha_i - \alpha_s\rvert$	>30°	30 - 20°	20 - 10°	10 - 5°	<5°
P/W/T	**F_1**	**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
P W	$\lvert\beta_j\rvert$ $\lvert\beta_i\rvert$	<20°	20 - 30°	30 - 35°	35 - 45°	>45°
P/W	**F_2**	**0.15**	**0.40**	**0.70**	**0.85**	**1.00**
T	**F_2**	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**
P W	$\lvert\beta_j - \beta_s\rvert$ $\lvert\beta_i - \beta_s\rvert$	>10°	10-0°	0°	0 - (-10°)	<-10°
T	$\lvert\beta_j + \beta_s\rvert$	<110°	110 - 120°	>120°	-	-
P/W/T	**F_3**	**0**	**-6**	**-25**	**-50**	**-60**

Note : P - planar failure; T - toppling failure; W - wedge failure
α_s - slope strike; α_j - joint strike; α_i - plunge direction of line of intersection
β_s - slope dip; β_j - joint dip; β_i - plunge of line of intersection

Slope Mass Rating (SMR)

Values of adjustment factor F4 for method of excavation (RAMANA, 1985)

Method of Excavation	**F_4 Value**
Natural slope	+15
Pre-splitting	+10
Smooth blasting	+8
Normal blasting or Mechanical excavation	0
Poor blasting	-8

Slope Mass Rating (SMR)

Various stability classes as per SMR values (RAMANA, 1985)

Class No.	**V**	**IV**	**III**	**II**	**I**
SMR Value	0-20	21-40	41-60	61-80	81-100
Rock Mass Description	Very bad	Bad	Normal	Good	Very good
Stability	Completely unstable	Unstable	Partially stable	Stable	Completely stable
Failures	Big planar or soil like or circular	Planar or big wedges	Planar along some joint and many wedges	Some block failure	No failure
Probability of Failure	0.9	0.6	0.4	0.2	0

Slope Mass Rating (SMR)

Suggested supports for various SMR classes

SMR Classes	SMR Values	Suggested Supports
Ia	91-100	None
Ib	81-90	None, scaling is required
IIa	71-80	(None, toe ditch or fence), spot bolting
IIb	61-70	(Toe ditch or fence nets), spot or systematic bolting, spot shotcrete
IIIa	51-60	(Toe ditch and/or nets), spot or systematic bolting, spot shotcrete
IIIb	41-50	(Toe ditch and/or nets), systematic bolting/anchors, systematic shotcrete, toe wall and/or dental concrete
IVa	31-40	Anchors, systematic shotcrete, toe wall and/or concrete (or re-excavation), drainage
IVb	21-30	Systematic reinforced shotcrete, toe wall and/or concrete, re-excavation, deep drainage
Va	11-20	Gravity or anchored wall, re-excavation

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.

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)

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	1.0	minimum strength.
enough to prevent rock wall contact	(nominal)	
J. Sandy, gravely or crushed zone thick	1.0	
enough to prevent rock wall contact	(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	SRF	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated		
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 disintegrated rock (excavation depth < 50 m)	5.0	
C. Single weakness zones containing clay, or chemically disintegrated 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	

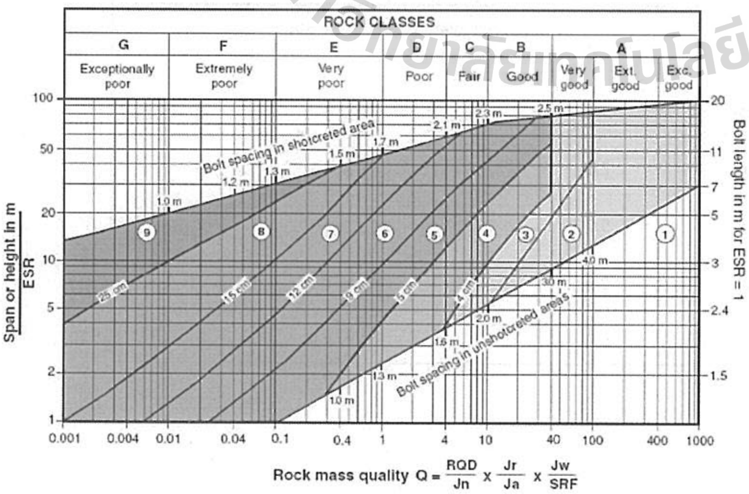
Rock Tunnelling Quality Index, Q (NGI)

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			*SRF*	
b. Competent rock, rock stress problems				
	σ_c/σ_1	$\sigma_t\sigma_1$		
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	
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	

NOTES:

2. For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, 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.
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).

Estimated support categories



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

REINFORCEMENT CATEGORIES:

1) Unsupported
2) Spot bolting
3) Systematic bolting
4) Systematic bolting, (and unreinforced shotcrete, 4 - 10 cm)
5) Fibre reinforced shotcrete and bolting, 5 - 9 cm
6) Fibre reinforced shotcrete and bolting, 9 - 12 cm
7) Fibre reinforced shotcrete and bolting, 12 - 15 cm
8) Fibre reinforced shotcrete, > 15 cm, reinforced ribs of shotcrete and bolting
9) Cast concrete lining

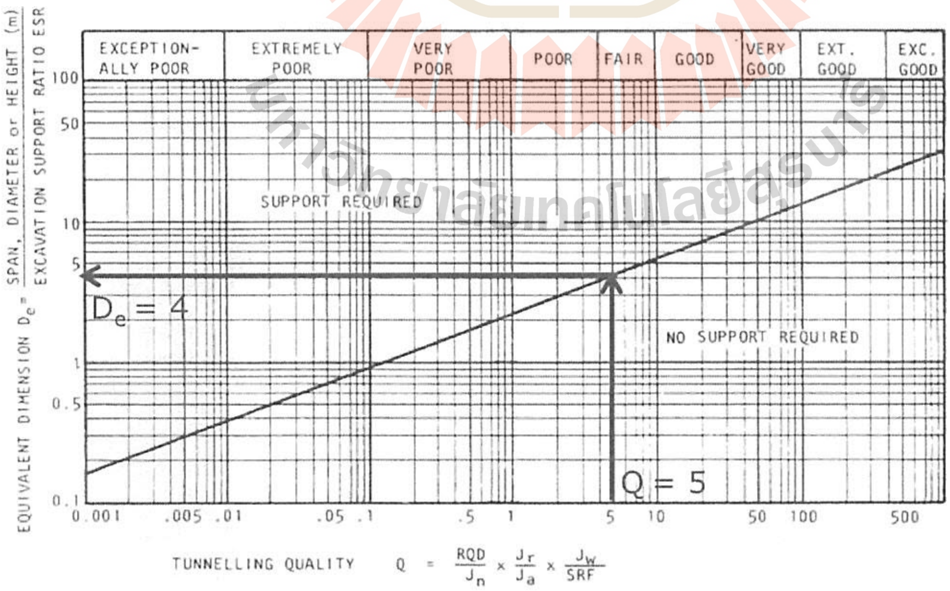
Example

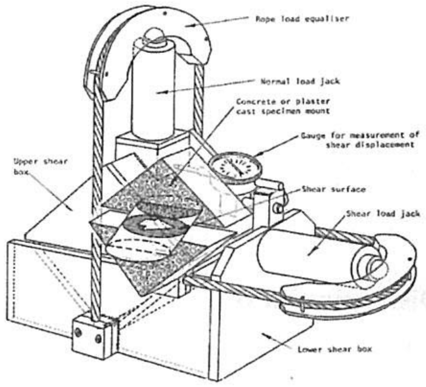
Item	Description	Value
1. Rock Quality	Good	RQD = 80%
2. Joint sets	Two sets	Jn = 4
3. Joint roughness	Rough	Jr = 3
4. Joint alteration	Clay gouge	Ja = 4
5. Joint water	Large inflow	Jw = 0.33
6. Stress reduction	Medium stress	SRF = 1.0

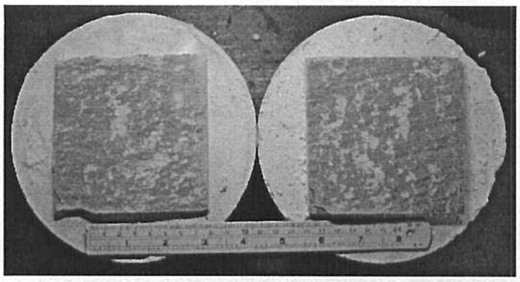
$$Q = \frac{80}{4} \times \frac{3}{4} \times \frac{0.33}{1} = 5$$

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)





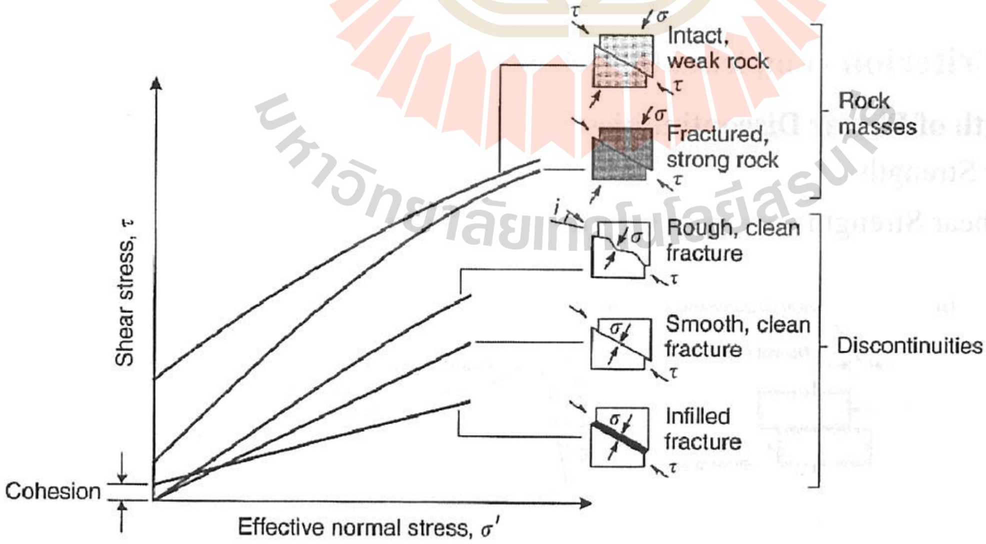


434422 Surface Excavation & Design

Topic 5 Rock Shear Strength Properties and their Measurement

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

Shear stress vs. Normal stress



Joint Shear Strength

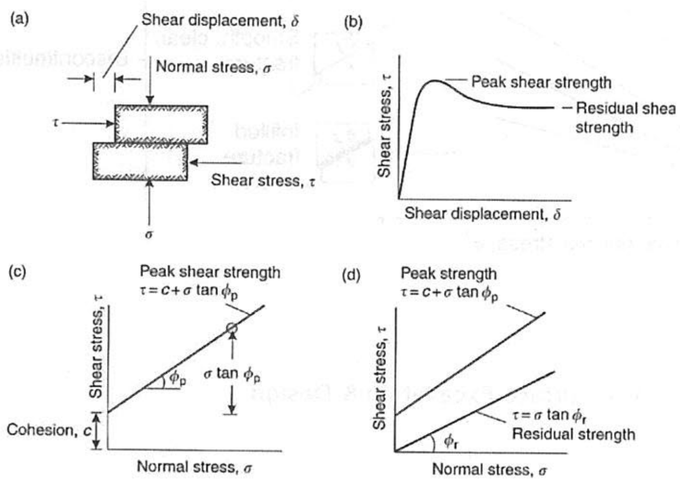
Criterions

1. **Coulomb Criterion** (Shear Strength of Planar Discontinuities)
2. **Patton Criterion** (Shear strength on an inclined plane)
3. **Ladanyi and Archambault Criterion** (Surface Roughness)
4. **Barton Criterion** (Surface Roughness)
5. **Hoek and Brown Criterion** (Fractured Rock Masses)

Coulomb Criterion (Empirical Criterion)

Shear Strength of Planar Discontinuities

- Peak Shear Strength
- Residual Shear Strength



Coulomb Criterion

- **Peak Shear Strength**

$$\tau = c_p + \sigma \tan \phi_p$$

$$\tau = c_p + (\sigma\text{-}u) \tan \phi_p \qquad \textbf{(Effective Stress Law)}$$

- **Residual Shear Strength**

$$\tau = \sigma \tan \phi_r$$

$$\tau = (\sigma\text{-}u) \tan \phi_r \qquad \textbf{(Effective Stress Law)}$$

where u is the water pressure within the discontinuity

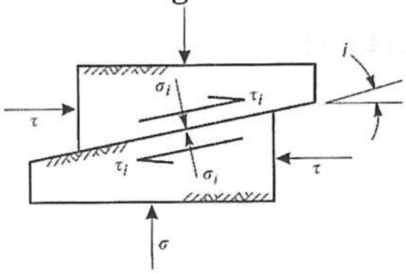
Coulomb Criterion

Table 4.1 Typical ranges of friction angles for a variety of rock types

Rock class	*Friction angle range*	*Typical rock types*
Low friction	20–27°	Schists (high mica content), shale, marl
Medium friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High friction	34–40°	Basalt, granite, limestone, conglomerate

Patton Criterion

Shear strength on an inclined plane



i = *asperities*

$$\tau_i = \tau \, Cos^2 i - \sigma \sin i \, Cos \, i \qquad (6.3)$$

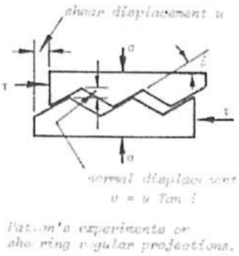
$$\sigma_i = \sigma \, Cos^2 i - \sigma \sin i \, Cos \, i \qquad (6.4)$$

If it is assumed that the discontinuity surface has zero cohesive strength and that its shear strength is given by

$$\tau_i = \sigma_i \, Tan \, \phi \qquad (6.5)$$

sub equation 6.3 & 6.4 into equation 6.5

$$\tau = \sigma \, Tan \, (\phi + i) \qquad (6.6)$$



Patton Criterion

Shear strength on an inclined plane

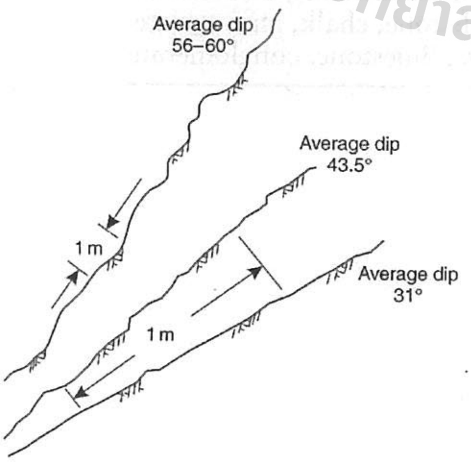


Figure 4.10 Patton's observations of bedding plane traces in unstable limestone slopes (Patton, 1966).

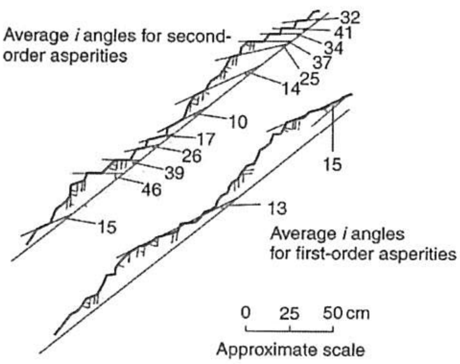


Figure 4.11 Measurement of roughness angles i for first- and second-order asperities on rough rock surfaces (Patton, 1966).

Patton Criterion

Shear strength on an inclined plane

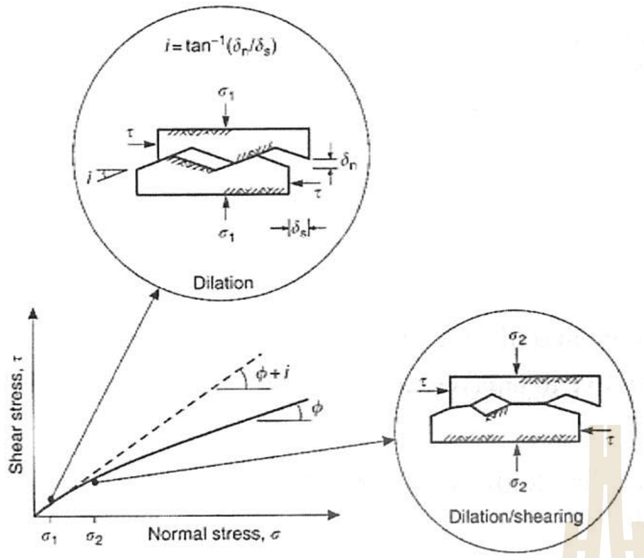


Figure 4.12 Effect of surface roughness and normal stress on friction angle of discontinuity surface (Transportation Research Board, 1996).

Ladanyi and Archambault Criterion

Surface Roughness

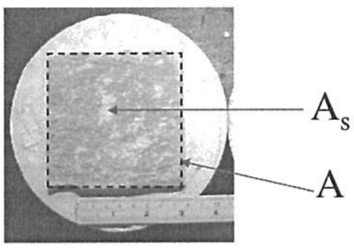
$$\tau = \frac{\sigma(1-a_s)(\dot{v}+\tan\phi)+a_s.\tau_r}{1-(1-a_s)\dot{v}\tan\phi}$$

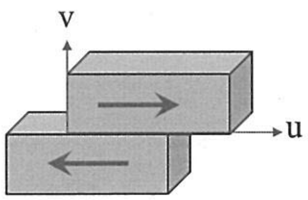
where a_s = proportion of the discontinuity surface which is sheared through projections of intact rock material

$= A_s/A$

$\dot{v}$ = dilation rate dv/du at peak shear strength

τ_r = shear strength of the intact rock material





Ladanyi and Archambault Criterion

Shear strength proposal by Fairhurst (1964):

$$\tau_r = \sigma_J \frac{\sqrt{1+n}-1}{n}\left(1+n\frac{\sigma}{\sigma_J}\right)^{\frac{1}{2}}$$

where σ_J = uniaxial compressive strength of the rock material (σ_C)

n = ratio of uniaxial compressive to uniaxial tensile strength (σ_C/σ_T)

Hoek (1968) has suggested that, for most hard rocks, n is approximately equal to 10

$$v = \left(1-\frac{\sigma}{\sigma_J}\right)^K \tan i \quad \text{and} \quad a_s = 1-\left(1-\frac{\sigma}{\sigma_J}\right)^L$$

where, for rough rock surfaces, K = 4 and L = 1.5.

Effect of asperities on stability of sliding block

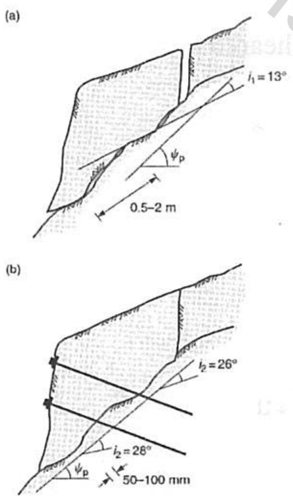


Figure 4.13 Effect of asperities on stability of sliding blocks: (a) shear strength of displaced block controlled by first-order asperities (i_1); (b) tensioned rock bolts prevent dilation along potential sliding surface and produce interlock along second-order asperities (i_2).

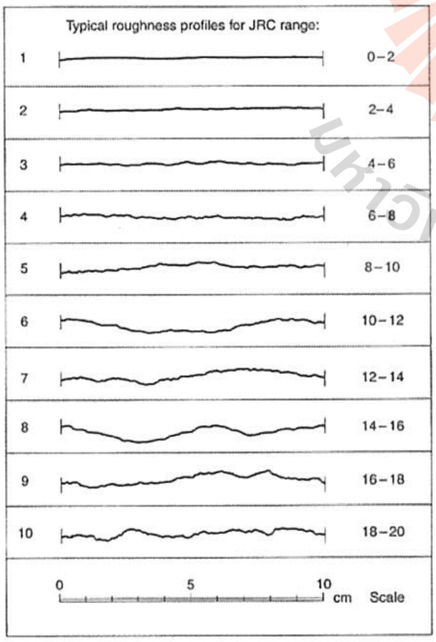
Barton Criterion

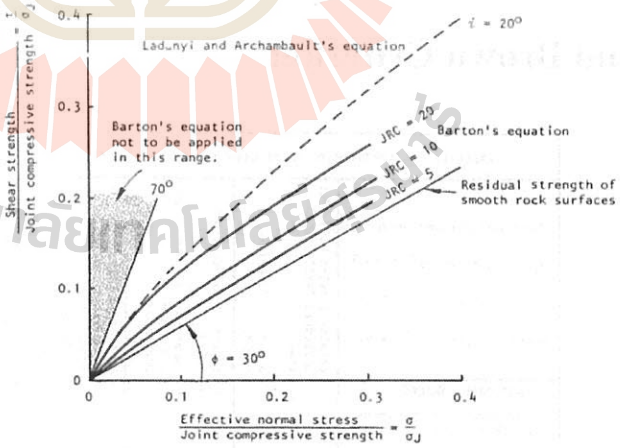
Predicting the shear strength of rough joints was proposed by Barton (1973)

$$\tau = \sigma \tan\left(\phi_b + JRC.Log_{10}\frac{\sigma_J}{\sigma}\right)$$

where JRC = Joint Roughness Coefficient

Barton Criterion





For comparison the residual strength of a smooth joint with $\phi = 30°$ and Ladanyi and Archambault's equation for $i = 20$ and $\phi = 30$

Hoek and Brown Criterion

Fractured Rock Masses *(Closely Jointed Rock Masses)*

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

where σ_c = uniaxial compressive strength of the intact rock pieces and

m and s = dimensionless constants which depend upon the shape and degree of interlocking of the individual pieces of rock within the mass.

$$\tau = A\sigma_c\,(\sigma/\sigma_3 - T)^B$$

where A and B = constants defining the shape of the Mohr failure envelope and

$$T = \frac{1}{2}\left(m - \sqrt{m^2 + 4s}\right)$$

Hoek and Brown Criterion

TABLE IV - APPROXIMATE RELATIONSHIP BETWEEN ROCK MASS QUALITY AND EMPIRICAL CONSTANTS

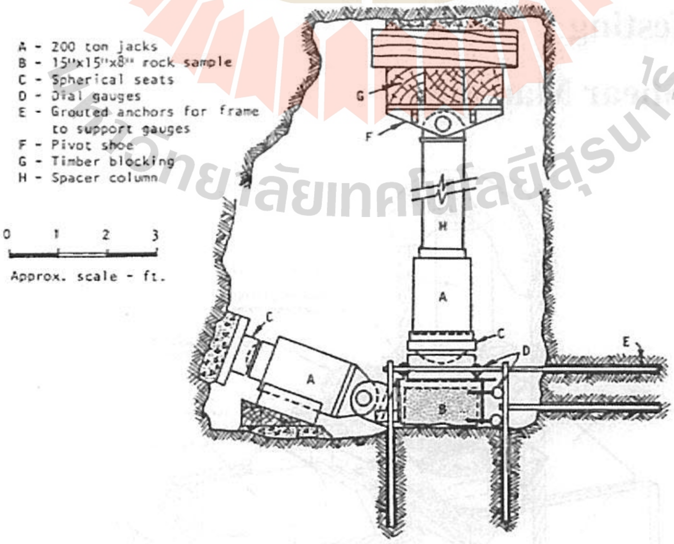
Empirical failure criterion $\sigma_1 = \sigma_3 + \sqrt{m\sigma_c\sigma_3 + s\sigma_c^2}$ $\tau = A\sigma_c(\sigma/\sigma_c - T)^B$ where $T = \frac{1}{2}(m - \sqrt{m^2 + 4s})$	CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, 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 AND METAMORPHIC CRYSTALLINE ROCKS *amphibolite, gabbro gneiss, granite, norite and quartz-diorite.*
INTACT ROCK SAMPLES *Laboratory size specimens free from joints* CSIR rating 100 NGI rating 500	m = 7.0 s = 1.0 A = 0.816 B = 0.658 T = -0.140	m = 10.0 s = 1.0 A = 0.918 B = 0.677 T = -0.099	m = 15.0 s = 1.0 A = 1.044 B = 0.692 T = -0.067	m = 17.0 s = 1.0 A = 1.086 B = 0.696 T = -0.059	m = 25.0 s = 1.0 A = 1.220 B = 0.705 T = -0.040
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at ±3m.* CSIR rating 85 NGI rating 100	m = 3.5 s = 0.1 A = 0.651 B = 0.679 T = -0.028	m = 5.0 s = 0.1 A = 0.739 B = 0.692 T = -0.020	m = 7.5 s = 0.1 A = 0.848 B = 0.702 T = -0.013	m = 8.5 s = 0.1 A = 0.883 B = 0.705 T = -0.012	m = 12.5 s = 0.1 A = 0.998 B = 0.712 T = -0.008
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.* CSIR rating 65 NGI rating 10	m = 0.7 s = 0.004 A = 0.369 B = 0.669 T = -0.006	m = 1.0 s = 0.004 A = 0.427 B = 0.683 T = -0.004	m = 1.5 s = 0.004 A = 0.501 B = 0.695 T = -0.003	m = 1.7 s = 0.004 A = 0.525 B = 0.698 T = -0.002	m = 2.5 s = 0.004 A = 0.603 B = 0.707 T = -0.002

Hoek and Brown Criterion

FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1m.* CSIR rating 44 NGI rating 1.0	m = 0.14 s = 0.0001 A = 0.198 B = 0.662 T = -0.0007	m = 0.20 s = 0.0001 A = 0.234 B = 0.675 T = -0.0005	m = 0.30 s = 0.0001 A = 0.280 B = 0.688 T = -0.0003	m = 0.34 s = 0.0001 A = 0.295 B = 0.691 T = -0.0003	m = 0.50 s = 0.0001 A = 0.346 B = 0.700 T = -0.0002
POOR QUALITY ROCK MASS *Numerous weathered joints at 30 to 500mm with some gouge - clean waste rock.* CSIR rating 23 NGI rating 0.1	m = 0.04 s = 0.00001 A = 0.115 B = 0.646 T = -0.0002	m = 0.05 s = 0.00001 A = 0.129 B = 0.655 T = -0.0002	m = 0.08 s = 0.00001 A = 0.162 B = 0.672 T = -0.0001	m = 0.09 s = 0.00001 A = 0.172 B = 0.676 T = -0.0001	m = 0.13 s = 0.00001 A = 0.203 B = 0.686 T = -0.0001
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced < 50mm with gouge - waste with fines.* CSIR rating 3 NGI rating 0.01	m = 0.007 s = 0 A = 0.042 B = 0.534 T = 0	m = 0.010 s = 0 A = 0.050 B = 0.539 T = 0	m = 0.015 s = 0 A = 0.061 B = 0.546 T = 0	m = 0.017 s = 0 A = 0.065 B = 0.548 T = 0	m = 0.025 s = 0 A = 0.078 B = 0.556 T = 0

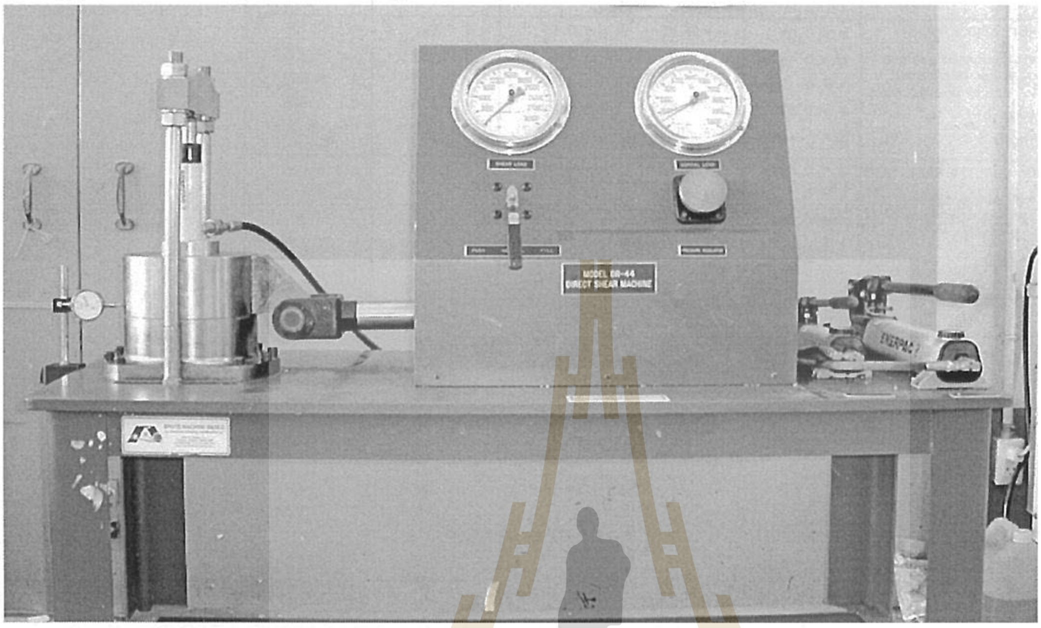
Shear Strength Testing

Field (In-situ) Tests



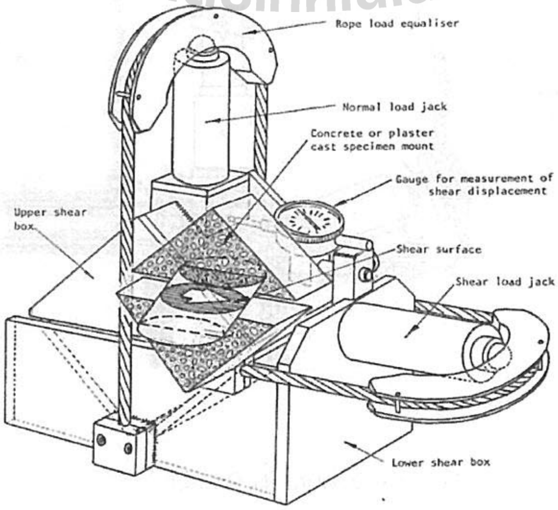
Shear Strength Testing

Large Scale Laboratory Tests



Shear Strength Testing

Portable Direct Shear Machine



Direct Shear Strength Testing

Designation: D 5607 – 02 (Reapproved 2006)

Standard Test Method for
Performing Laboratory Direct Shear Strength Tests of Rock Specimens Under Constant Normal Force[1]

This standard is issued under the fixed designation D 5607; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

Objectives: to determine shear strength of intact rock

Direct Shear Strength Testing

Apparatus

1) Direct Shear Machine
2) Pressure Maintain Device
3) Displacement Measurement Device

Specimens

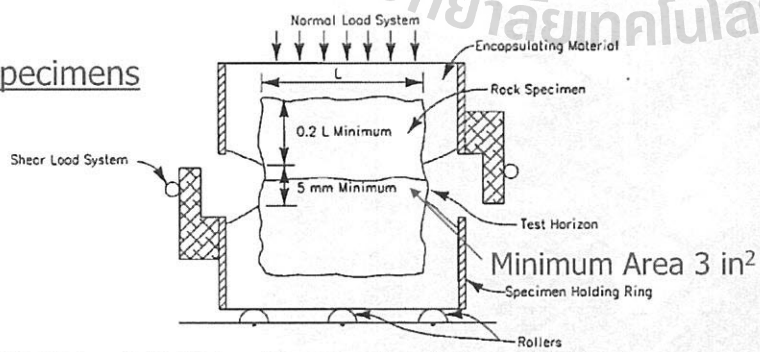
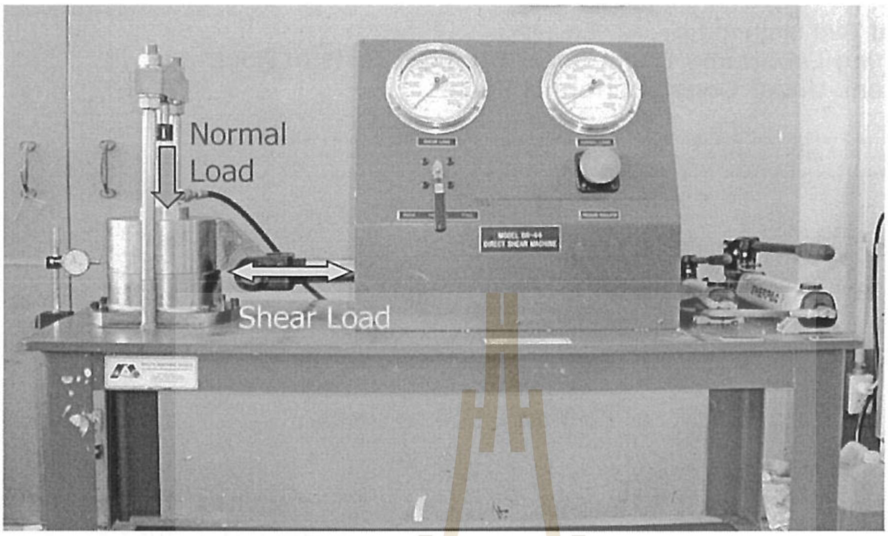


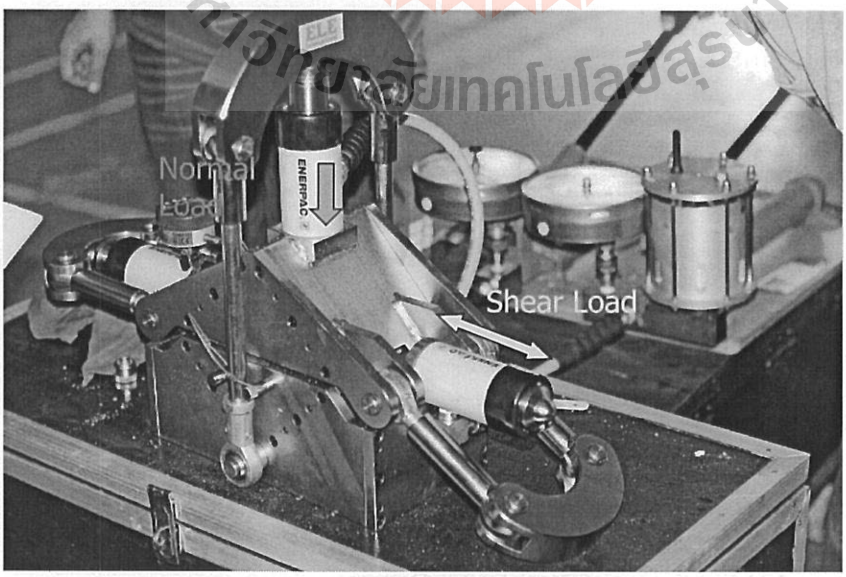
FIG. 2 Schematic Test Setup—Direct Shear Box with Encapsulated Specimen

Direct Shear Strength Testing



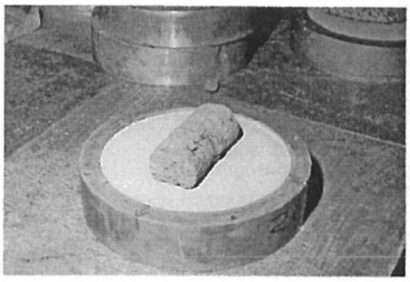
Laboratory Direct Shear Machine

Direct Shear Strength Testing



Portable Direct Shear Machine

Direct Shear Strength Testing



Note 1—In both Fig. 5 and Fig. 6 the shear box is cylindrical. Square boxes work just as well.
FIG. 6 Lower Half of a Specimen Encapsulated in Holding Ring

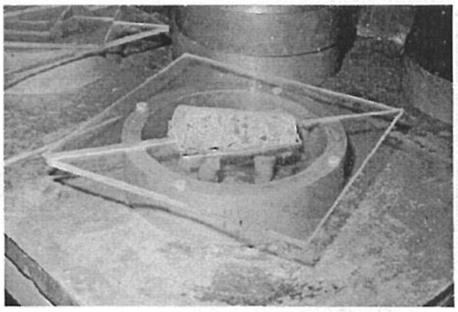
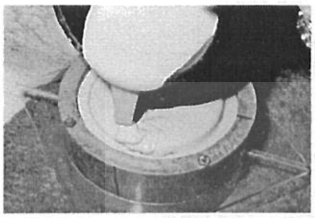


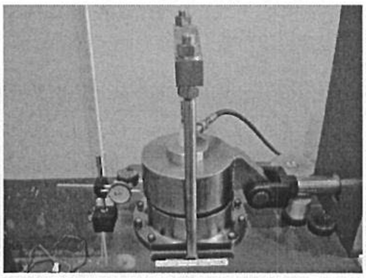
FIG. 5 Specimen Supported in Place By Modeling Clay Pins Which Are Removed After Encapsulating Material Cures and the Resulting Holes Filled With Encapsulating Material



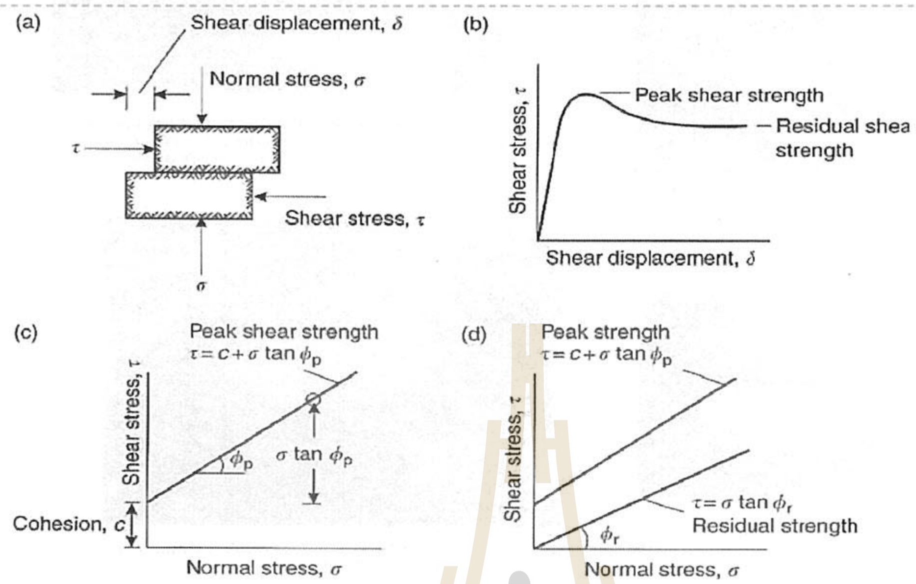
Note 1—Note the split plastic plates for isolating the shear zone.
FIG. 3 View Showing Pouring Encapsulating Material Around Upper Half of Specimen

FIG. 7 Removing Spacer Plates After Encapsulating Material Has Cured

Direct Shear Strength Testing



Direct Shear Strength Testing



Direct Shear Strength Testing

Calculate the following engineering stresses:

Apparent normal stress $\sigma = \frac{P_n}{A}$

Apparent shear stress $\tau = \frac{P_s}{A}$

where:
P_n = normal load,
P_s = shear load, and
A = nominal initial cross-sectional area

For Core Specimens

the area is determined by:

$$A = \frac{\pi D^2}{4 \cos\Theta}$$

where:
D = core diameter, and
Θ = angle of tip.

Direct Shear Strength Testing

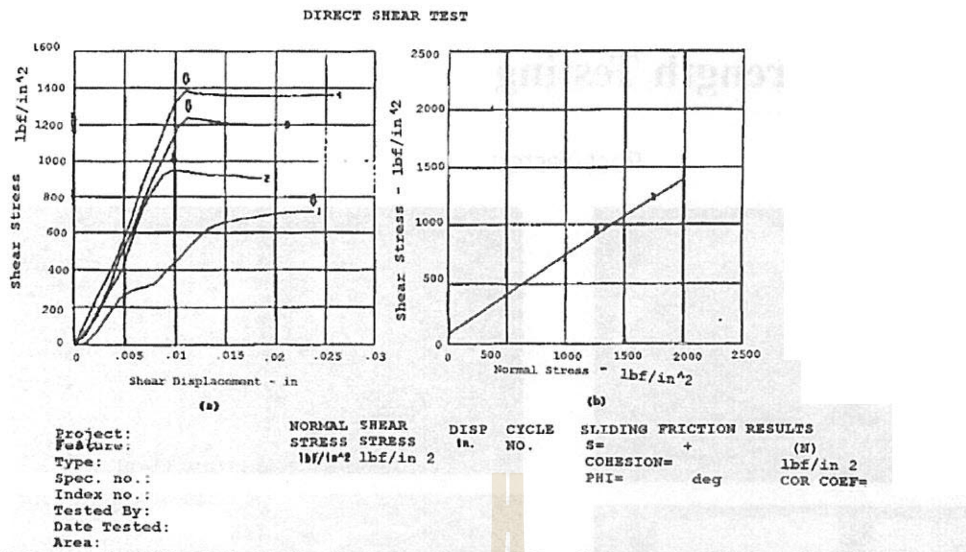
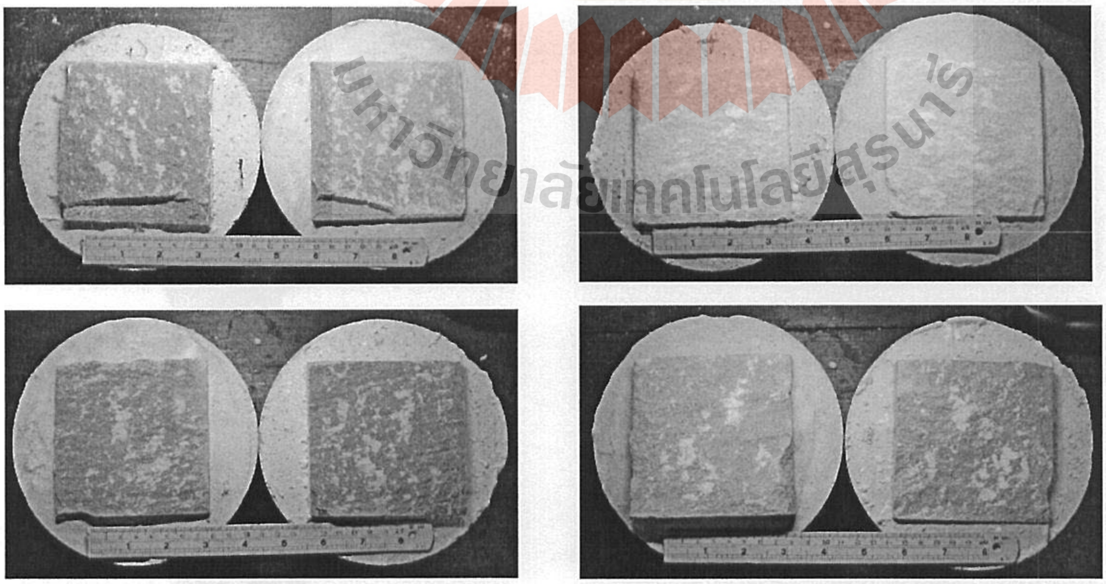


FIG. 8 Typical Presentation Sliding Friction Test Results: (a) Shear Stress and Shear Displacement and (b) Shear Strength and Normal Stress

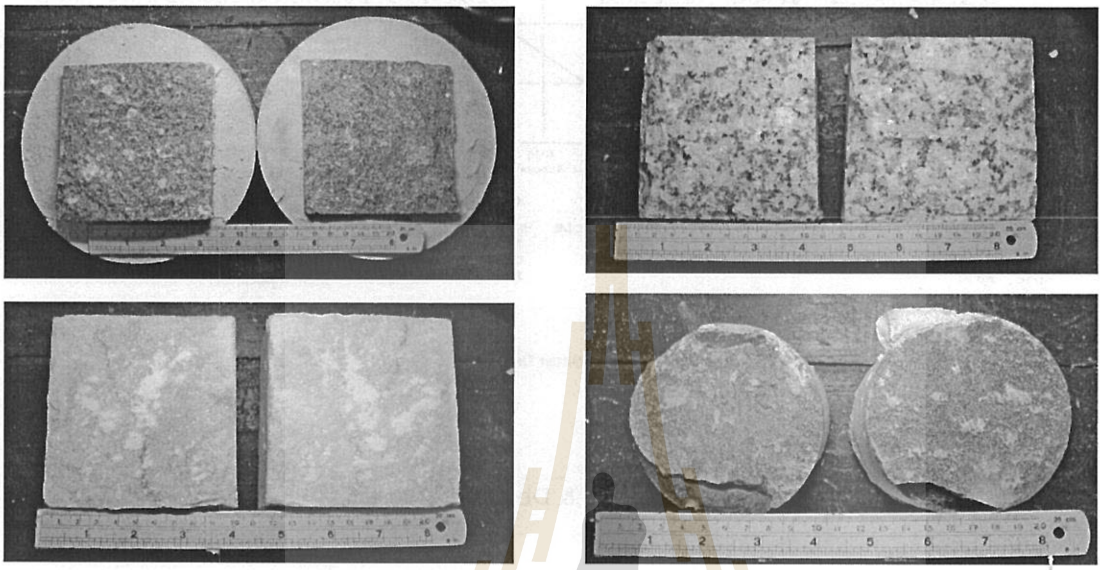
Direct Shear Strength Testing

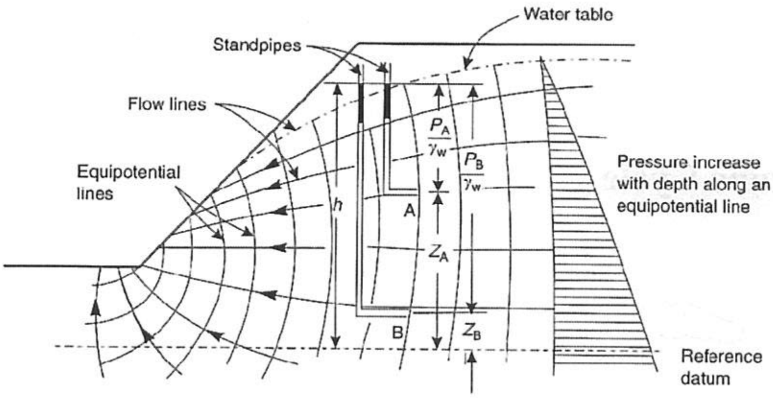
Post-Tested



Direct Shear Strength Testing

Post-Tested





434422 Surface Excavation & Design
Topic 6 Groundwater Flow and Pressure

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

Groundwater Flow in Rock Mass

Two approaches to obtained data on water pressure distributions

1. **Deduction of the groundwater flow pattern** from consideration of the permeability of the rock mass and sources of groundwater. **(calculation / graphical methods)**

2. **Direct measurement of water levels** in boreholes or wells or of water pressure by means of piezometers installed in boreholes.

The Hydrologic Cycle

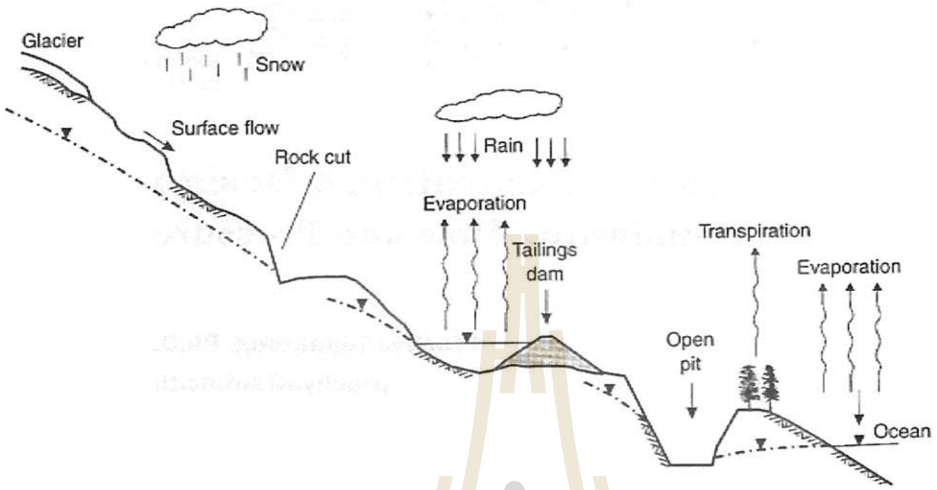


Figure 5.1 Simplified representation of a hydrologic cycle showing some typical sources of ground water (modified from Davis and de Wiest (1966)).

Water Table

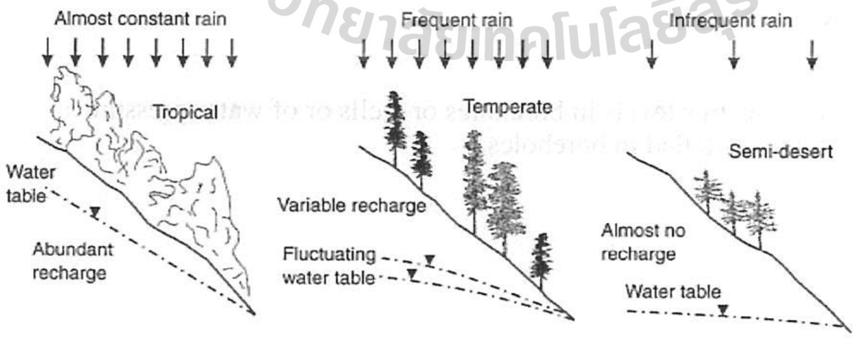
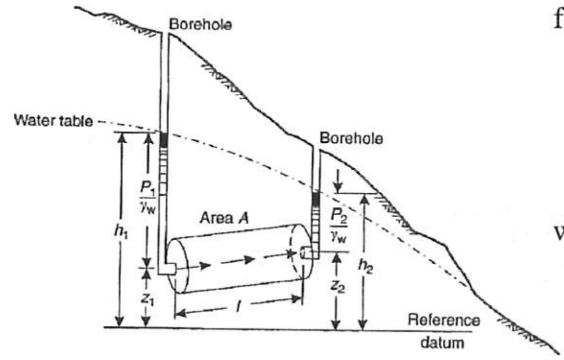


Figure 5.2 Relationship between water table level and precipitation (modified from Davis and de Wiest (1966)).

Darcy's Law



for Homogenous Materials

$$v = Ki$$

where v = velocity of fluid

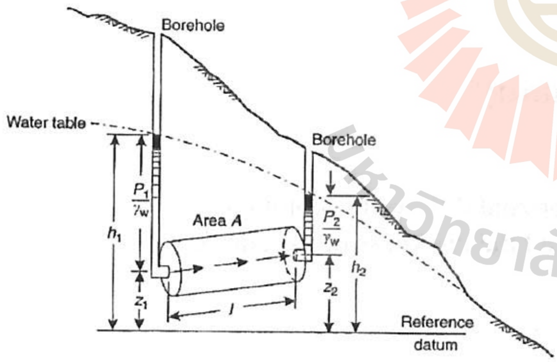
K = hydraulic conductivity (m/s, ft/s)

i = hydraulic gradient $((h_1\text{-}h_2)/l)$

when Q = flow rate (Q = v/A)

$$Q = KAi$$

Darcy's Law



from

$$Q = KAi$$

so that;

$$K = Q/Ai$$

sub $i = (h_1\text{-}h_2)/l$

$$K = \frac{Ql}{A(h_1 - h_2)} = \frac{Vl}{(h1 - h_2)}$$

or $Q = \dfrac{KA(h_1 - h_2)}{l}$

V = Q/A = discharge velocity

Hydraulic Conductivity

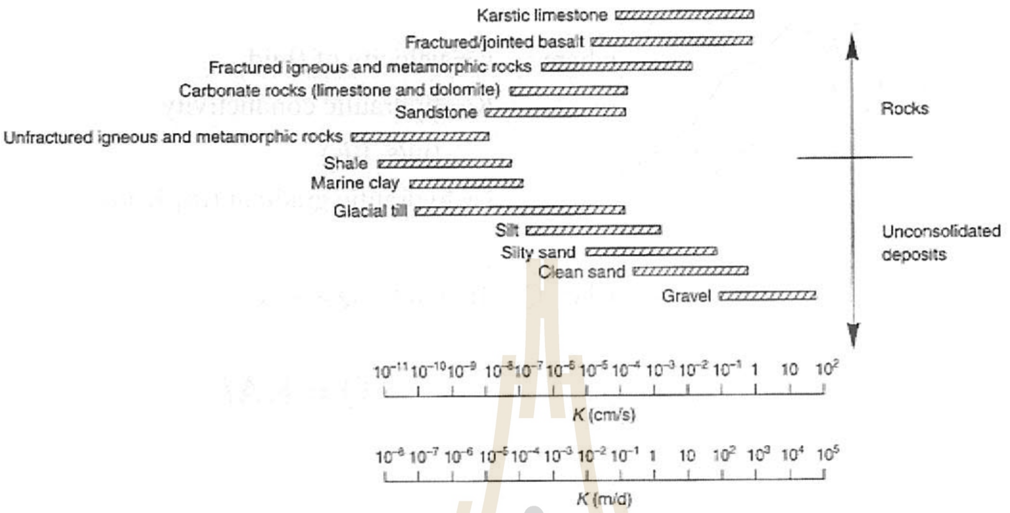


Figure 5.4 Hydraulic conductivity of various geologic materials (Atkinson, 2000).

Other Measures of the Flow Proportionality

- *Transmissivity*

  In saturated groundwater analysis with nearly horizontal flow, it is common practice to combine the hydraulic conductivity and the thickness of the aquifer, b into a single variable,

  $$T = bK$$

  where T = transmissivity (m^2/s, ft^2/s)

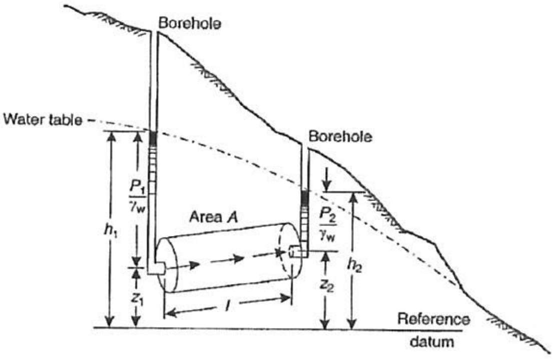
- *Permeability*

  When the fluid is other than water at standard conditions, the conductivity is replaced by the permeability of the media. The two properties are related by,

  $$K = k\gamma / \mu$$

  where,
  k = permeability, (m^2 or ft^2),
  γ = unit weight of fluid
  μ = fluid absolute viscosity, ($N\ s/m^2$ or $lb\ s/ft^2$)

Bernoulli Equation



$$zg + \frac{v^2}{2} + \int_{P_1}^{P_2} \frac{\partial(P - P_0)}{\rho} = \text{constant}$$

or

$$z + \frac{v^2 g}{2} + \frac{\Delta P}{\rho g} + \text{Friction Head} = \text{Total Head}$$

Velocity Head ($\frac{v^2 g}{2}$), Elevation Head (z), Pressure Head ($\frac{\Delta P}{\rho g}$)

<u>Case of flow in rock mass</u>

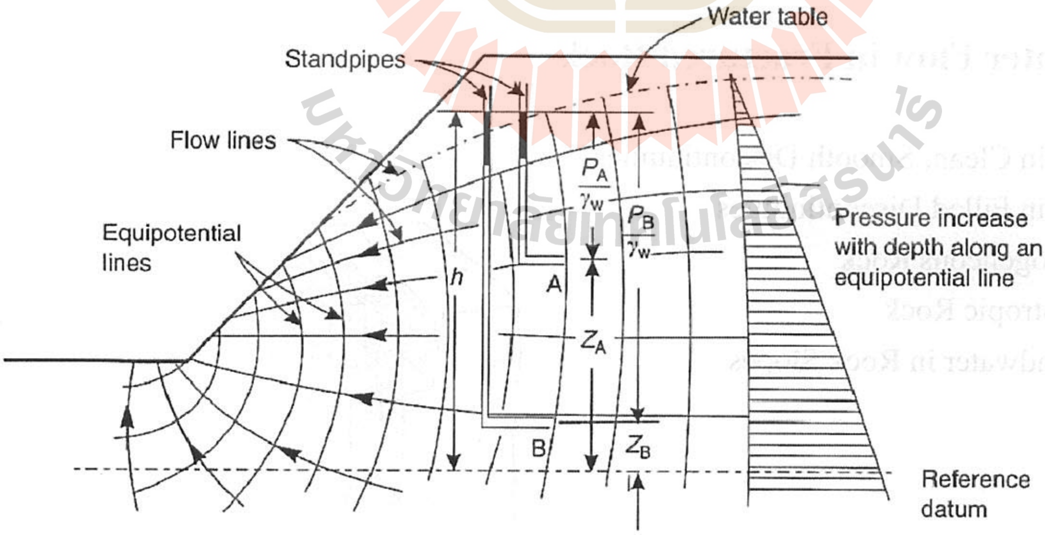
Velocity Head = 0

Friction Head = 0

$$\text{Total Head} = z + \frac{\Delta P}{\rho g} = z + \frac{P}{\gamma_w}$$

γ_w = density of water

Flow Nets in a Rock Slope



$$\text{Total Head} = z + \frac{P}{\gamma_w}$$

Discharge / Recharge Area

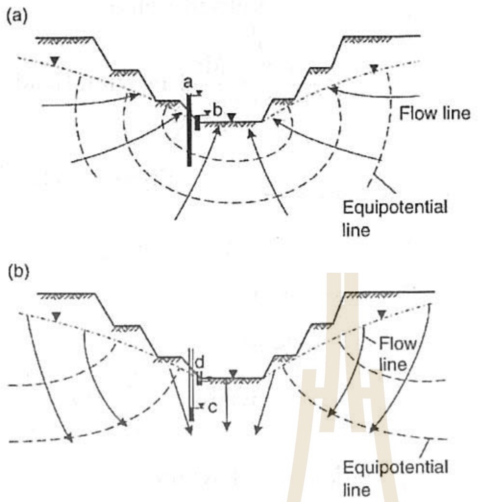


Figure 5.6 Ground water conditions for pit slopes in regional (a) discharge and (b) recharge areas (Patton and Deere, 1971).

Groundwater Flow in Fractured Rock

- Flow in Clean, Smooth Discontinuities
- Flow in Filled Discontinuities
- Heterogeneous Rock
- Anisotropic Rock
- Groundwater in Rock Slopes

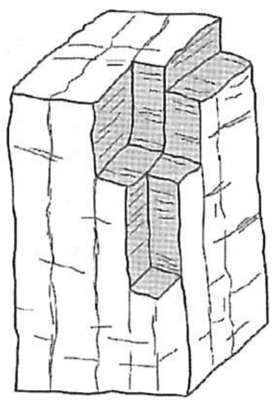


Figure 5.7 Rock mass with persistent vertical joints and relatively high vertical hydraulic conductivity (modified from Atkinson (2000)).

Flow in Clean, Smooth Discontinuities

The Hydraulic Conductivity, K

$$K \approx \frac{ge^3}{12\nu b}$$

where g = gravitational acceleration (9.81 m/s^2)

e = opening of cracks or fissures

b = spacing between cracks and

ν = the coefficient of kinematic viscosity (1.01×10^{-6} m^2/s for pure water at 20°C)

Assumptions of Discontinuities;

- parallel
- smooth
- clean
- laminar flow

Flow in Clean, Smooth Discontinuities

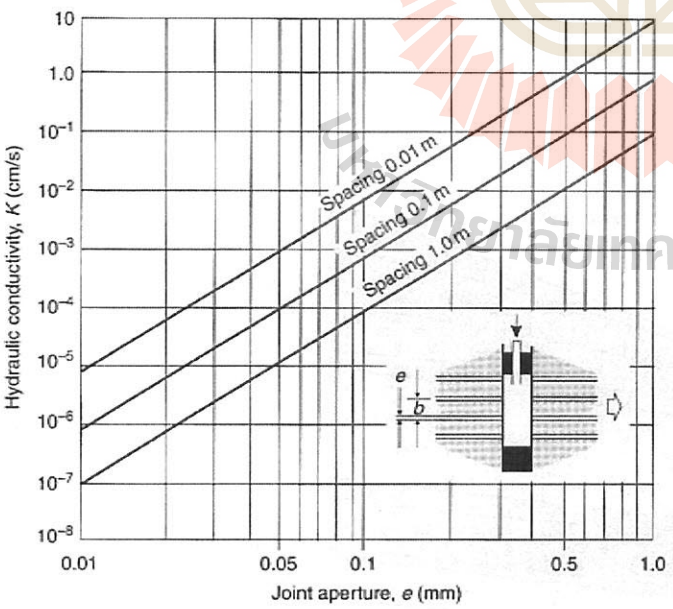


Figure 5.8 Influence of joint aperture *e* and spacing *b* on hydraulic conductivity *K* in the direction of a set of smooth parallel joints in a rock mass.

Flow in Filled Discontinuities

The Hydraulic Conductivity for Fracture Systems

$$K = \frac{eK_f}{b} + K_r$$

where K_f = hydraulic conductivity of the filling

K_r = hydraulic conductivity of the intact rock

e = opening of cracks or fissures

b = spacing between cracks and

Heterogeneous Rock

$$\frac{K_1}{K_2} = \frac{\tan\theta_1}{\tan\theta_2}$$

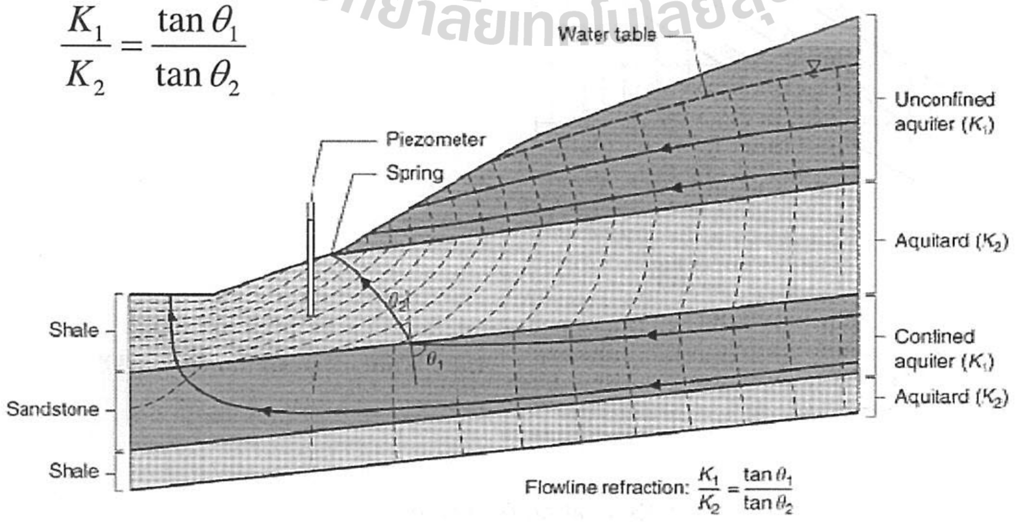


Figure 5.9 Water flow and pressure distribution in aquifers and aquitards formed by dipping sandstone and shale beds (Dr P. Ward, plots by W. Zawadzki).

Anisotropic Rock

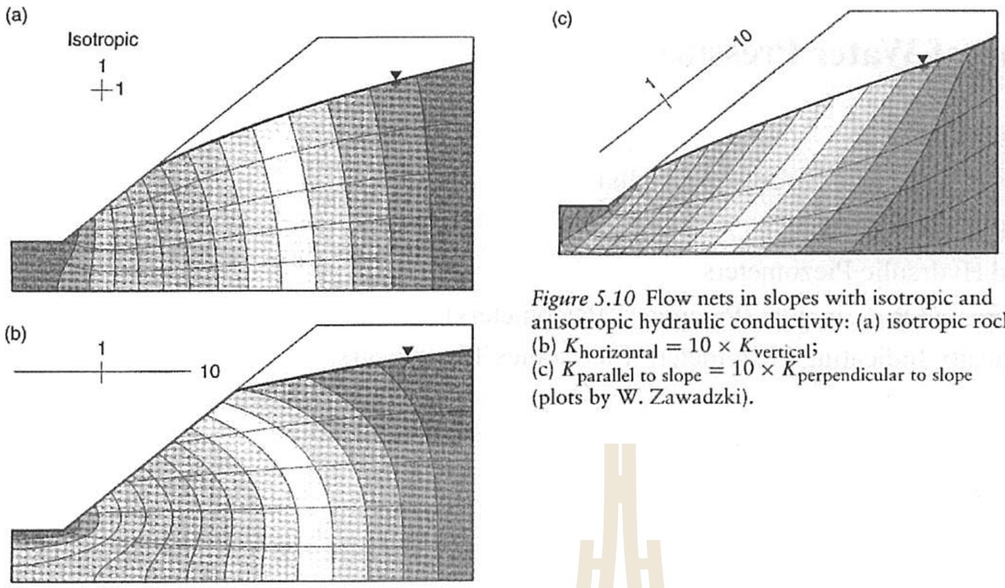


Figure 5.10 Flow nets in slopes with isotropic and anisotropic hydraulic conductivity: (a) isotropic rock; (b) $K_{horizontal} = 10 \times K_{vertical}$; (c) $K_{parallel\ to\ slope} = 10 \times K_{perpendicular\ to\ slope}$ (plots by W. Zawadzki).

Groundwater in Rock Slopes

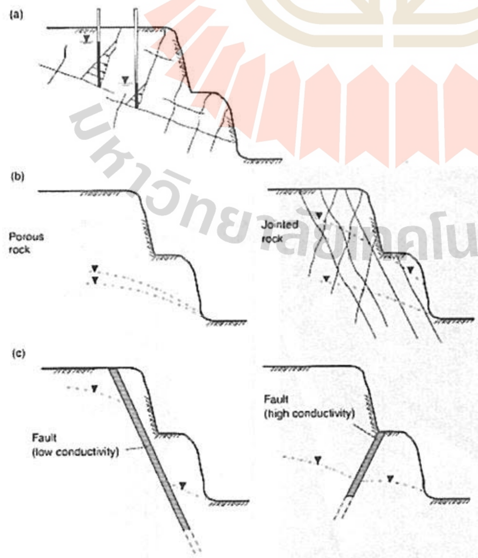


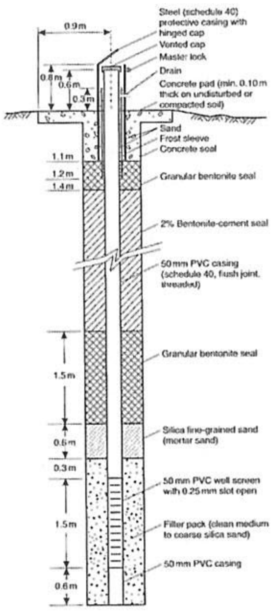
Figure 5.11 Relationship between geology and ground water in slopes: (a) variation in water pressure in joints related to persistence; (b) comparison of water tables in slopes excavated in porous and jointed rock; (c) faults as low conductivity ground water barrier, and high conductivity sub-surface drain (Patton and Deere, 1971).

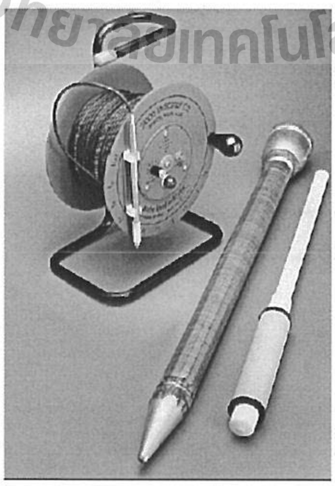
Measurement of Water Pressure

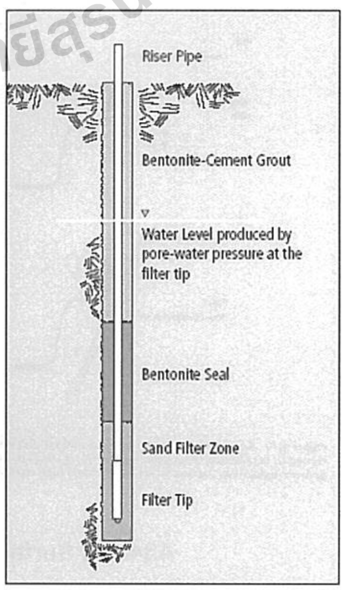
Types of Piezometer

- Open Piezometers (Observation Wells)
- Standpipe Piezometers
- Closed Hydraulic Piezometers
- Air Actuated Piezometers (Pneumatic Piezometers)
- Electrically Indicating Piezometers (Electronics Transducers)

Standpipe Piezometers







Water level in standpipe (Casagrande) piezometer is produced by pore-water pressure at the filter tip.

Air Actuated Piezometers (Pneumatic Piezometers)

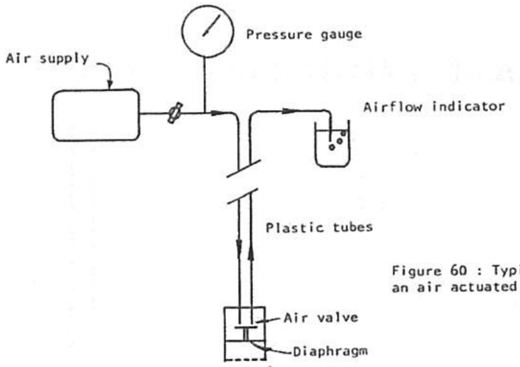
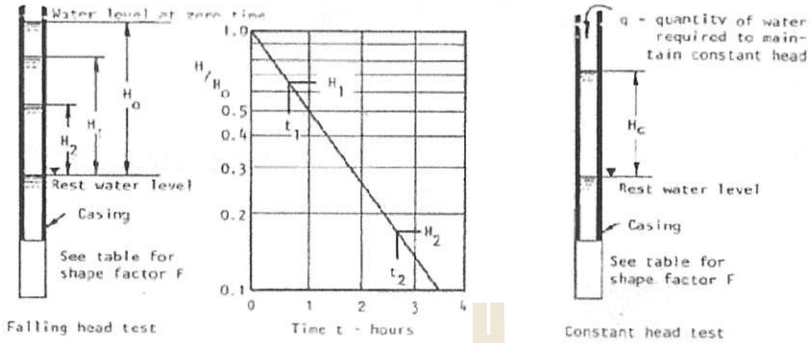


Figure 60 : Typical circuit for an air actuated piezometer.

Field Measurement of Hydraulic Conductivity

- Falling Head Tests
- Constant Head Tests
- Pumping Tests

Field Measurement of Hydraulic Conductivity



- **Falling Head Tests**

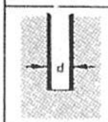
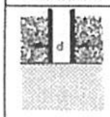
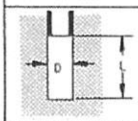
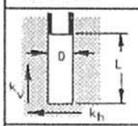
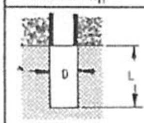
$$K = \frac{A}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$

- **Constant Head Tests**

$$K = \frac{q}{FH_c}$$

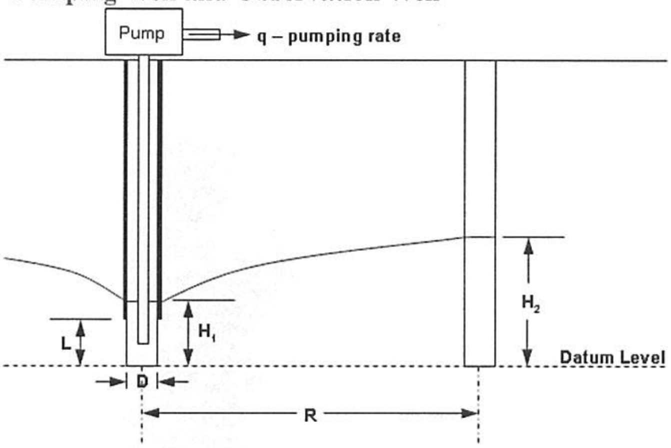
where A = cross section area of the water column.
F = shape factor
H_1, H_2 = water levels in the borehole measured from the rest water level, at times t1 and t2 respectively.
q = flow rate and
H_c = water level, measured from the rest water level, maintained during a constant head test.

Shape Factors

End conditions		Shape factor F
	Casing flush with end of borehole in soil or rock of uniform permeability. Inside diameter of casing is d cms.	F = 2.75d
	Casing flush with boundary between impermeable and permeable strata. Inside diameter of casing is d cms.	F = 2.0d
	Borehole extended a distance L beyond the end of the casing. Borehole diameter is D.	$F = \frac{2\pi L}{\log_e(2L/D)}$ for L > 4D
	Borehole extended a distance L beyond the end of the casing in a stratified soil or rock mass with different horizontal and vertical permeabilities.	For determination of k_h: $F = \frac{2\pi L}{\log_e(2m\ L/D)}$ where $m = (k_h/k_v)^{\frac{1}{2}}$, L > 4D
	Borehole extended a distance L beyond the end of the casing which is flush with an impermeable boundary.	$F = \frac{2\pi L}{\log_e(4L/D)}$ for L > 4D

Pumping Tests

- Pumping Well and Observation Well



$$K = \frac{q \ln(2^{R/D})}{2\pi L(H_1 - H_2)}$$

where q = pumping rate required to maintain a constant pressure in the test cavity

L = length of the test cavity

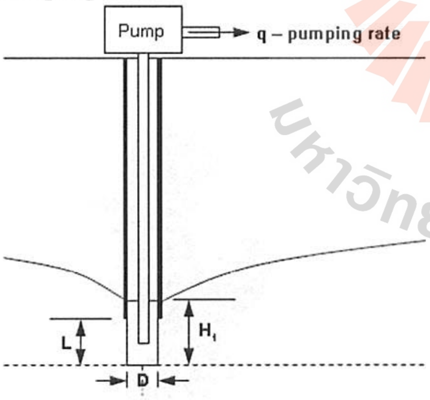
H_1 = total head in the test cavity

D = borehole diameter

H_2 = total head measured at a distance R from the borehole.

Pumping Tests

- Pumping Well



$$K = \frac{q \ln(2m^{L/D})}{2\pi L\, H_C}$$

where in this case, $m = (K/K_p)^{1/2}$

K = permeability at right angles to the borehole (quantity required)

K_p = permeability parallel to the borehole which,

if cross flow is neglected, is equal to the permeability of the intact rock

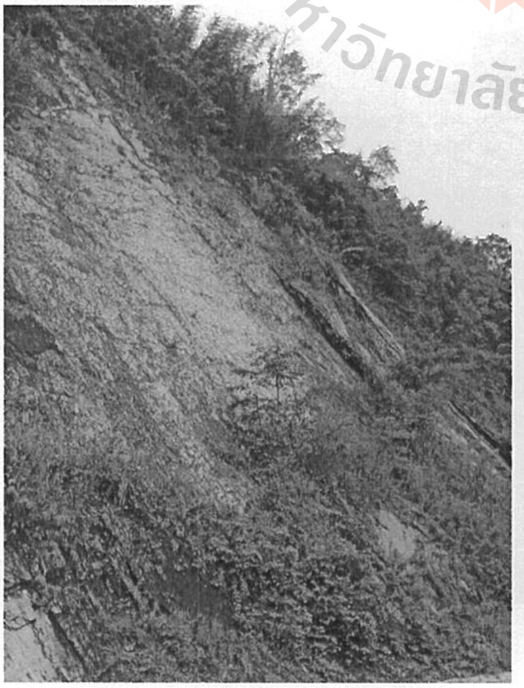
H_c = constant head above the original groundwater level in the borehole

434422 Surface Excavation & Design

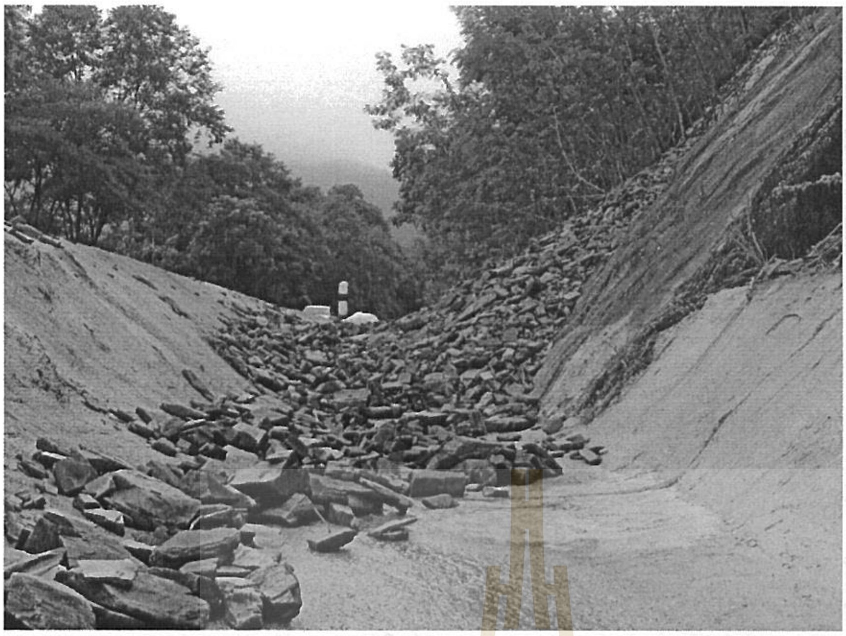
Topic 7 Plane Failure

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

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

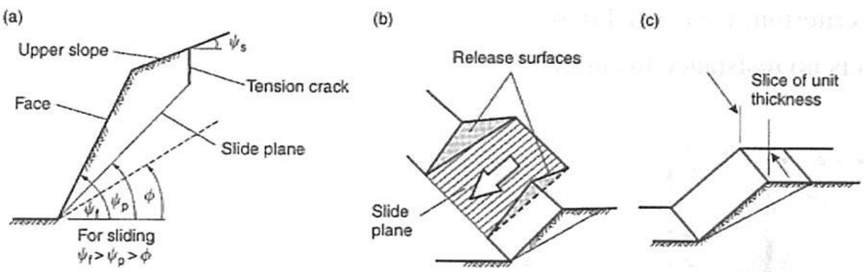


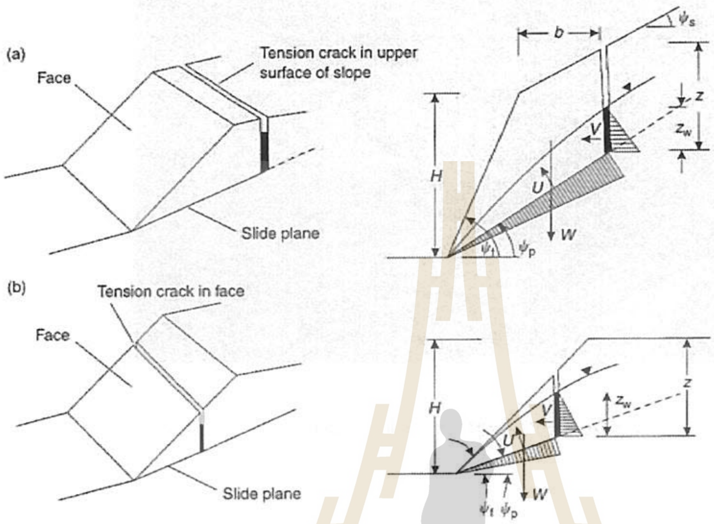
Figure 6.2 Geometry of slope exhibiting plane failure: (a) cross-section showing planes forming a plane failure; (b) release surfaces at ends of plane failure; (c) unit thickness slide used in stability analysis.

Plane Failure Analysis

The geometry of the slope is defined two cases:

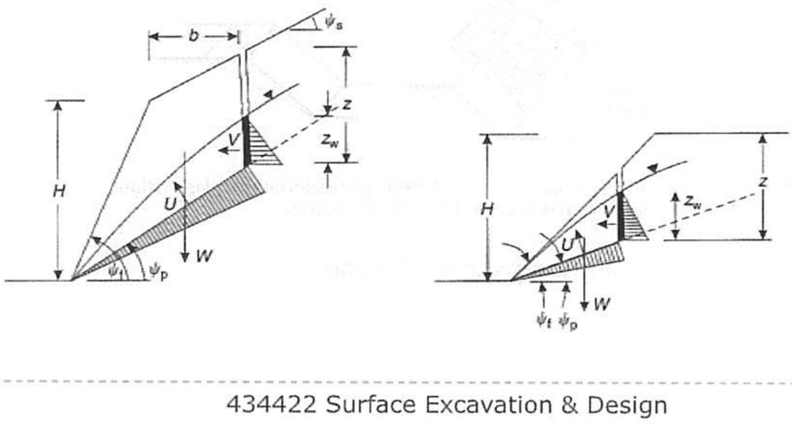
(a) A slope having a tension crack in its upper surface

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

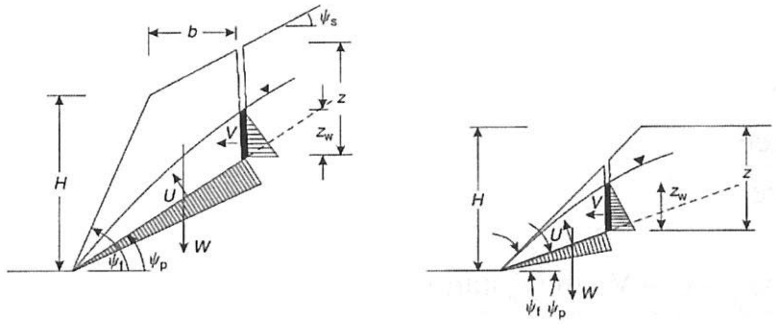


Assumptions Required for Analysis

- 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, $\tau = c + \sigma \tan \phi$
- Release surfaces is no resistance to sliding.

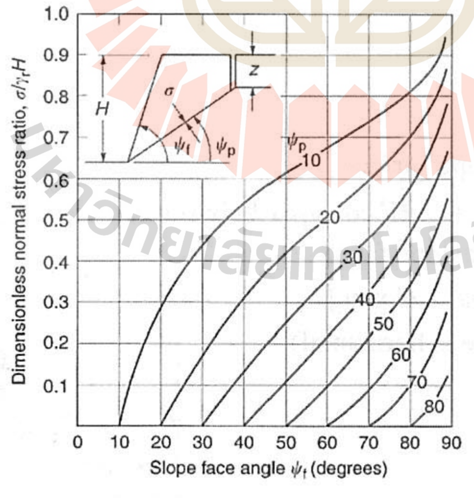


Symbols



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

Normal Stress acting on Slide Plane



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

where $z/H = 1-(\cot\psi_f \tan\psi_p)^{1/2}$, and $\psi_s = 0$

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

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

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

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

F.S. Calculations

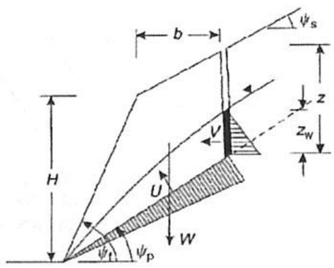
For the tension crack in the upper slope surface

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

(for ψ_s= dip angle of upper slope face)

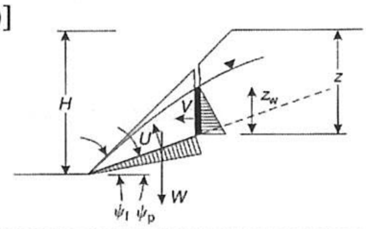
$W = \frac{1}{2}\gamma_r H^2 [(1 - (z/H)^2)\cot\psi_p - \text{Cot}\ \psi_f]$

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



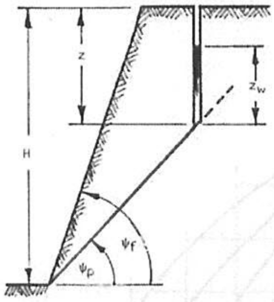
For the tension crack in the slope face

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



Simplify the Calculations

In case of $\psi_s = 0$, upper slope face is horizontal, following dimensionless form :



$$F.S. = \frac{(2c/\gamma_r H).P + \left(Q.\cot\psi_p - R(P+S)\right)\tan\phi}{Q + R.S\cot\psi_p}$$

where $P = (1 - z/H).\ \text{cosec}\ \psi_p$

(a) tension crack is in the upper slope surface:

$$Q = [(1 - z/H)^2)\cot\psi_p - \cot\psi_f\,]\sin\psi_p$$

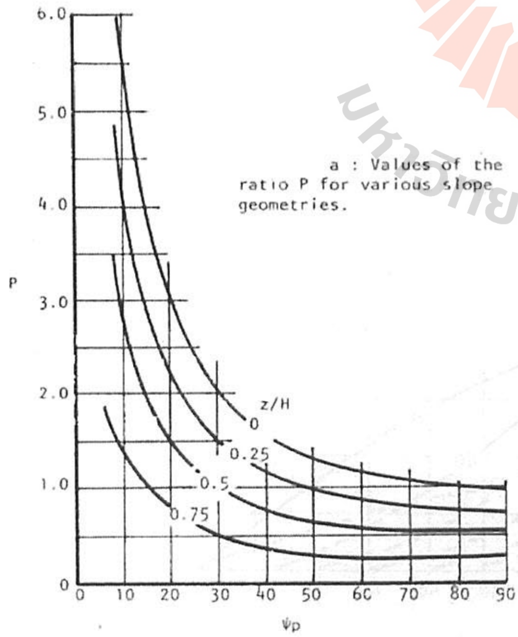
(b) tension crack is in the slope face:

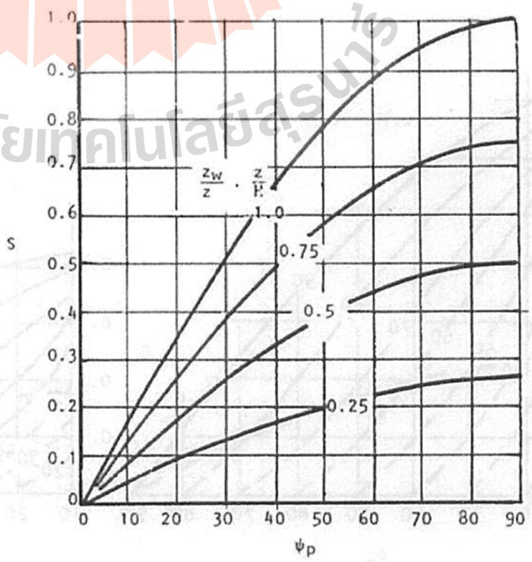
$$Q = [(1 - z/H)^2\cos\psi_p\,(\cot\psi_p\,.\tan\psi_f - 1)]$$

$$R = \frac{\gamma_w}{\gamma_r}.\frac{z_w}{z}.\frac{z}{H}$$

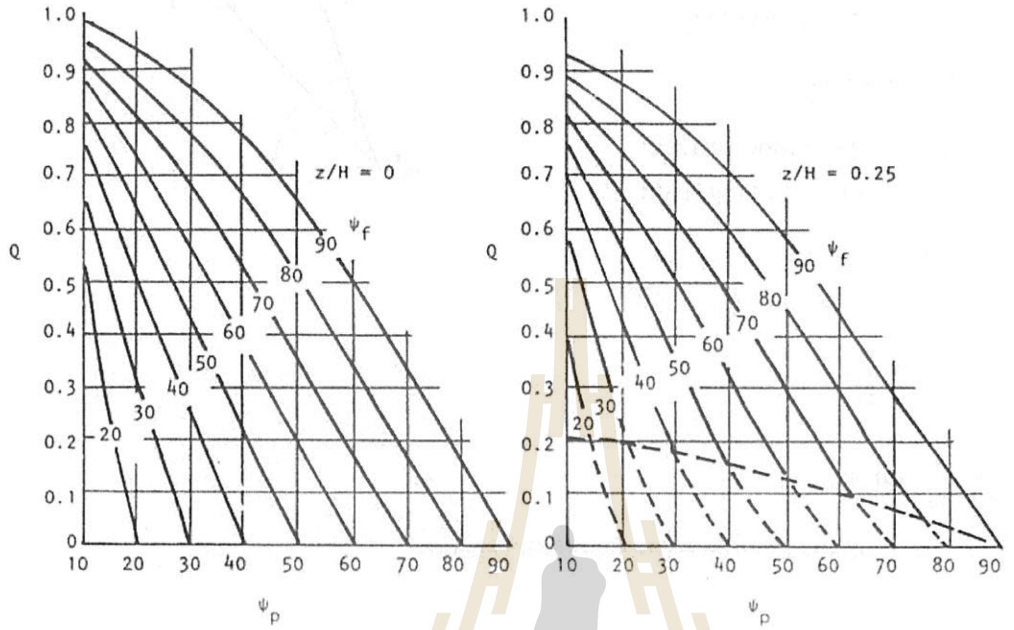
$$S = \frac{z_w}{z}.\frac{z}{H}\sin\psi_p$$

P - chart & S - chart

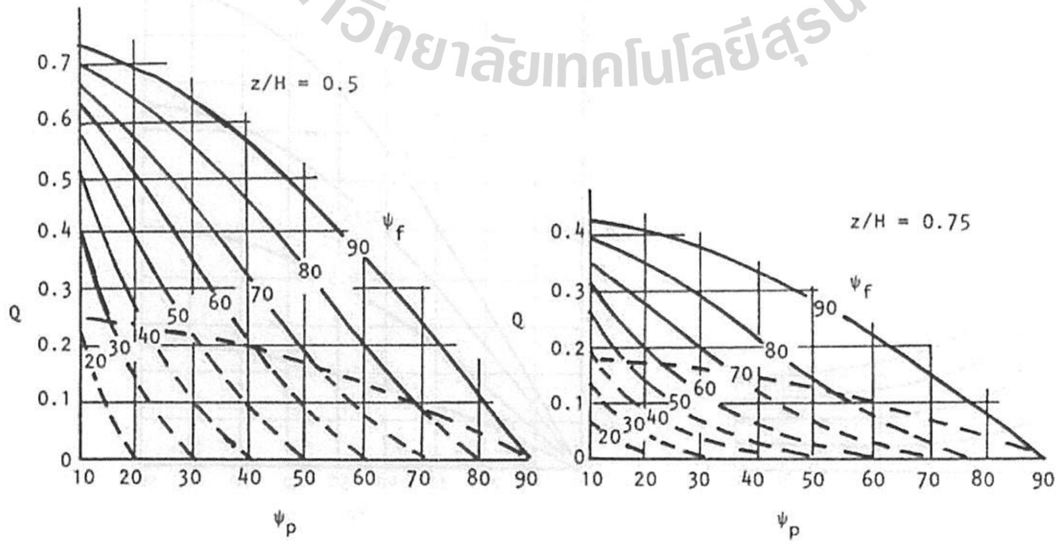




Q - chart



Q - chart



Influence of Groundwater

Dry slopes (forces V and U are both zero)

$$F.S. = \frac{c.A}{W.\sin\ \psi_p} + \cot\ \psi_p.\tan\phi$$

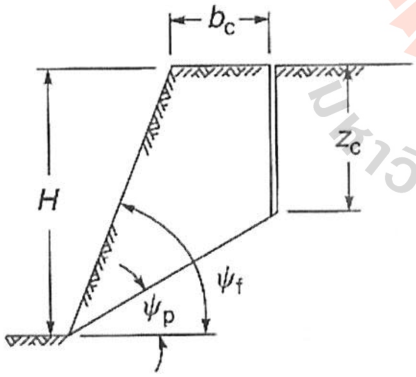
$$F.S. = \frac{2c}{\gamma_r H}.\frac{P}{W} + \cot\ \psi_p.\tan\phi$$

Water in tension crack only (uplift force U = 0)

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

$$F.S. = \frac{2c/\gamma_r H.P + (Q.\cot\psi_p - RS)\tan\phi}{Q + RS.\cot\psi_p}$$

Critical Tension Crack Depth and Location



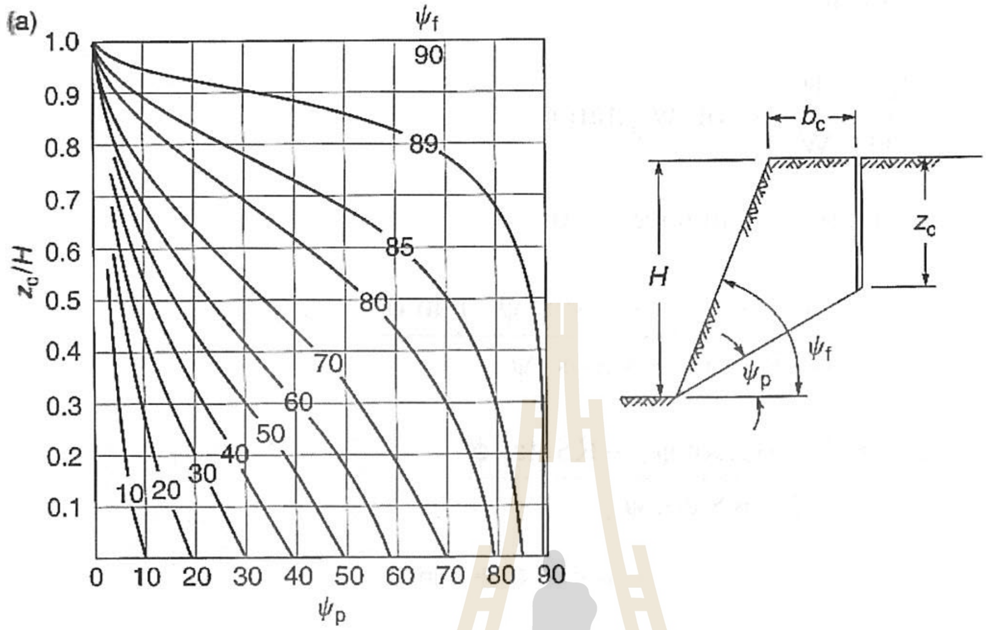
critical tension crack depth (z_c)

$$z_c/H = 1 - \sqrt{\mathrm{Cot}\ \psi_f\ .\ \mathrm{Tan}\ \psi_p}$$

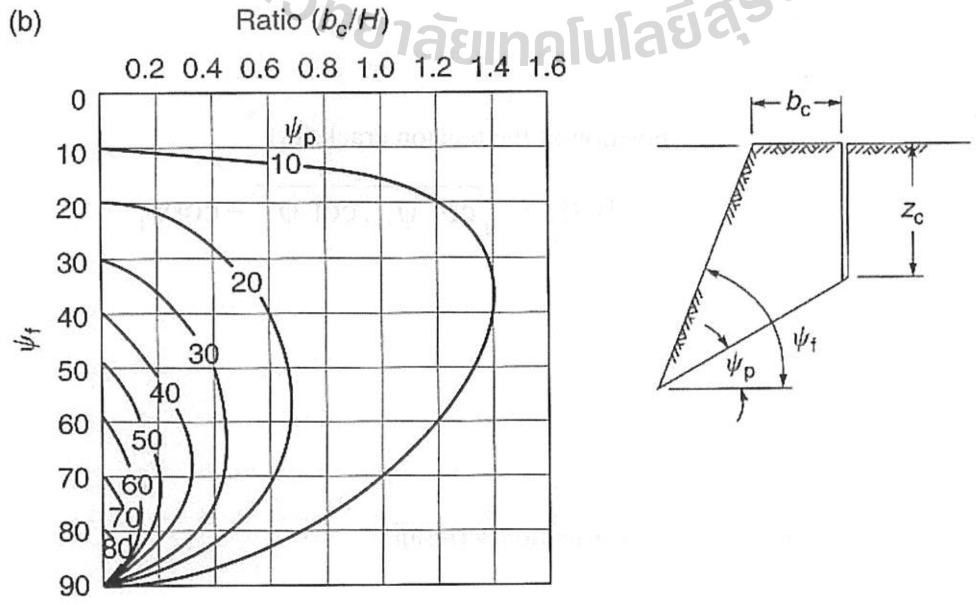
position of the tension crack (b_c)

$$b_c/H = \sqrt{\cot\ \psi_f\ .\cot\ \psi_p} - \cot\psi_f$$

Critical Tension Crack Depth (dry slope)



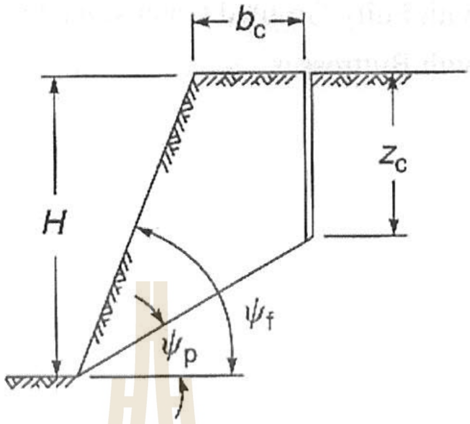
Critical Tension Crack Location (dry slope)



Critical Slide Plane Inclination

For dry slopes this gives the critical failure plane inclination ψ_{pc} as

$$\psi_{pc} = \tfrac{1}{2}(\psi_f + \phi)$$



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

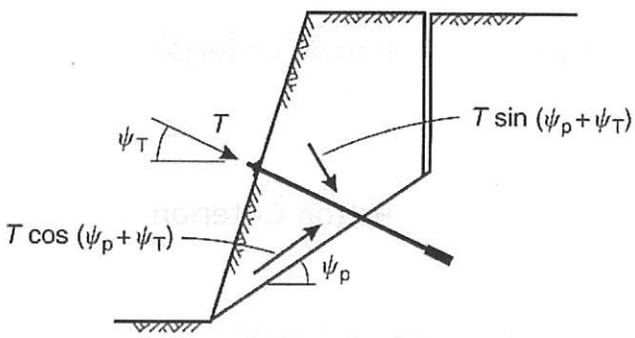
$$F.S. = \frac{\tan(\phi + i)}{\tan \psi_p}$$ Patton Criterion

Reinforcement of a Slope

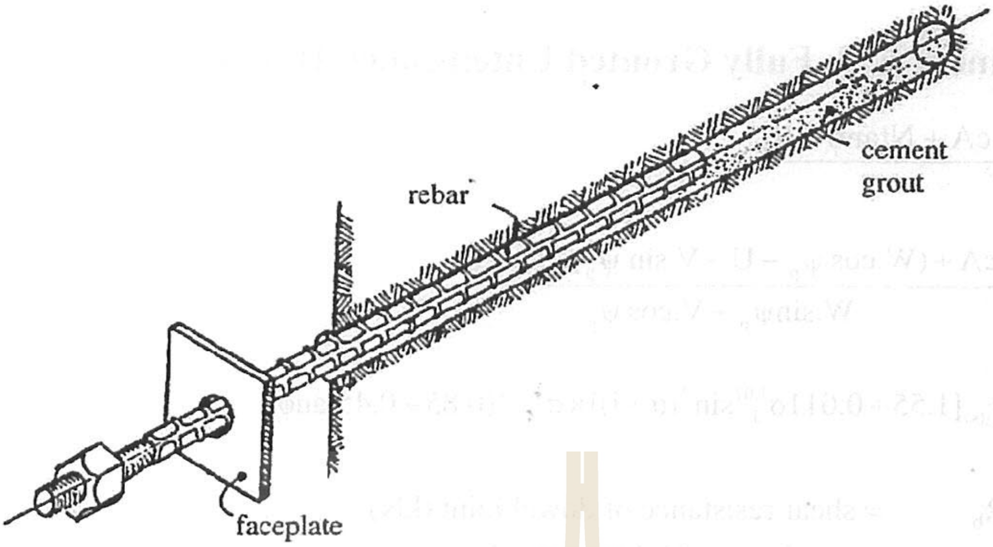
- -Reinforcement with Tensioned Anchors
- -Reinforcement with Fully Grouted Untensioned Dowels
- -Reinforcement with Buttresses

Reinforcement with Tensioned Anchors

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



Reinforcement with Fully Grouted Untensioned Dowels



Reinforcement with Fully Grouted Untensioned Dowels

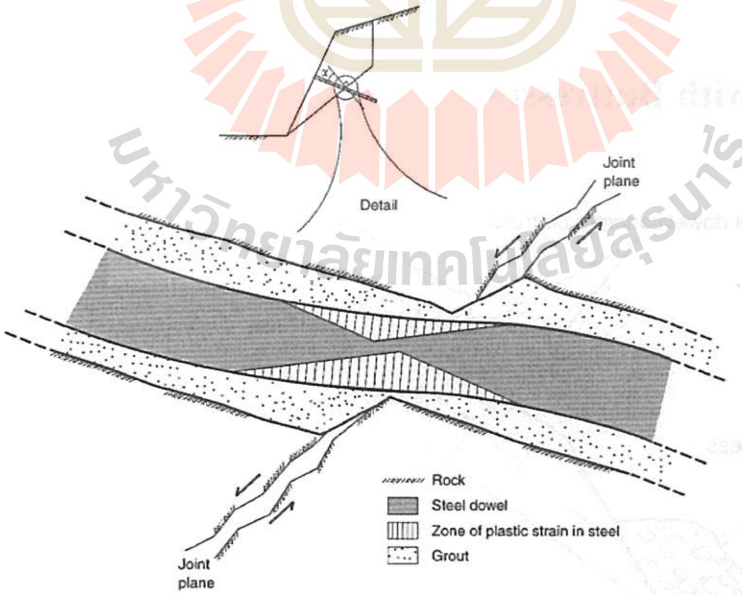


Figure 6.9 Strain in fully grouted steel dowel due to shear movement along joint (modified from Spang and Egger (1990)).

Reinforcement with Fully Grouted Untensioned Dowels

$$F.S. = \frac{cA + N\tan\phi + R_b}{S}$$

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

$$R_b = \sigma_{t(s)}[1.55 + 0.011\sigma_{ci}^{1.07}\sin^2(\alpha + i)] \times \sigma_{ci}^{-0.14}(0.85 + 0.45\tan\phi)$$

R_b	= shear resistance of dowel joint (kN)
i	= roughness of joint (asperities)
α	= dowel inclination (about b/w 30-45 degrees)
σ_{ci}	= compressive strength of rock and grout (MPa)
$\sigma_{t(s)}$	= tensile strength of steel bar (kN)

by Spang and Egger (1990)

Reinforcement with Buttresses

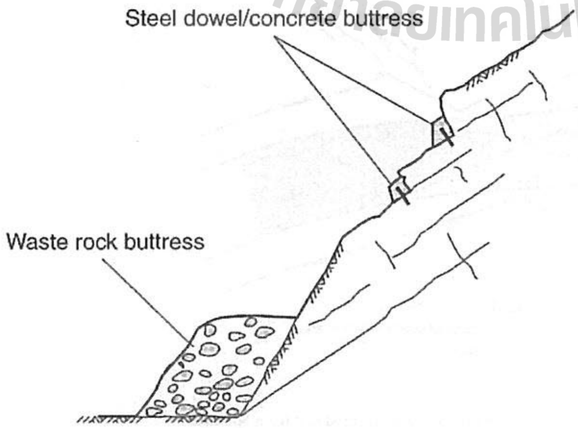
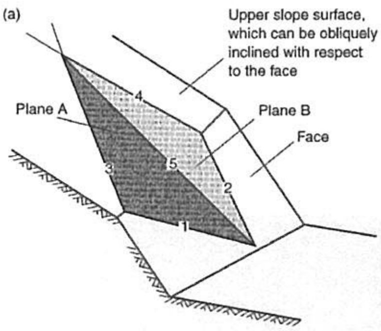


Figure 6.10 Reinforcement of slope with buttresses.



434422 Surface Excavation & Design

Topic 8 Wedge Failure

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

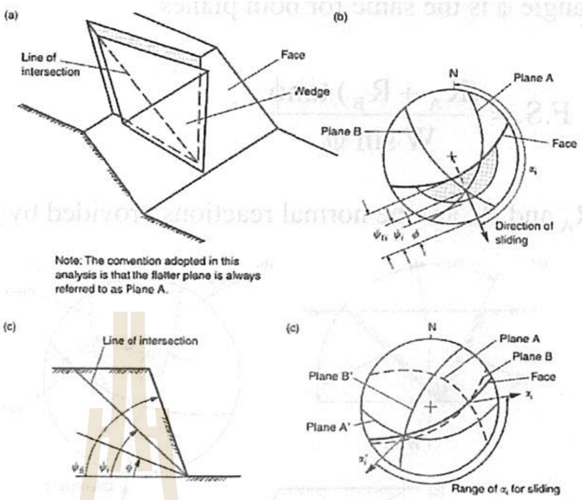
Wedge Failure

Wedge Failure

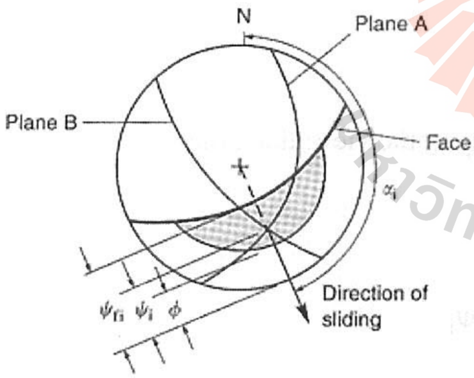
Wedge Failure

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



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

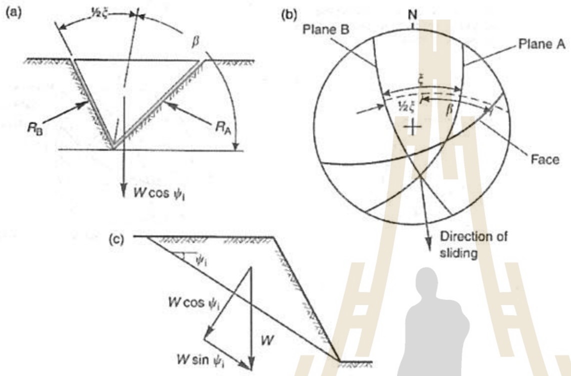
$$\psi_i = \tan\psi_A \cos(\alpha_A - \alpha_i) = \tan\psi_B \cos(\alpha_B - \alpha_i)$$

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 :

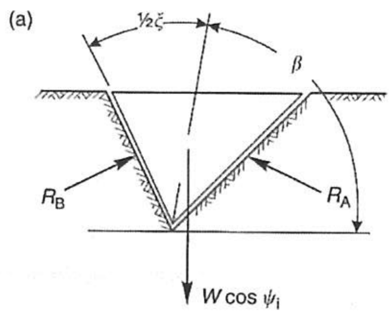
$$R_A \sin(\beta - ½\xi) = R_B \sin(\beta - ½\xi)$$
$$R_A \cos(\beta - ½\xi) + R_B \cos(\beta + ½\xi) = W \cos \psi_i$$

- Solving for R_A and R_B and adding :

$$R_A + R_B = \frac{W.\cos\psi_i.\sin\beta}{\sin\frac{1}{2}\xi}$$

- Hence :

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



Wedge Analysis including Cohesion, Friction and Water Pressure

The numbering used throughout this book is as follows:

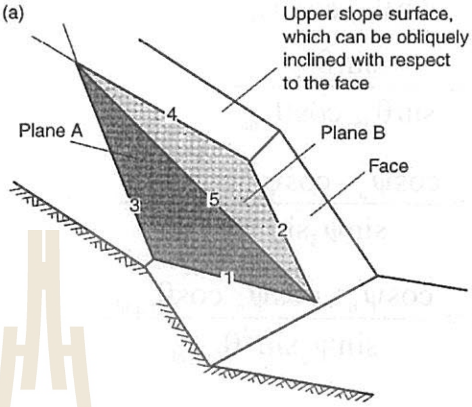
1 – Intersection of plane A with the slope face

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

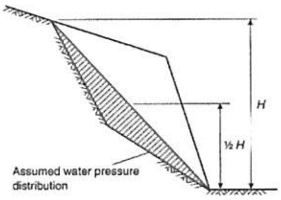
ϕ_A and ϕ_B = angles of friction on planes A and B

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 :

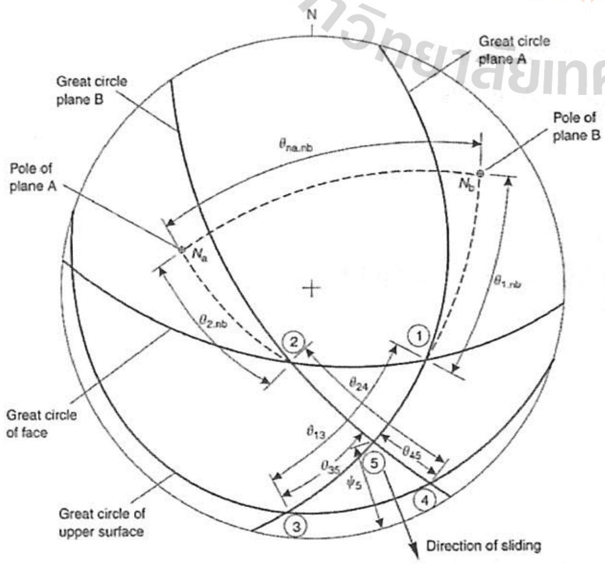
$$X = \frac{\sin\theta_{24}}{\sin\theta_{45}\ \cos\theta_{2.na}}$$

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

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

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

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

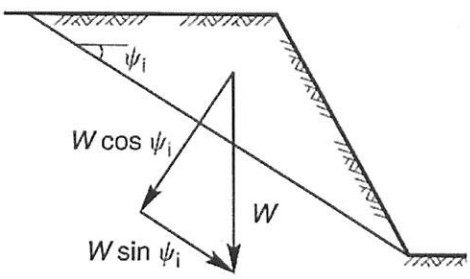
Analysis of Wedge Failure

- In other words:

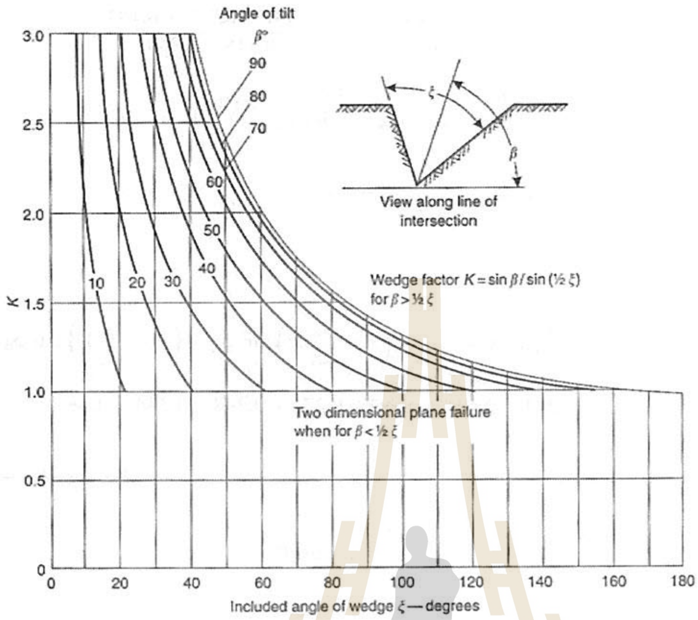
$$\mathbf{F.S._w = \mathit{K}\, F.S._p}$$

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.



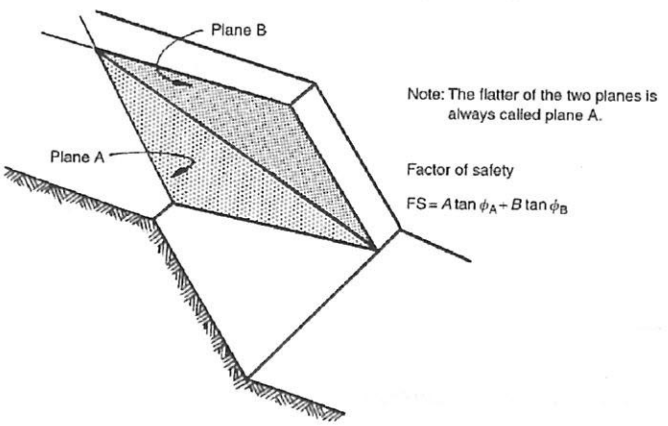
Wedge Factor, K



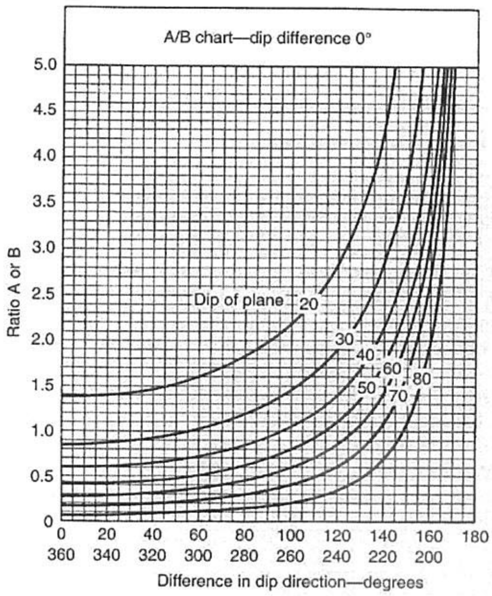
Wedge Stability Charts for Friction Only

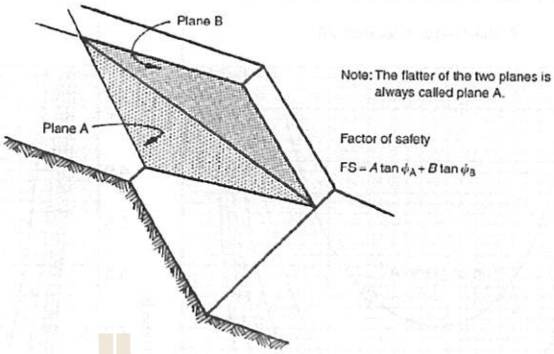
If the cohesive strength of the planes A and B is zero and the slope is fully drained,

$$\text{F.S.} = A \tan \phi_A + B \tan \phi_B$$

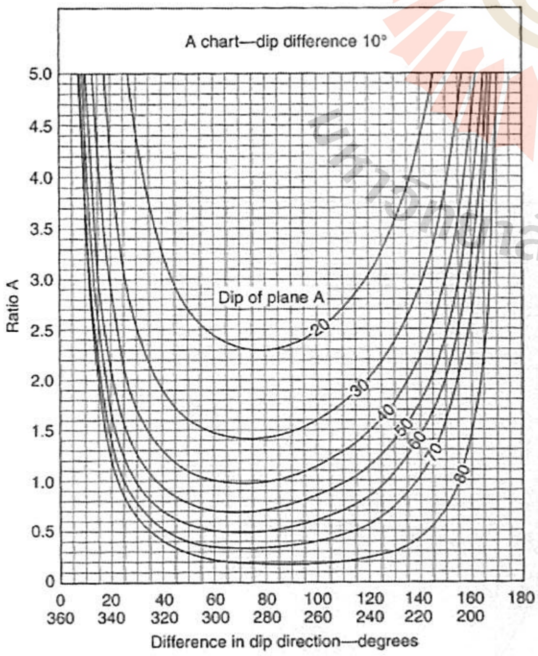


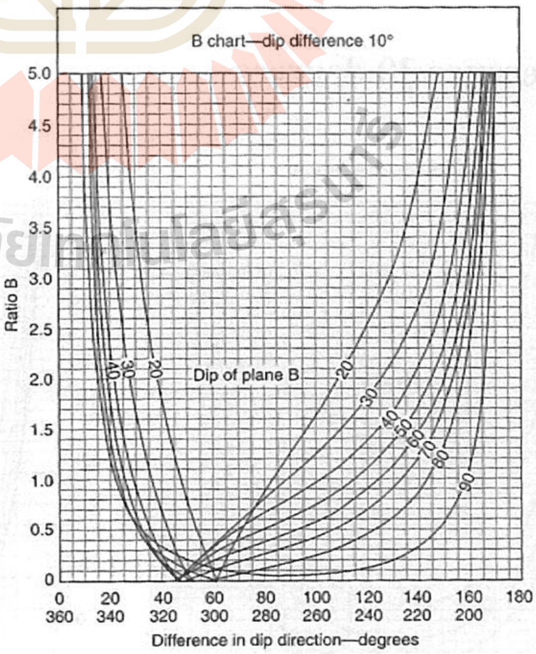
Dip Difference 0 degree



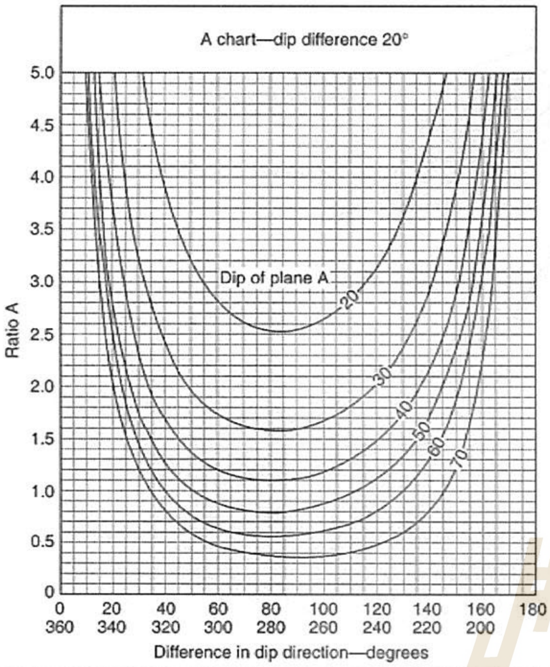


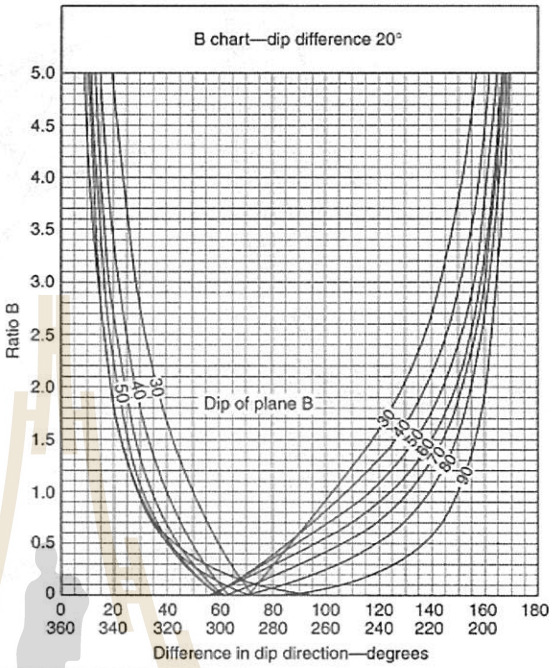
Dip Difference 10 degrees



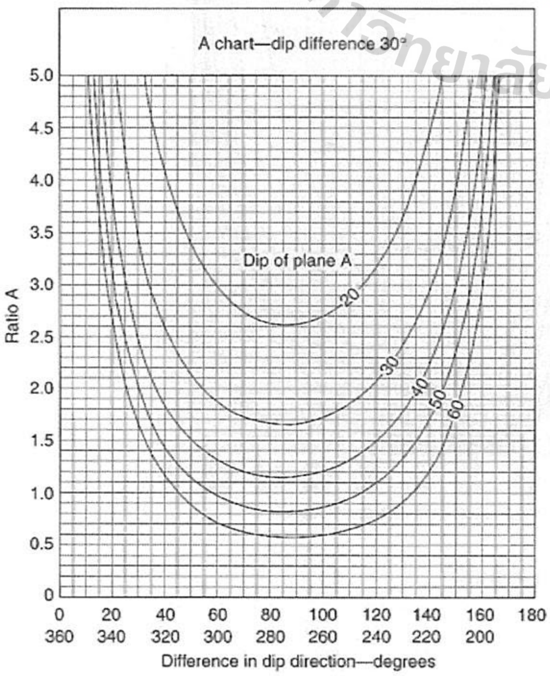


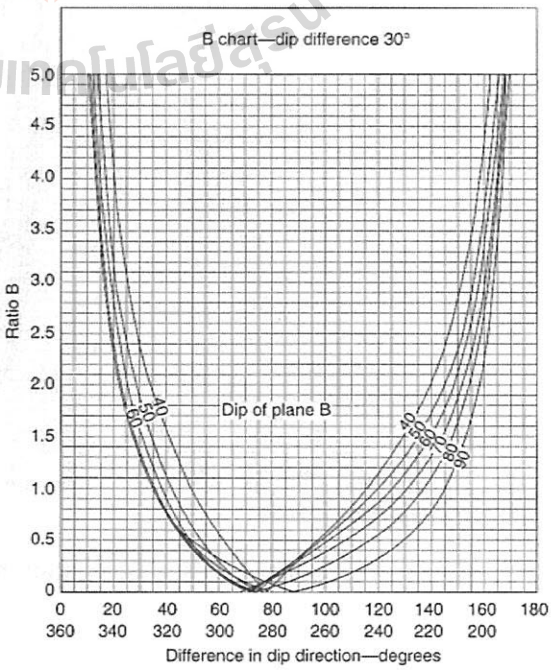
Dip Difference 20 degrees



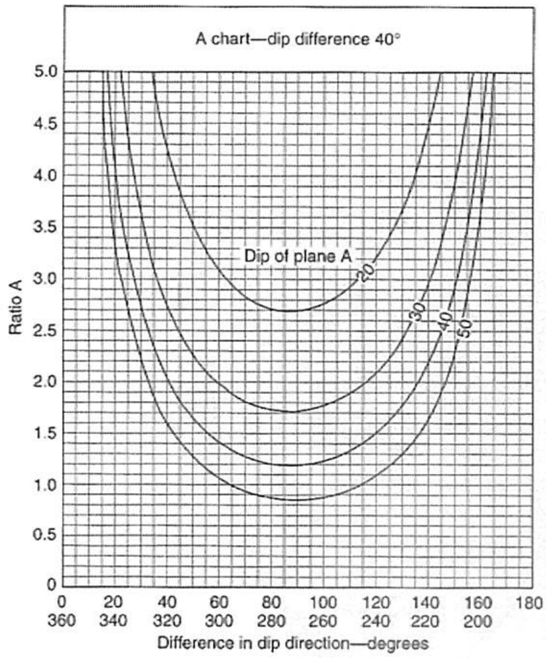


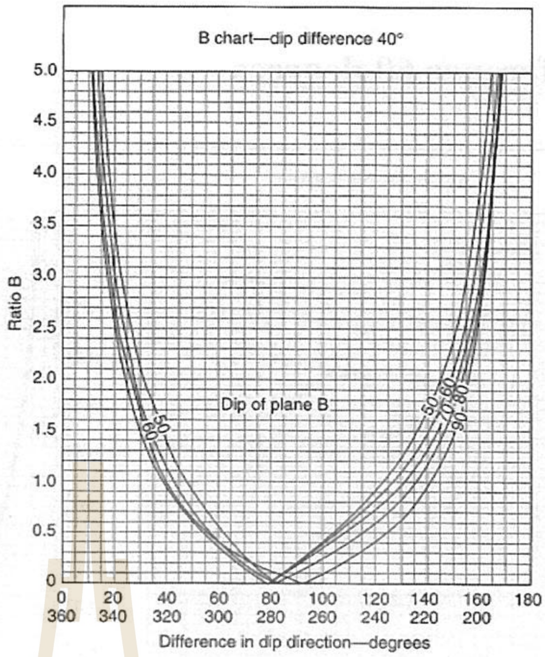
Dip Difference 30 degrees



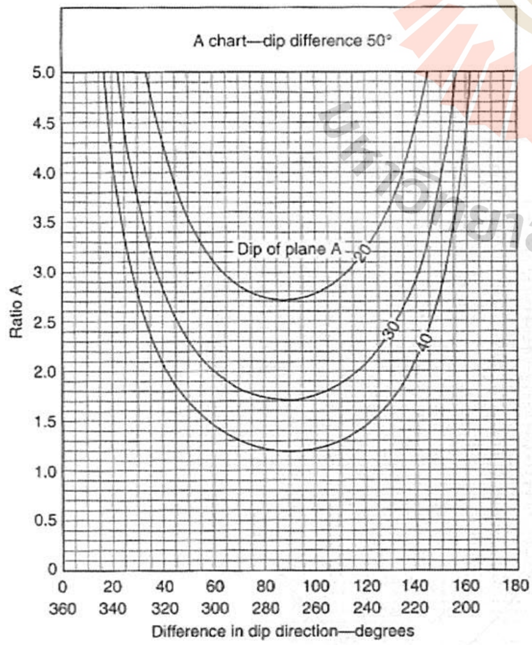


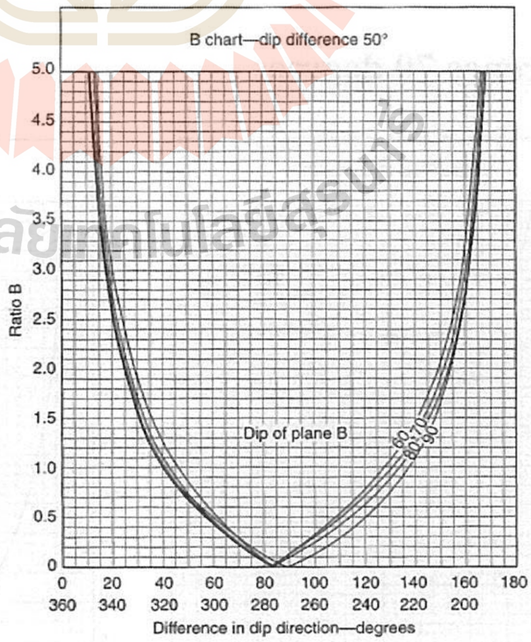
Dip Difference 40 degrees



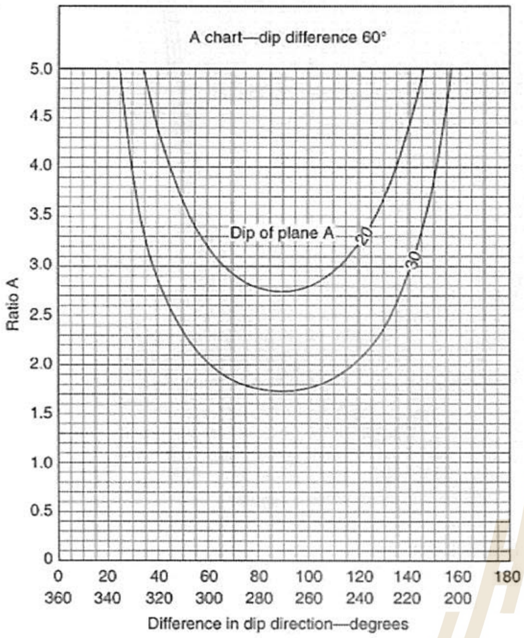


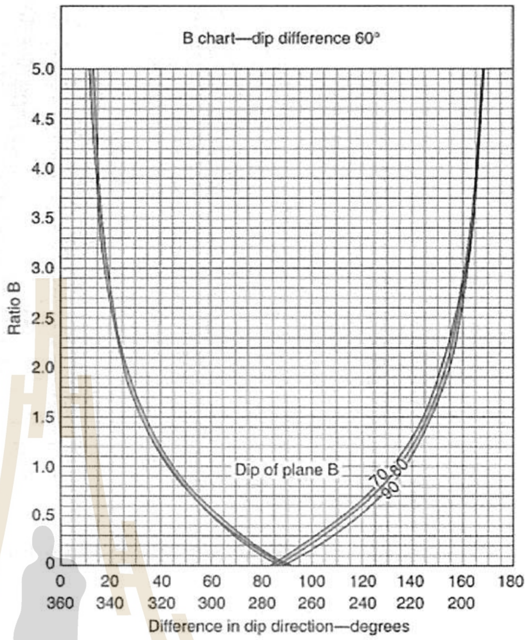
Dip Difference 50 degrees



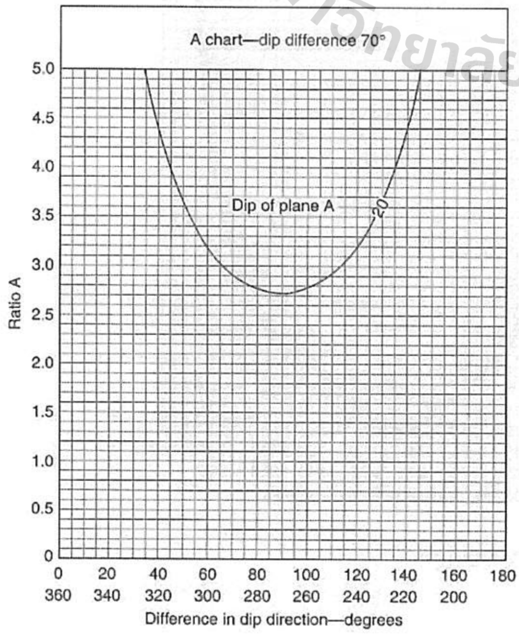


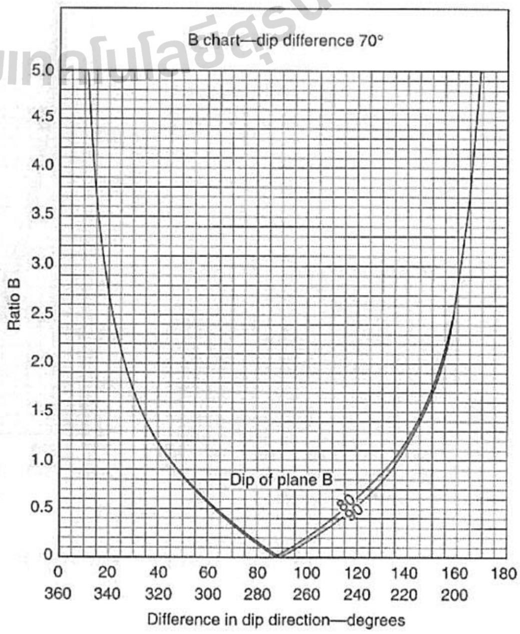
Dip Difference 60 degrees





Dip Difference 70 degrees

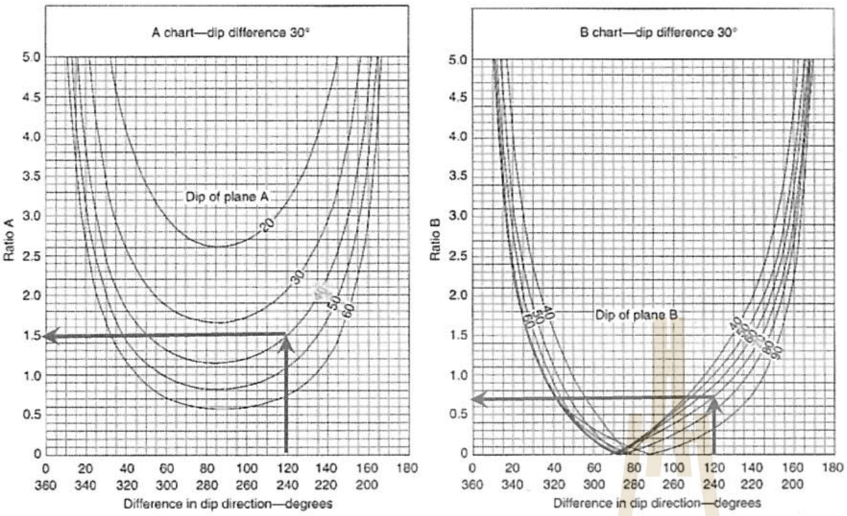




Example

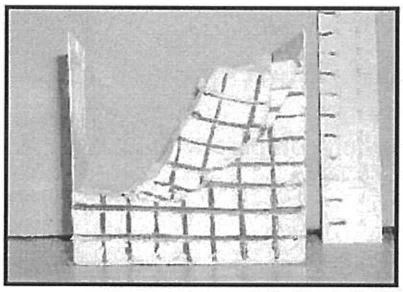
- In order to illustrate the use of these charts, consider the following example:

	dip°	dip direction°	friction angle°
Plane A	40	165	35
Plane B	70	285	20



A = 1.5
B = 0.7

$$F.S. = A \tan\phi_A + B \tan\phi_B$$
$$= 1.5 \tan 35 + 0.7 \tan 20$$
$$= 1.30$$



434422 Surface Excavation & Design
Topic 9 Circular Failure

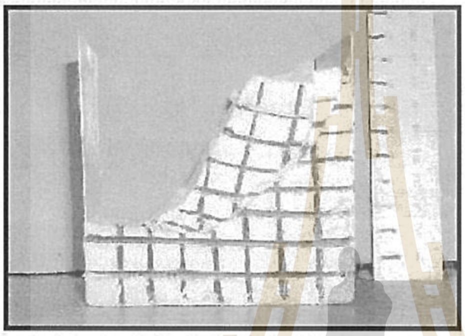
Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

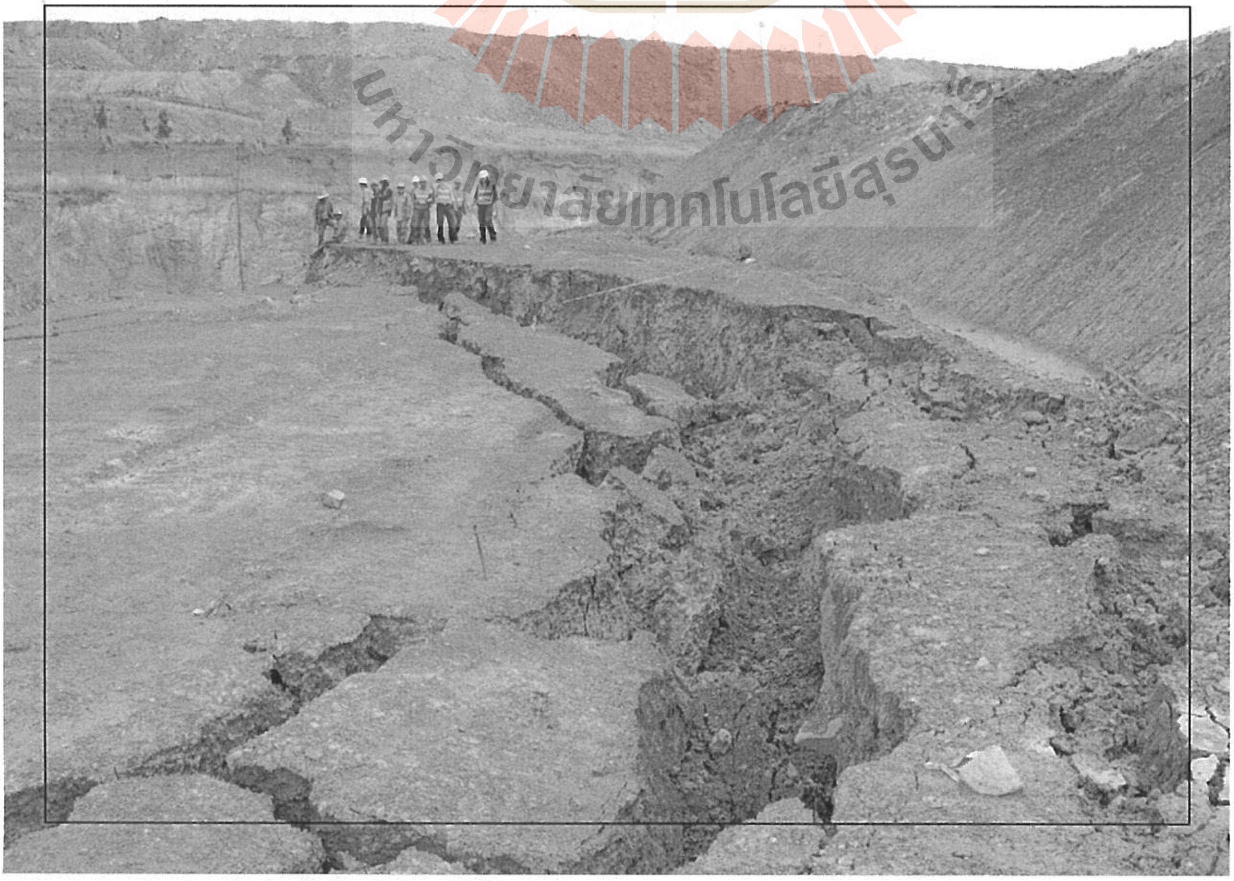
Circular Failure

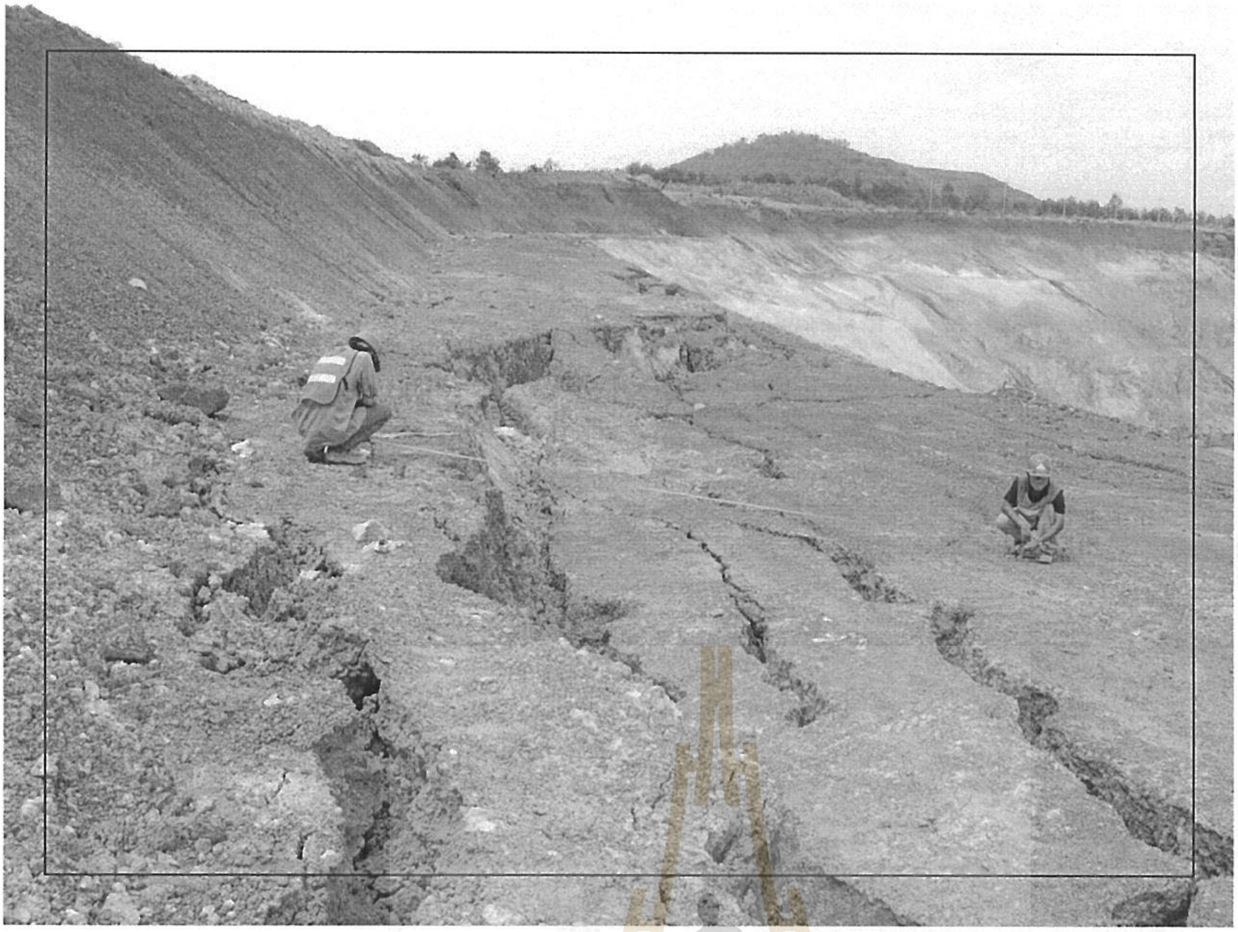
Conditions for Circular Failure and Methods of Analysis

- 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







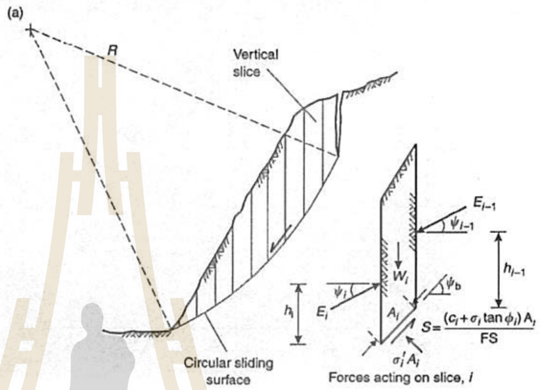
Stability Analysis Procedure

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 ($\tau = c + \sigma \,.\, \text{Tan}\, \phi$)
- 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

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

and rearranging this equation, we get

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

Groundwater Flow Assumptions

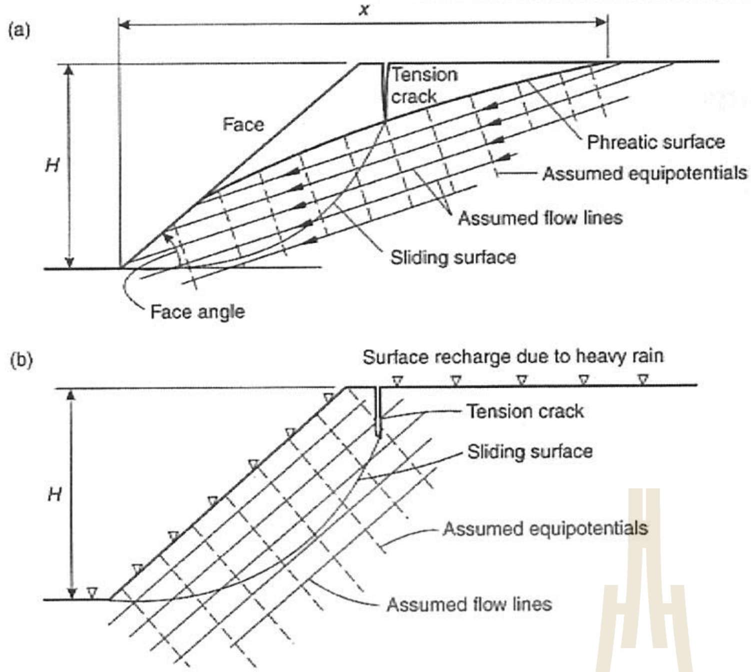
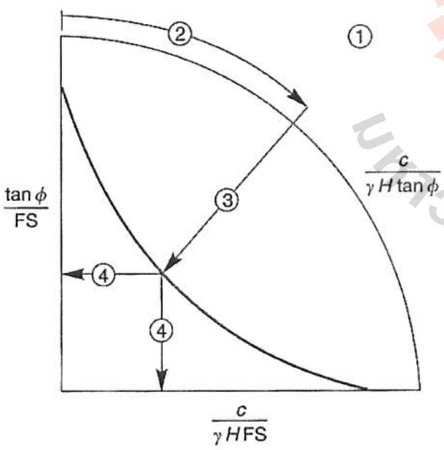


Figure 8.3 Definition of ground water flow patterns used in circular failure analysis of slopes in weak and closely fractured rock: (a) ground water flow pattern under steady state drawdown conditions where the phreatic surface coincides with the ground surface at a distance *x* behind the toe of the 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

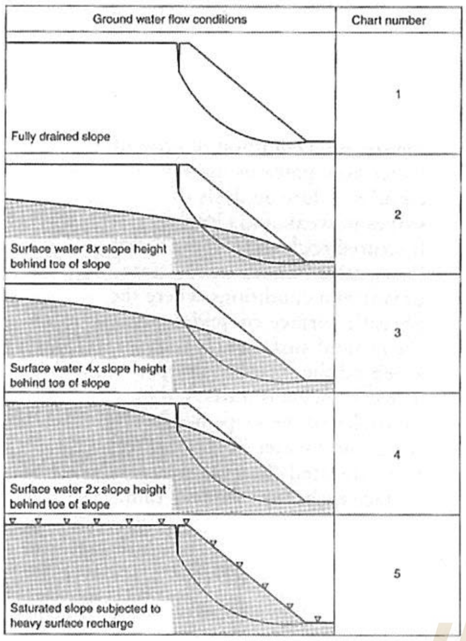
$$\frac{c}{\gamma H \tan\phi}$$

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 $\tan\phi/FS$ or $c/\gamma HFS$, depending upon which is more convenient, and calculate the F.S.

Groundwater Flow Conditions



Circular Failure Charts No.1

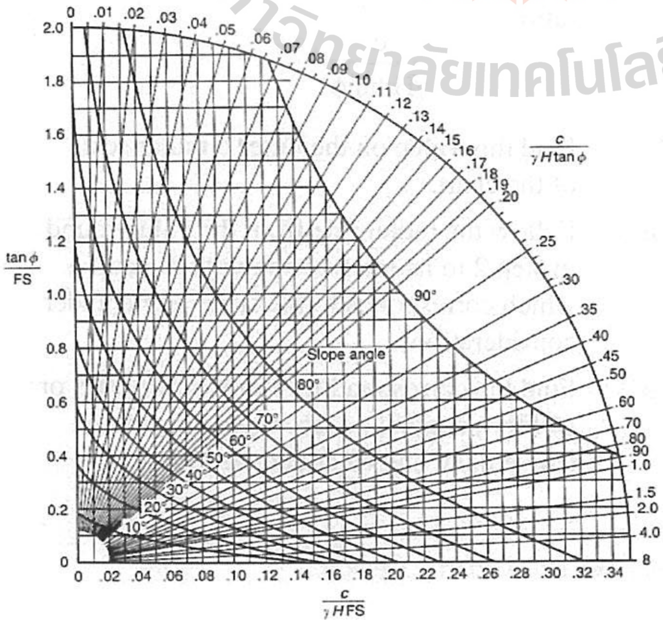


Figure 8.6 Circular failure chart number 1—fully drained slope.

Circular Failure Charts No.2

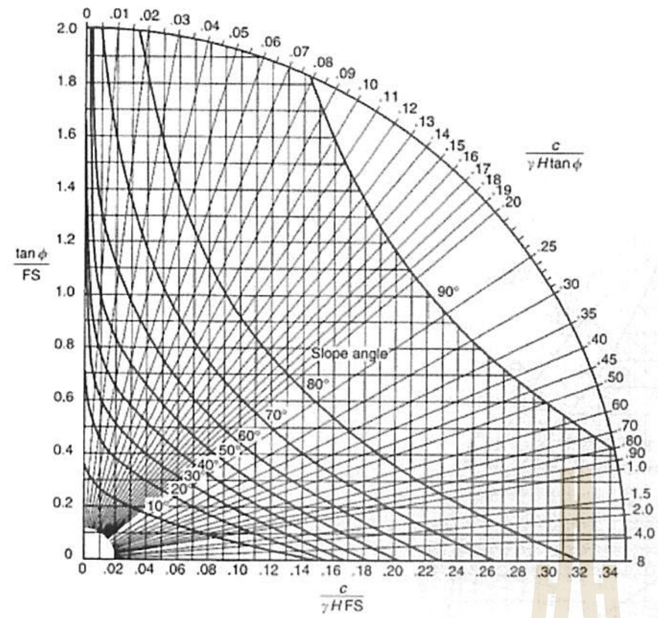


Figure 8.7 Circular failure chart number 2—ground water condition 2 (Figure 8.5).

Circular Failure Charts No.3

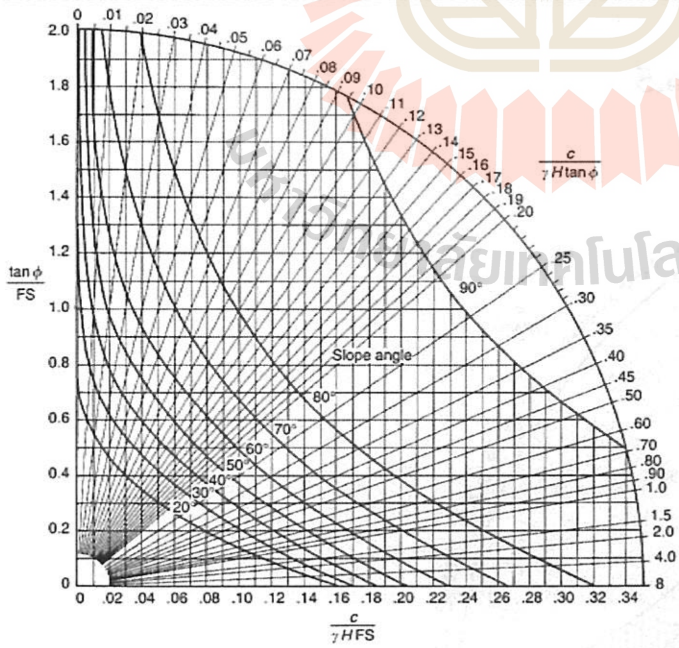


Figure 8.8 Circular failure chart number 3—ground water condition 3 (Figure 8.4).

Circular Failure Charts No.4

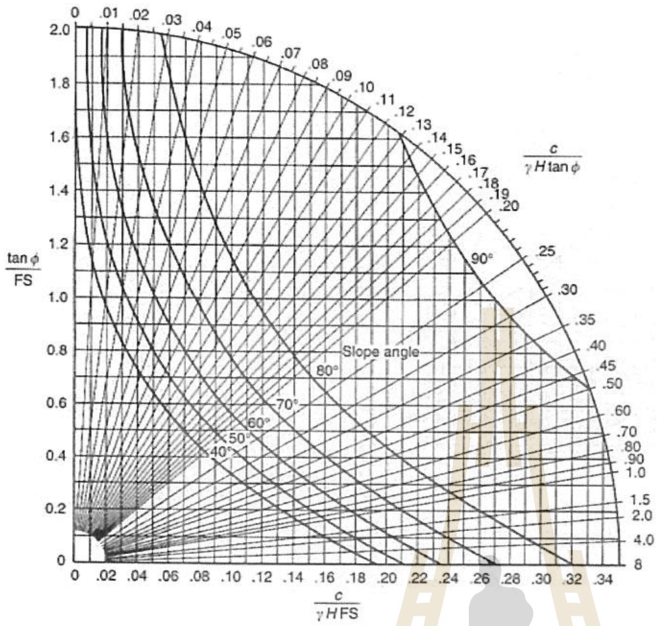


Figure 8.9 Circular failure chart number 4—ground water condition 4 (Figure 8.4).

Circular Failure Charts No.5

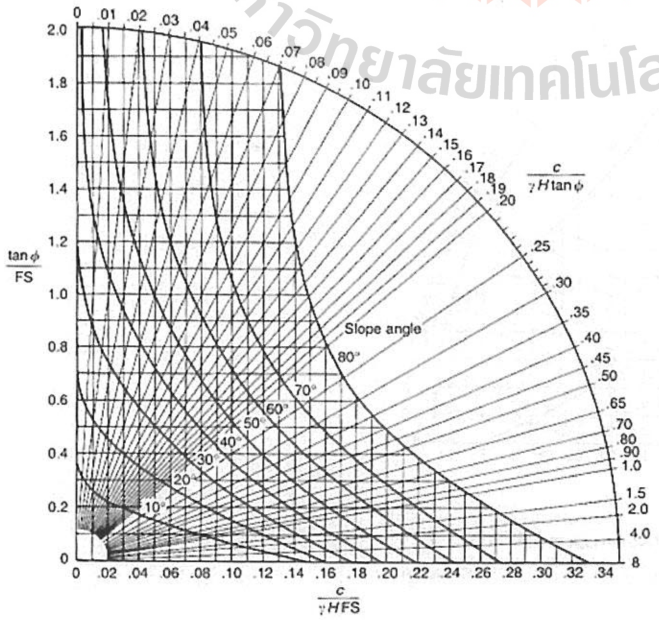
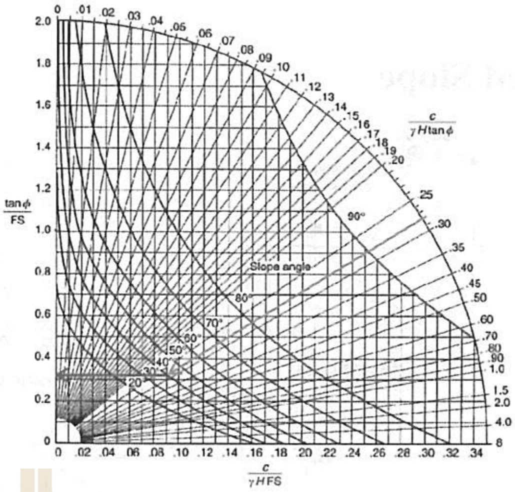


Figure 8.10 Circular failure chart number 5—fully saturated slope.

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

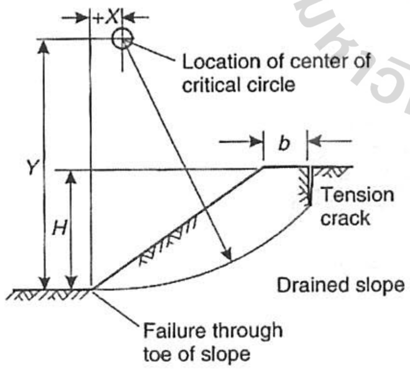
$$\frac{c}{\gamma H \tan\phi} = 0.28$$

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

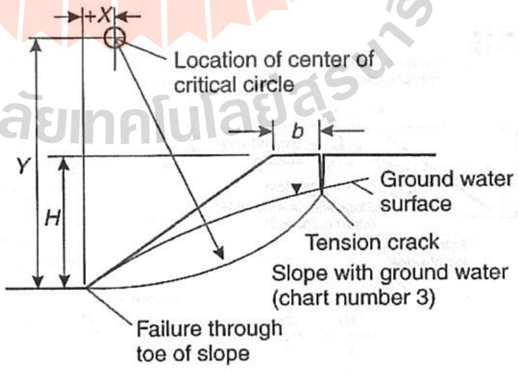
Step 4 : Calculate the F.S.
F.S. = (0.32/tan 30) = 1.80

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).

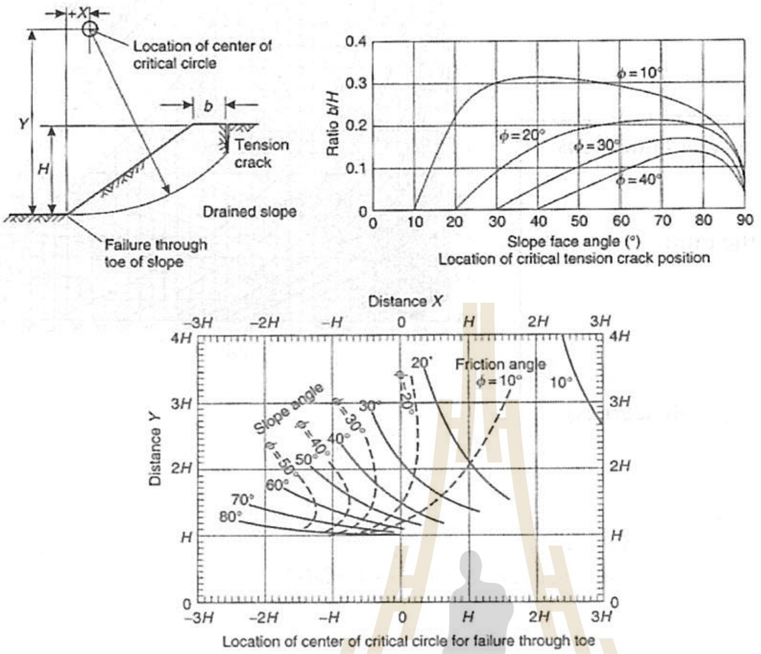


Drained Slope

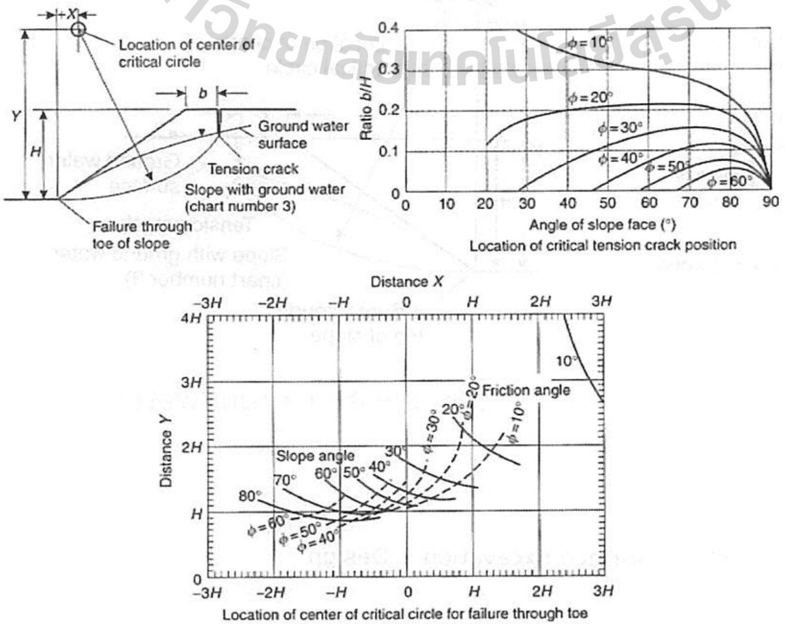


Slope with Groundwater

Drained Slope

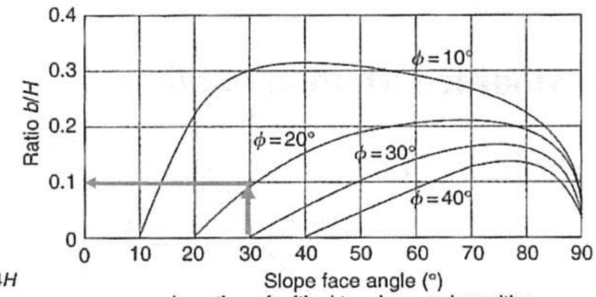


Slope with Groundwater (chart no.3)

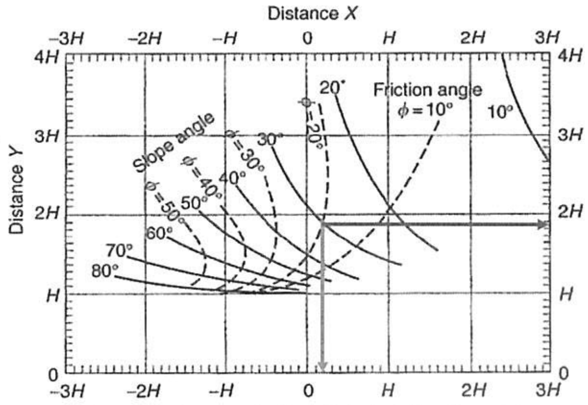


Example of Find the

Given:
Drained Slope
Slope height, H = 15.2 m.
Slope angle, ψ_f = 30 degrees
Friction angle, ϕ = 20 degrees

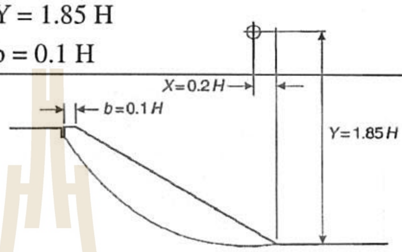


Location of critical tension crack position

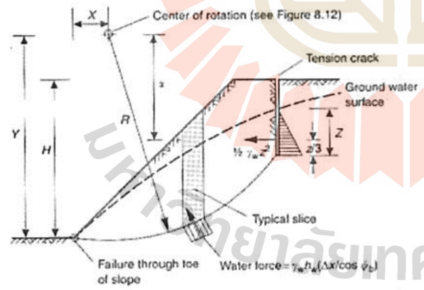


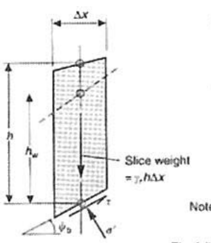
Location of center of critical circle for failure through toe

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



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





Factor of safety:

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

where

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

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

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

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

Note: angle ψ_b is negative when sliding uphill

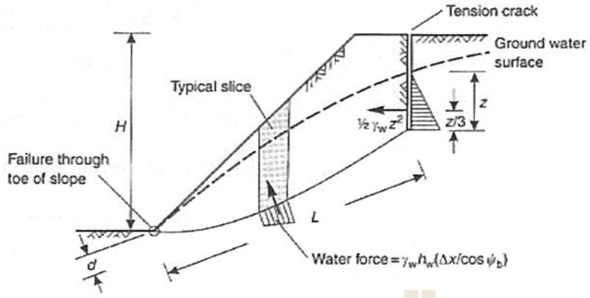
The following conditions must be satisfied for each slice:

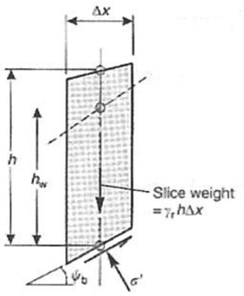
$$(1)\ \sigma' = \frac{\gamma_r h - \gamma_w h_w - c(\tan \psi_b / FS)}{1 + Y/FS} \quad (8.8)$$

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

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:

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

where

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

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

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

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

Note: angle ψ_b is negative when sliding uphill

Approximate correction factor f_0

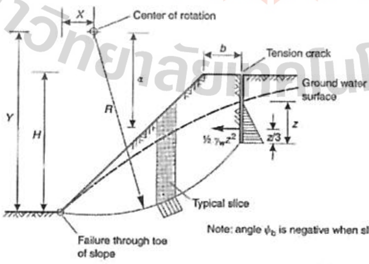
$$f_0 = 1 + K(d/L - 1.4(d/L)^2) \quad (8.15)$$

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

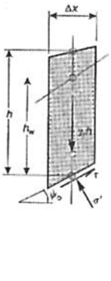
$c' > 0$, $\phi' > 0$; $K = 0.50$

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)



Note: angle ψ_b is negative when sliding uphill



Factor of safety:

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

where

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

and

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

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

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

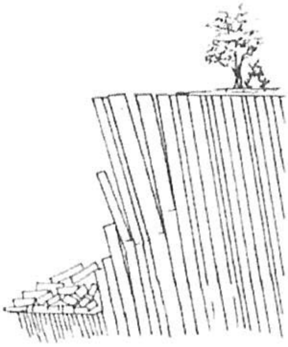
where $= \sigma_{3n} = \sigma'_{3max}/\sigma_{ci}$

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.



434422 Surface Excavation & Design
Topic 10 Toppling Failure

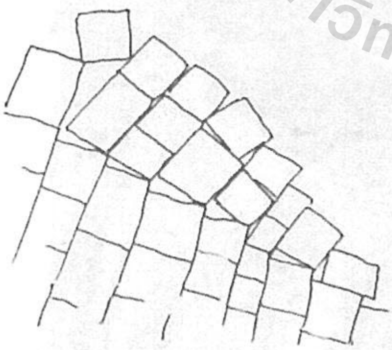
Prachya Tepnarong, Ph.D.
prachya@sut.ac.th

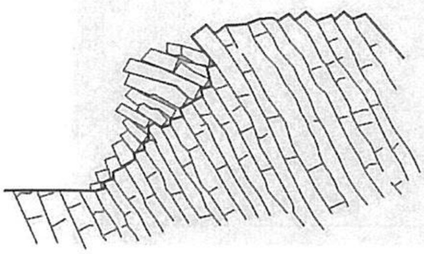
Type of Toppling Failure

Goodman and Bray (1976)

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

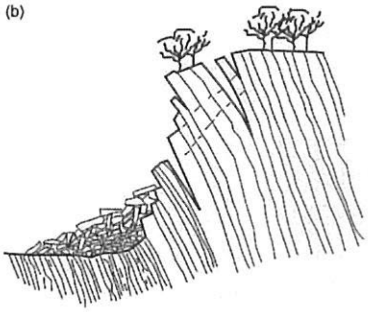
1. Block Toppling





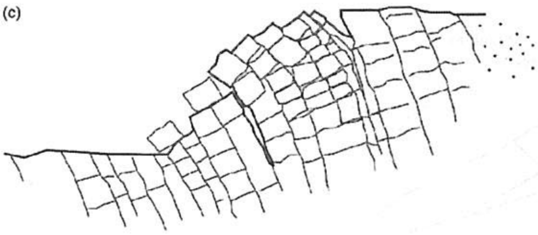
2. Flexural Toppling

(b)

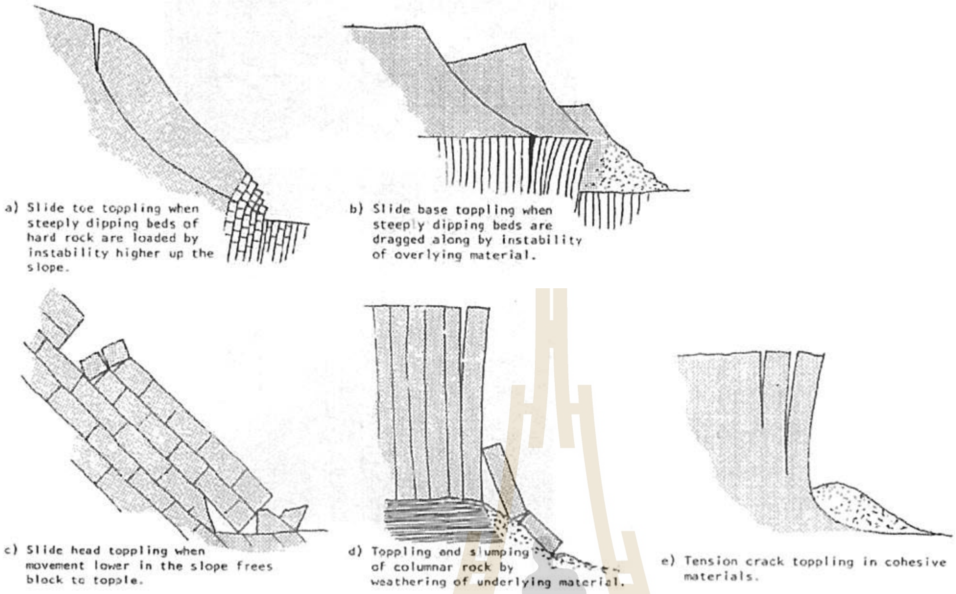


3. Block-Flexural Toppling

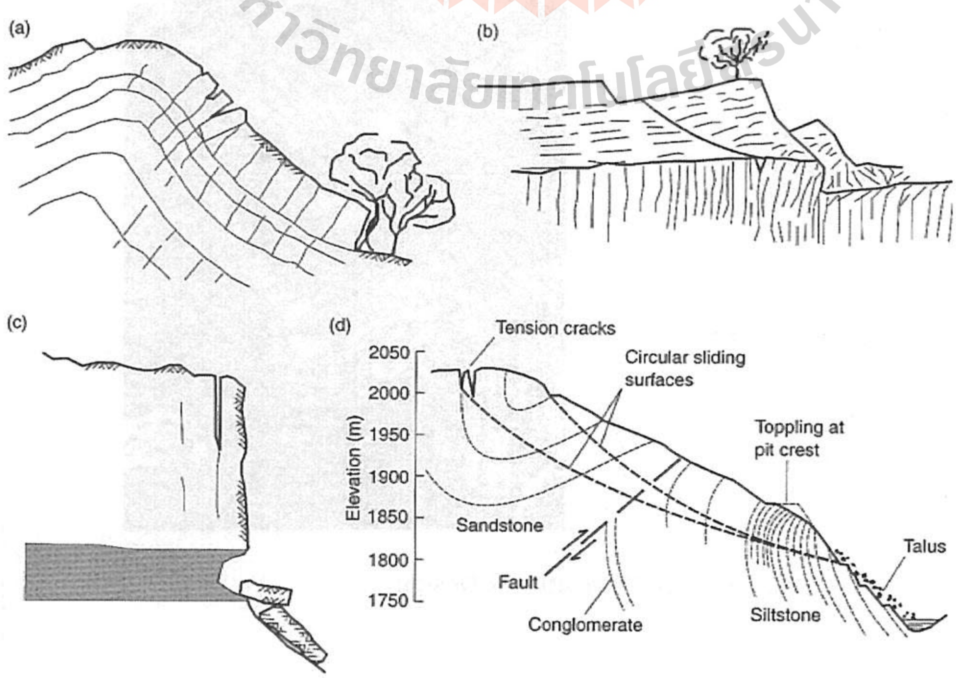
(c)



4. Secondary Toppling Modes



4. Secondary Toppling Modes (cont.)

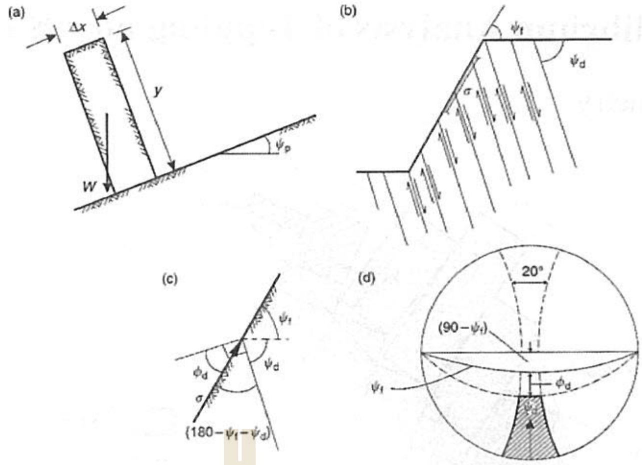


Kinematics of Block Toppling Failure

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

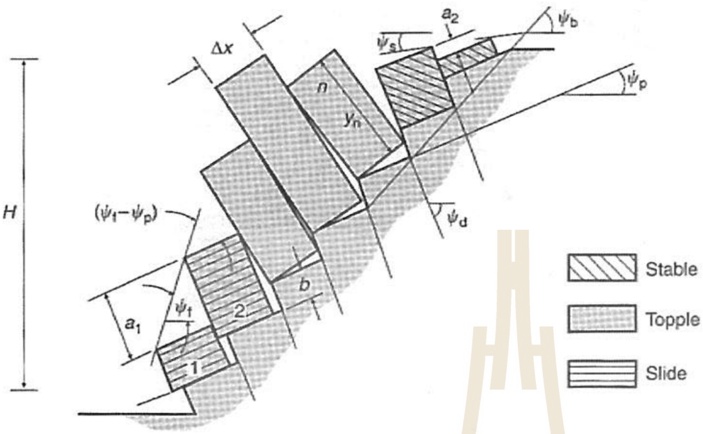
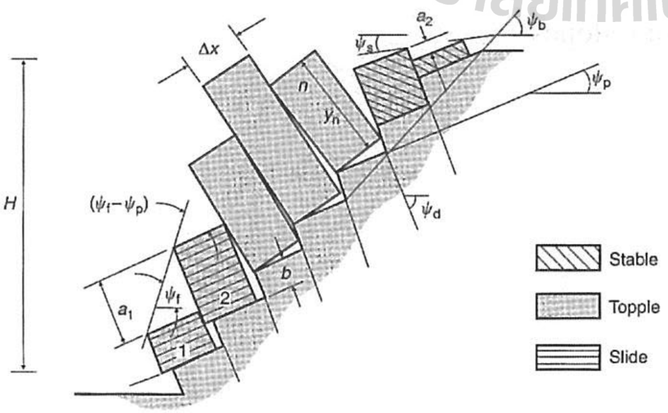


Figure 9.7 Model for limiting equilibrium analysis of toppling on a stepped base (Goodman and Bray, 1976).

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

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

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

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

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

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

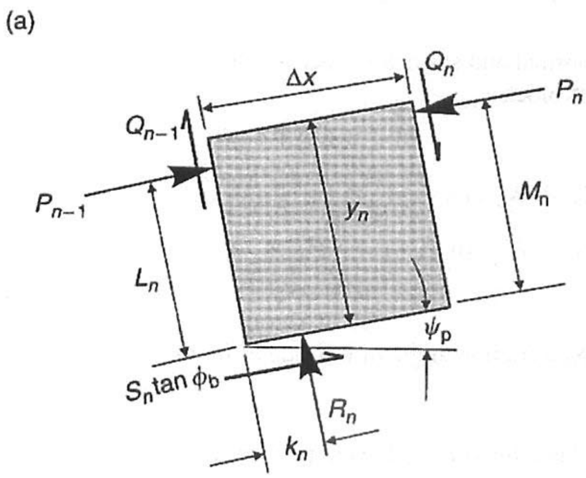
ψ_p = dip of the base of the block

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



in position below crest of slope

$M_n = y_n$

$L_n = y_n - a_1$

is the slope crest

$M_n = y_n - a_2$

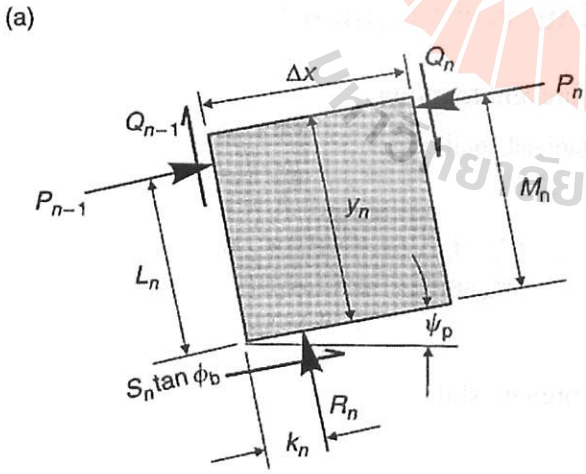
$L_n = y_n - a_1$

above the slope crest

$M_n = y_n - a_2$

$L_n = y_n$

Limit Equilibrium Analysis of Toppling on a Stepped Base



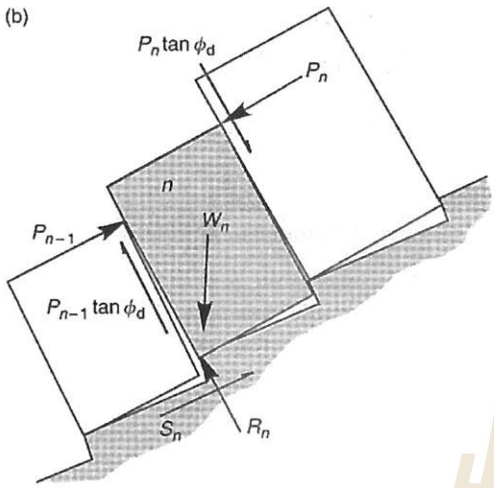
For limit friction on the side of block

$Q_n = P_n \tan \phi_d$

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

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

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

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

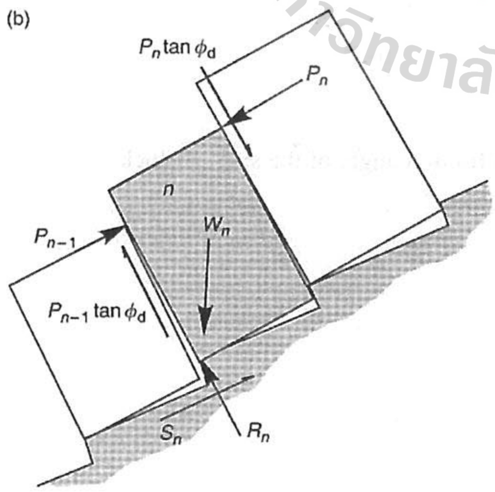
ϕ_d = friction angle of the side of block

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

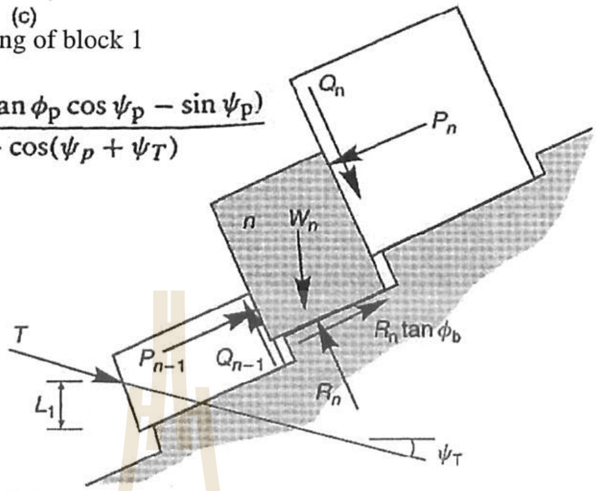
Cable Force Required to Stability a Slope

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

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



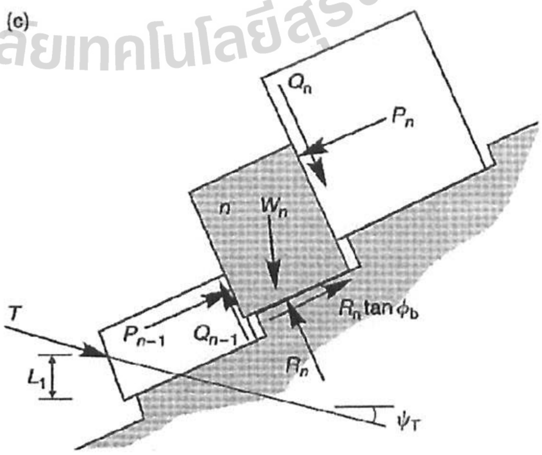
Cable Force Required to Stability a Slope

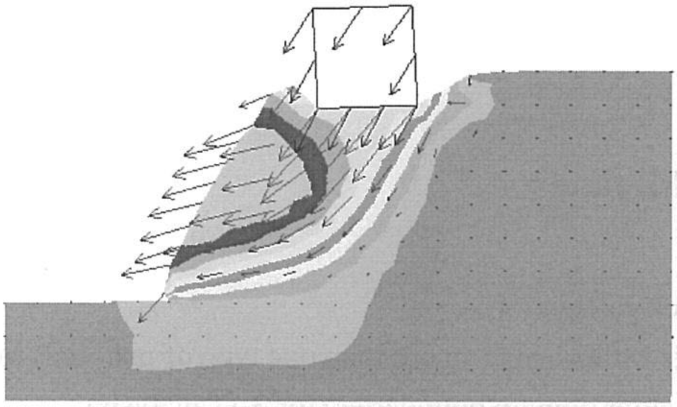
when the force T is applied to block 1,

the normal and shear force on the base are,

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

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





434422 Surface Excavation & Design
Topic 11 Numerical Analysis

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

Rock Slope Stability Analysis

1. **Deterministic Methods (Analytical Solution)**
   - **Limit Equilibrium (Factor of Safety)**
   - **Kinematics Analysis**
2. **Numerical Methods (Computer Simulation/Modeling)**
3. **Block Theory Method (Discontinuity Method)**
4. **Artificial Intelligence Methods (Expert System)**

Numerical Methods

Advantages :

- Allow quick calculation
- Incorporate Multi-layers (more than one type of material) in one domain
- Allow irregular domain boundaries (for 3-D analysis)

Numerical Methods

Disadvantages :

- Only give approximate solutions, accuracy is always less than analytical solutions
- True and in-depth technical knowledge is necessary
- Strong assumptions usually posed
  - Reduce 3-D domains to 2-D domains
  - Path and loading sequence
  - Loading rate
  - Time-dependent
  - Non-linear behavior
  - Coupled effects between solid and water
  - Multi phases flow
  - Large deformation/ displacement is commonly not allowed
  - Pre-existing joints or fractures
- Results auditing is necessary, but usually overlooked
- Required precise and representative material properties

Numerical Methods

Conditions Requirement for Numerical Analysis

- **Equilibrium**
- **Strain Compatibility**
- **Stress-Strain Relation**
- **Boundary Condition**

Numerical Methods

1. **Finite Element Method (FEM) /Finite Difference Method (FDM)**
2. **Boundary Element Method (BEM)**
3. **Discrete Element Method (DEM)**

1. Finite Element Method (FEM) /Finite Difference Method (FDM)

- **Domain Methods**
- **Continuum Material**
- **Mesh (Element & Nodal Point)**
- **Properties → Element**
- **Location → Nodal Point**
- **FEM → Integral Solving**
- **FDM →Differentiation Solving**

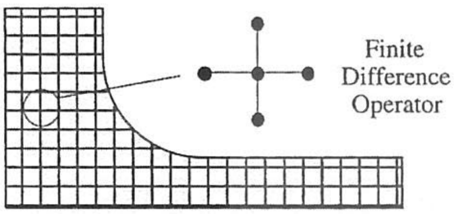


Figure 1.1 Finite Difference Grid

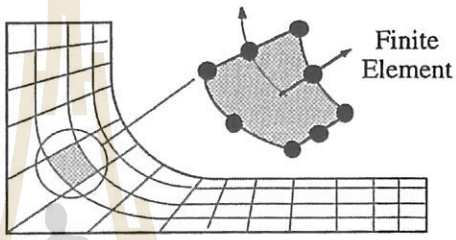


Figure 1.2 Finite Element Mesh

2. Boundary Element Method (BEM)

- **Infinite medium problems**
- **1 type of medium**
- **Required only surface grids**

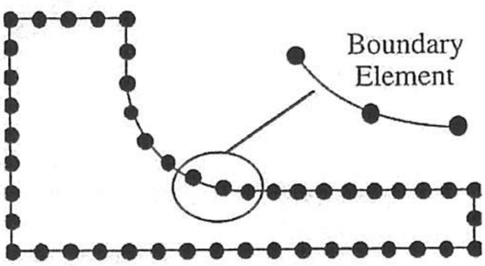
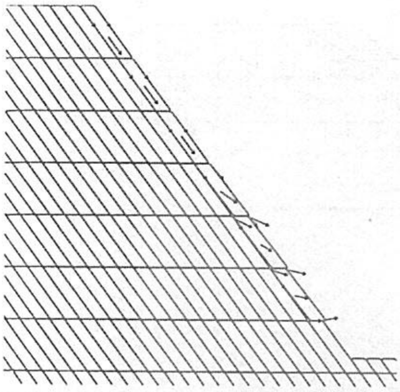


Figure 1.3 Boundary Element Mesh

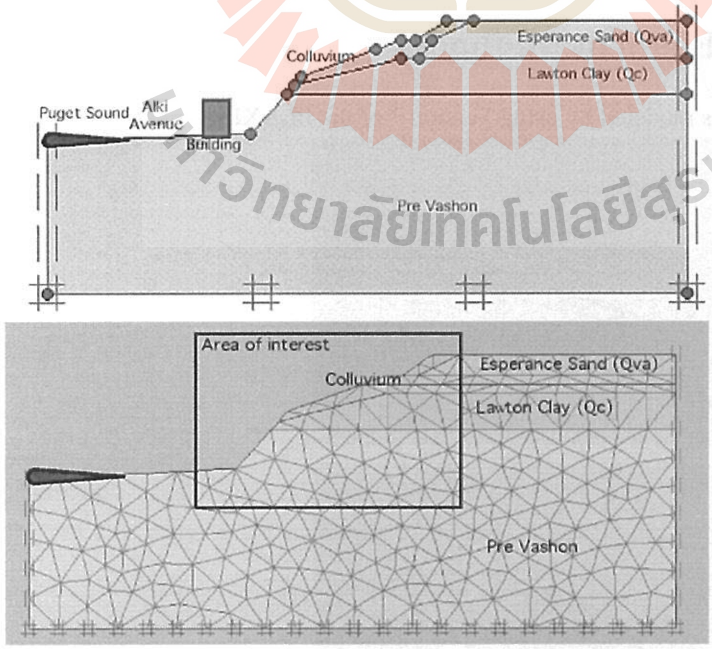
3. Discrete Element Method (DEM)

- **Discontinuity Method**
- **Mesh (Element & Nodal Point)**
- **Dynamics Equilibrium**
- **Not deformation (Movement only)**



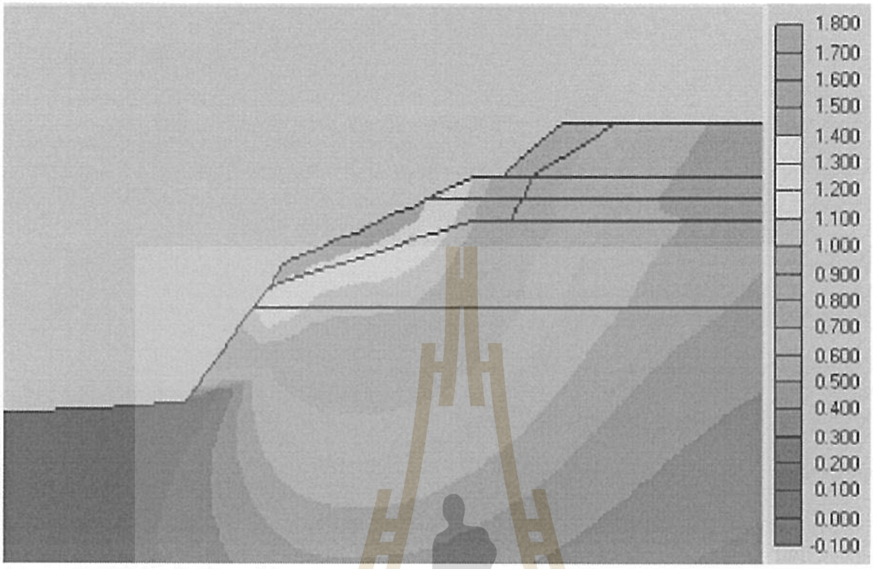
Example for Finite Element Method

Stability of the Alki Landslide is modeled by using Version 7 of the PLAXIS©



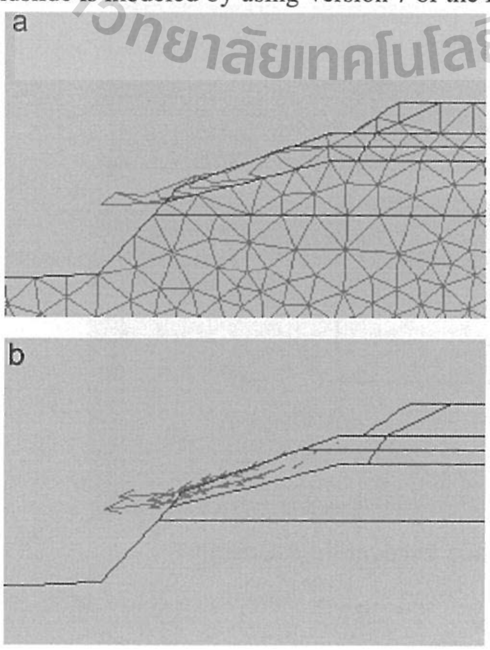
Example for Finite Element Method

Stability of the Alki Landslide is modeled by using Version 7 of the PLAXIS©



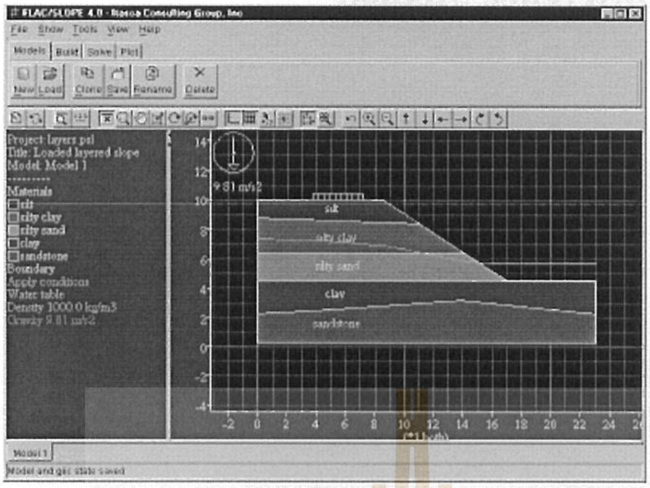
Example for Finite Element Method

Stability of the Alki Landslide is modeled by using Version 7 of the PLAXIS©



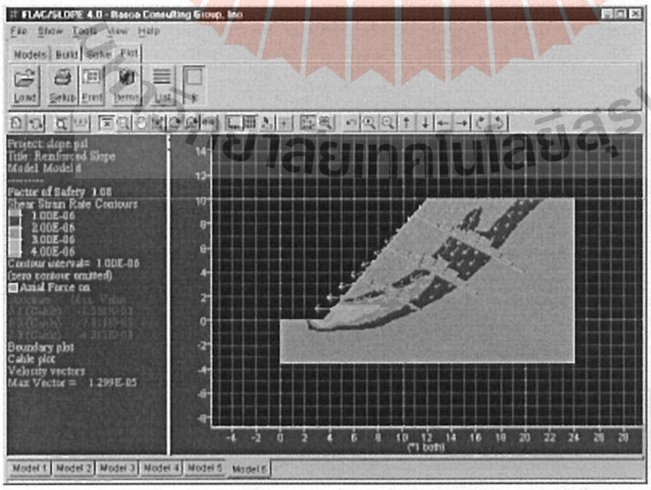
Example for Finite Difference Method

FLAC SLOPE 4.0



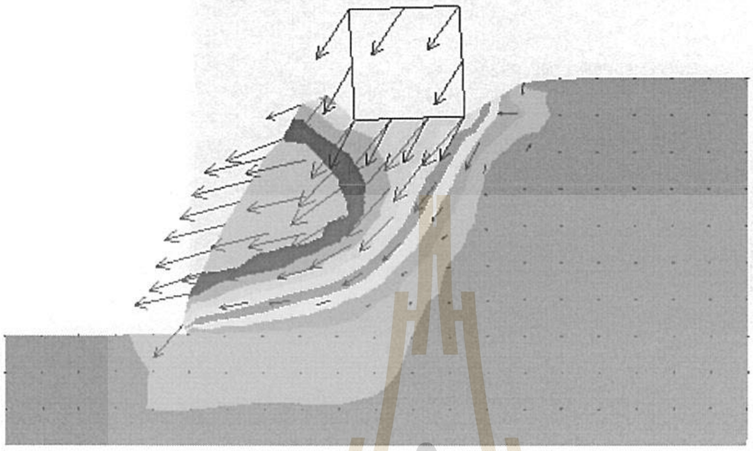
Example for Finite Difference Method

FLAC SLOPE 4.0



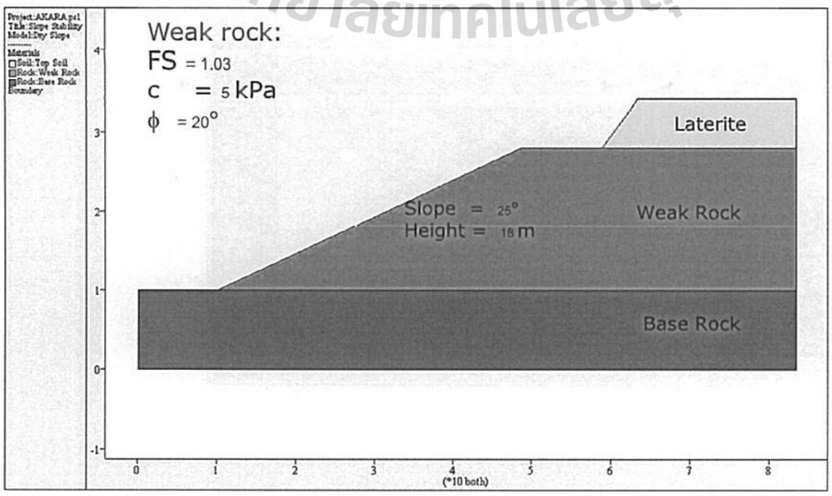
Example for Finite Difference Method

FLAC SLOPE 4.0



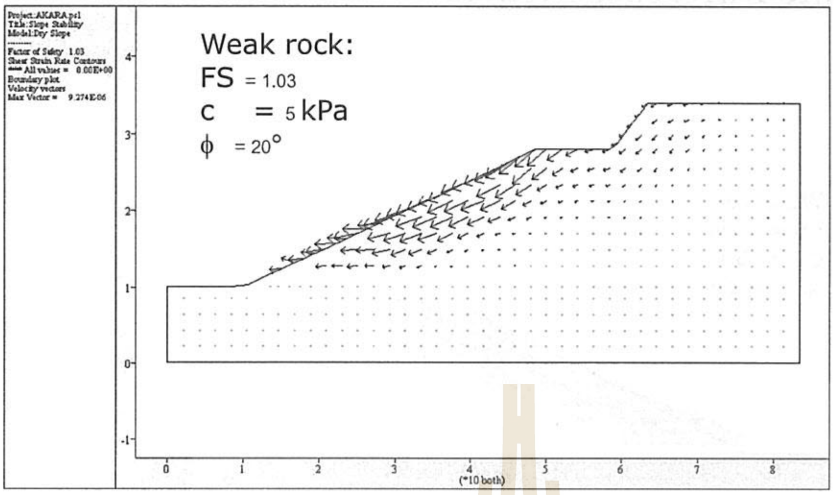
Example for Finite Difference Method

FLAC SLOPE 4.0



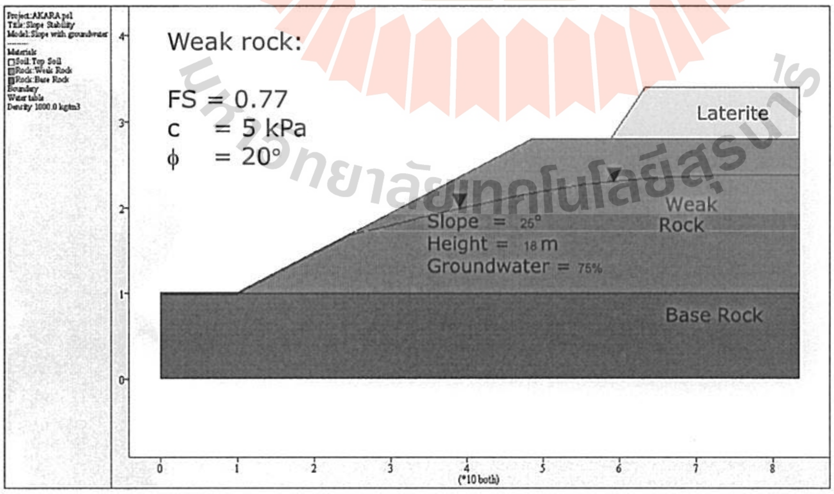
Example for Finite Difference Method

FLAC SLOPE 4.0



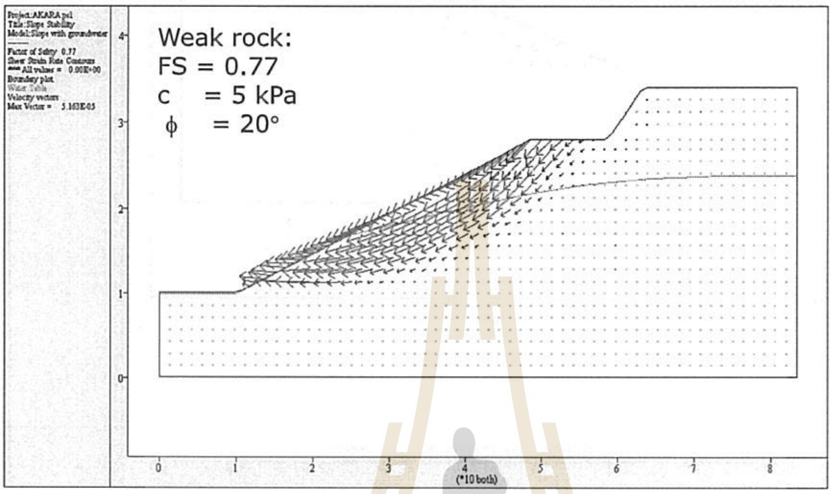
Example for Finite Difference Method

FLAC SLOPE 4.0



Example for Finite Difference Method

FLAC SLOPE 4.0



434422 Surface Excavation & Design

Topic 12 Slope Excavation Methods

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

Introduction for Slope Blasting

Blasting Rock Slope

- to obtain good fragmentation
- induce less damage to the remaining rock slope

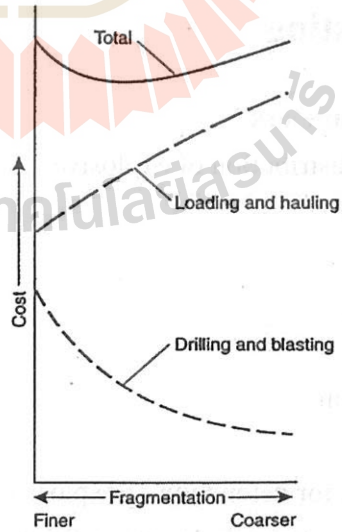
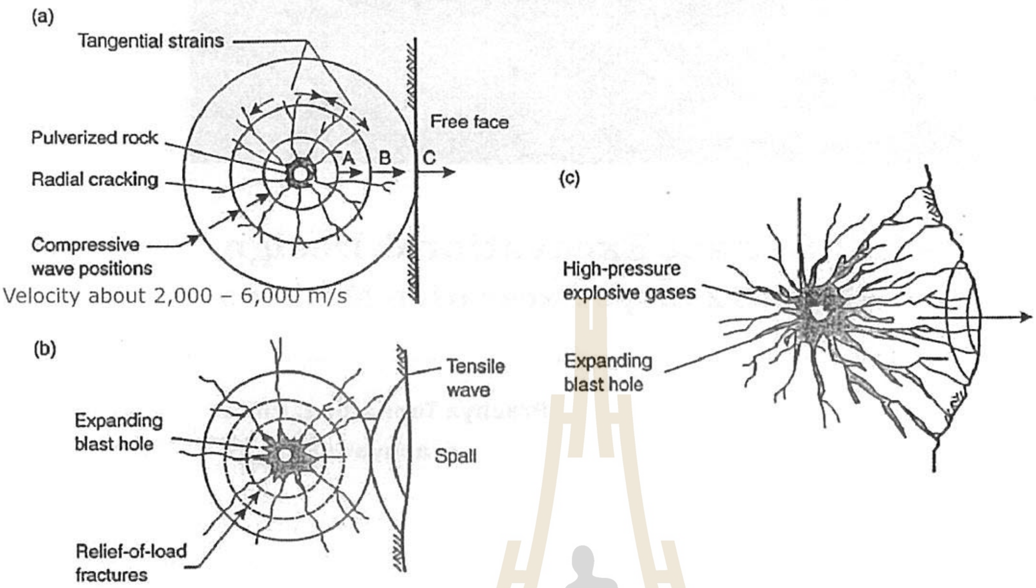


Figure 11.2 Effect of fragmentation on the cost of drilling, blasting, loading and hauling.

Mechanisms of Rock Fracturing by Explosive



Velocity about 2,000 – 6,000 m/s

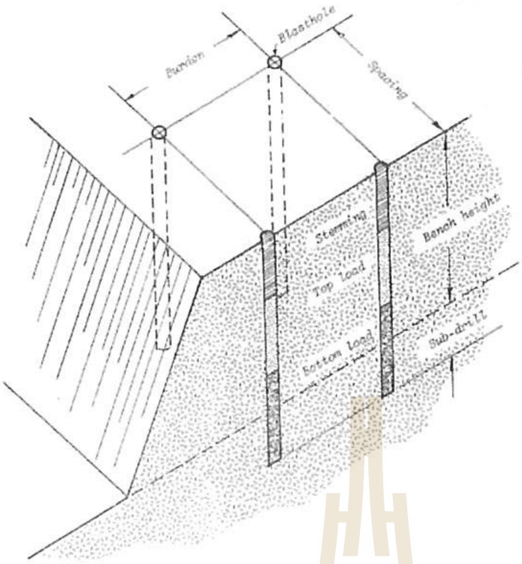
Production Blasting

Drill-and-Blast Parameters

- Type, weight and distribution of explosive
- Blasthole diameter
- Effective burden
- Effective spacing
- Sub-drill depth
- Blasthole inclination
- Stemming
- Initiation sequence for detonation of explosive
- Delays between successive hole or row firing.

Production Blasting

Definition of Bench Blasting Term



Drill-and-Blast Parameters

1. Type, weight and distribution of explosive.

The strength of an explosive is a measure of the work done by a certain weight or volume of explosive. This strength can be expressed in absolute units or as a ratio relative to a standard explosive such as gelignite or ANFO (Ammonium Nitrate / Fuel Oil)

Charge Factor = Explosive Weight (kg) / Rock Weight (ton)

Example:

Charge Factor = 0.5, Hydrogel = ?? kg

Rock Weight 1 ton → ANFO = 0.5 kg

ANFO → Weight Strength = 100%

Hydrogel → Weight Strength = 111 %

→ (0.5 x 100) / 111 = 0.45 kg

Drill-and-Blast Parameters

TABLE VI - STRENGTHS OF EXPLOSIVES COMPARED TO ANFO

Explosive	*Weight strength % ANFO*	*Bulk Strength % ANFO*	*Specific Gravity*
ANFO (Gravity loaded)	100	100	0.82
ANFO (Pressure loaded)	100	109	0.92
A.N. Gelatine Dynamite '75'	114	195	1.40
A.N. Gelignite '60'	95	174	1.50
A.N. 'Ligdyn 40'	85	149	1.43
A.N. 'Ligdyn 25'	68	119	1.42
'Anzite' Blue	114	193	1.40
'Anzite' Red	114	193	1.40
'Anzite' Yellow	97	165	1.43
'Aquamex'	100	170	1.39
Blasting Gelatine	127	233	1.50
'Exactex'	90	107	0.96
'Geophex'	85	163	1.55
'Hydrogel'	111	205	1.50
'Hydromex' M1	95	124	1.50
'Hydromex' M2	127	233	1.50
'Hydromex' M4	152	279	1.50

Drill-and-Blast Parameters

TABLE VI - STRENGTHS OF EXPLOSIVES COMPARED TO ANFO

Explosive	*Weight strength % ANFO*	*Bulk Strength % ANFO*	*Specific Gravity*
'Molanal' A	82	140	1.3-1.4
'Molonal' D	114	195	1.3-1.4
'Molonal' DQ	114	195	1.3-1.4
'Monograin'	90	107	0.90
'Plastergel'	95	174	1.50
'Quarigel'	101	186	1.50
Quarry 'Monobel'	100	121	0.98
'Rollex' 60	97	174	1.45
'Roxite'	63	121	1.65
'Seismex'	101	174	1.10
'Seismex' (Aluminised)	113	151	1.10
S.N. Gelignite 50%	89	163	1.50
Semigel	106	226	1.20
Semigel No. 2	99	135	1.12
'Ajax'	71	135	1.50
'Dynagex'	57	84	1.39
'Dynobel' No. 2	81	109	1.10
'Morcol'	80	116	1.20
'Polar' A3 'Monobel'	71	86	0.98

Drill-and-Blast Parameters

2. Blasthole Diameter

Blasthole diameter d ≤ Bench height/40

too large
→ fly rock
→ damage to the remaining rock
→ air blast

too small
→ choking
(แรงไม่พอที่จะกระแทกหินออกมาได้)

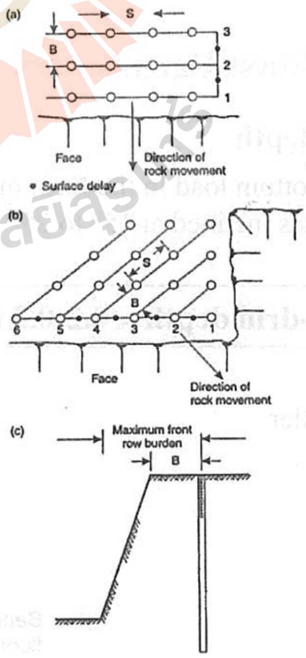
Drill-and-Blast Parameters

3. Effective Burden

$B_e \cong$ 40 times of Blasthole Diameter

or

$B_e \cong 0.33H$ to $0.25H$



too small
→ fly rock
→ venting problem
(leak along fracture)

too large
→ choking
(แรงไม่พอที่จะกระแทกหินออกมาได้)
→ poor fragmentation

Drill-and-Blast Parameters

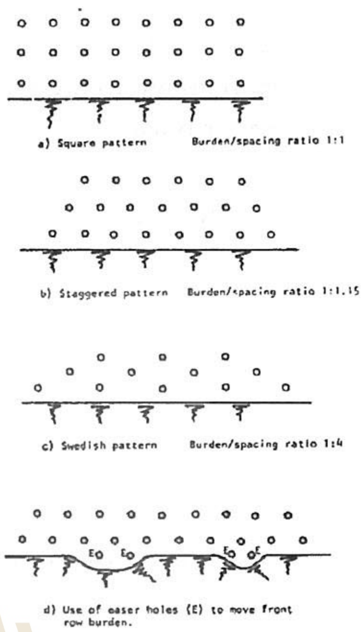
4. Effective Spacing

$S_e \cong 1.25\ B_e$ (Experience suggests)

B_e and S_e depend not only upon the blasthole pattern but also upon the sequence of firing.

too small → desensitization
(หลุมข้าง ๆ ระเบิดตาม)

too large → poor fragmentation



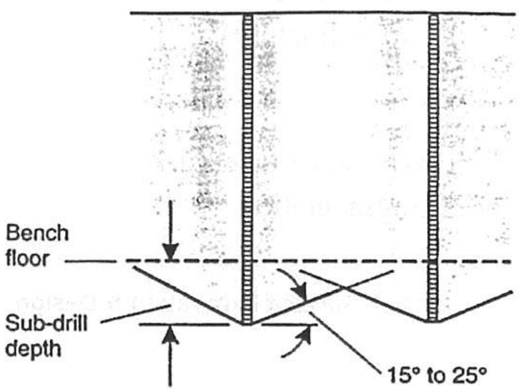
Drill-and-Blast Parameters

5. Sub-drill depth

- base of the bottom load in the form of an inverted cone with sides inclined at 15° to 25°

Sub-drill depth = 0.2-0.3 (S_e or B_e)

which use smaller



Drill-and-Blast Parameters

6. Blasthole inclination

- Inclined blastholes are obviously advantageous for the front row and, by drilling the blastholes parallel to the bench face, a constant front row burden is achieved.

- Some blasting engineers would argue that the use of blastholes drilled at **between 10° and 30°** to the vertical will give better fragmentations, greater displacement and reduced back-break problems.

Drill-and-Blast Parameters

7. Stemming

- Dry and well graded angular materials
- 10-15 mm crushed rock,
- The optimum stemming length depends upon the properties of the rock

Stemming depth ≅ 0.67 - 2 (B_e)

too little (shorter than two thirds of the burden)
→ fly rock
→ air blast
→ backbreak problems
→ **reducing the effectiveness of the blast**

too large
→ poor fragmentation

Drill-and-Blast Parameters

8. Initiation sequence for detonation of explosive

- The firing or initiating line will normally be connected to the middle of the front row trunk line.
- The blasting sequence, after the initiation of the first row, is controlled by the use of delays.

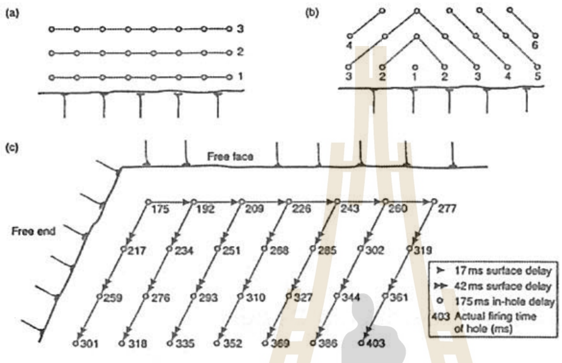


Figure 11.7 Typical detonation sequences: (a) square "row-by-row" detonation sequence; (b) square "V" detonation sequence; (c) hole-by-hole detonation using both surface and in-hole non-electric delays (W. Forsyth).

Drill-and-Blast Parameters

9. Delays between successive hole or row firing

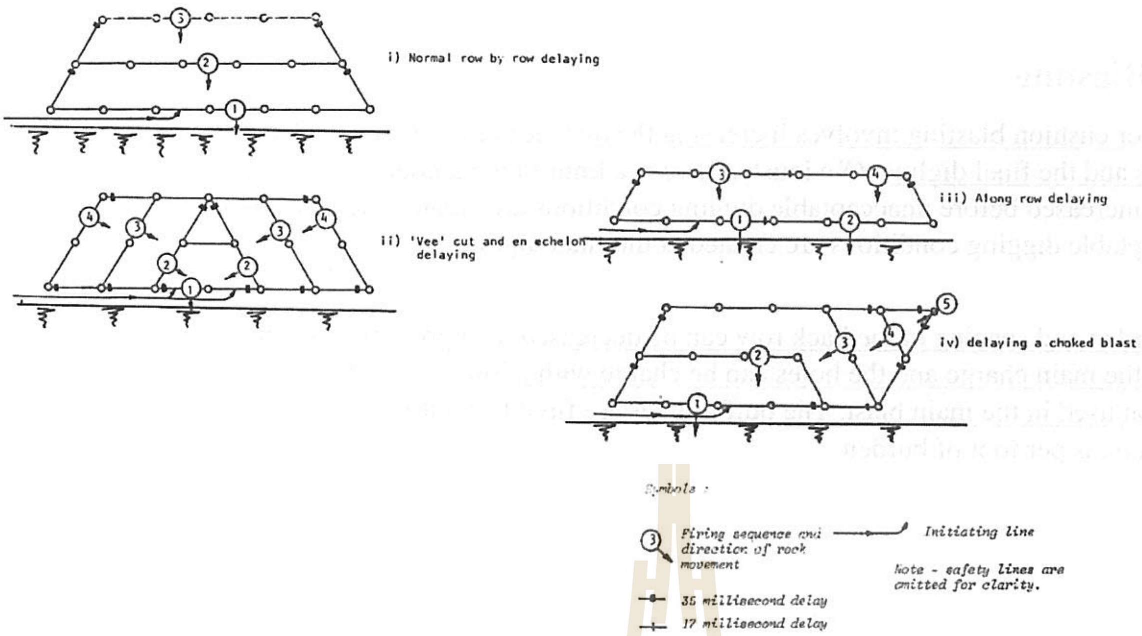
- Typically, delay intervals of 1 to 2 milliseconds per foot of burden (3 to 6 milliseconds per meter) are used in production blasting

too fast → desensitization (หลุมข้าง ๆ ระเบิดตาม)
→ air blast (รุนแรง)

Front row	-	instantaneous
Row 2	-	35 milliseconds delay
Row 3	-	70 milliseconds delay
Row 4	-	105 milliseconds delay

- The use of delays in a blast in one of the most powerful weapons in the fight against excessive blast damage and the instability of benches in open pit mines.

Drill-and-Blast Parameters



Controlling Slope Damage.

1. Choke blasting into excessive burden or broken muck piles should be avoided.
2. The front row charge should be adequately designed to move the front row burden.
3. The main charge and blasthole pattern should be optimized to give the best possible fragmentation and digging conditions for the minimum powder factor.
4. Adequate delays should be used to ensure good movement towards free faces and the creation of new free faces for following rows.
5. Delays should be used to control the maximum instantaneous charge to ensure that rock breakage does not occur in the rock mass which is supposed to remain intact.
6. Back row holes should be drilled at an optimum distance from the final digline to permit free digging and yet minimize damage to the wall. Experience can be used to adjust the back row positions and charges to achieve this result.

If all of these conditions have been satisfied and a bench instability problem due to over-break still exists, consideration should be give to the use of special blasting techniques such as **buffer blasting**, **Pre-splitting** and **smooth-wall blasting.**

Buffer Blasting

- Buffer or cushion blasting involves increasing the distance between the back row charges and the final digline. Obviously, there is a limit to the amount this distance can be increased before unacceptable digging conditions are created increased before unacceptable digging conditions are created at the final digline.

- The burden and spacing in the back row can be decreased to approximately one half that of the main charge and the holes can be charge with a lower strength explosive than that used in the main blast. The buffer holes are fired last with a delay of 1 to 2 milliseconds per foot of burden.

Pre-splitting

- Pre-splitting or pre-shearing is a technique which is used very extensively and very successfully in civil engineering excavations in hard rock. Its use in mining, particularly with large diameter blastholes, is less common but the technique merits serious consideration by open pit engineers.

- A row of closely spaced and usually small diameter holes is drilled along the line of the final face. These holes are lightly charged and the charge is de-coupled from the rock by leaving an air space between the charge and the walls of the blasthole.

- The row is fired before the man charge and the reinforcing effect of the closely spaced holes together with the very large burden results n the formation of a clean fracture running from one hole to the next. A good pre-split face is characterised by a clean fracture running between the parallel half barrels of the blastholes as illustrated in the margin photograph.

- Pre-split blasting is not usually successful in well jointed hard rocks, particularly where the joints are open and are inclined to the pre-split line. These open joints allow the explosion gases to vent and fracturing follows the joints rather than the intended pre-split line.

Pre-splitting

TABLE X - RECOMMENDED DIMENSIONS FOR SMOOTH-WALL AND PRE-SPLIT BLASTING											
Drillhole diameter		*Charge diameter*		*Explosive**		*SMOOTH-WALL BLASTING Spacing*		*Burden*		*PRE-SPLIT BLASTING Spacing***	
mm	in	mm	in	kg/m	lb/ft	m	ft	m	ft	m	ft
30	1.25	11	0.5	0.07	0.05	0.5	1.6	0.7	2.3	0.25-0.3	0.8-1.0
37	1.5	17	0.63	0.12	0.08	0.6	2.0	0.9	3.0	0.30-0.5	1.0-1.6
44	1.75	17	0.63	0.17	0.11	0.6	2.0	0.9	3.0	0.30-0.5	1.0-1.6
51	2.0	22	0.88	0.25	0.17	0.8	2.6	1.1	3.6	0.45-0.75	1.5-2.5
62	2.38	22	0.88	0.35	0.23	1.0	3.3	1.3	4.2	0.55-0.8	1.8-2.6
75	3.0	25	1.0	0.50	0.34	1.2	4.0	1.6	5.2	0.60-0.9	2.0-3.0
87	3.5	25	1.0	0.70	0.47	1.4	4.6	1.9	6.2	0.70-1.0	2.3-3.3
100	4.0	29	1.13	0.90	0.60	1.6	5.2	2.1	6.9	0.80-1.2	2.6-4.0
125	5.0	40	1.63	1.40	0.94	2.0	6.6	2.7	8.8	1.00-1.5	3.3-4.9
150	6.0	50	2.0	2.00	1.34	2.4	7.9	3.2	10.5	1.20-1.8	4.0-5.9
200	8.0	52	2.0	3.00	2.02	3.0	9.8	4.0	13.0	1.50-2.1	4.9-6.9
250	10.0	65	2.5	3.38	2.27	3.4	11.2	4.5	14.8	1.80-2.4	5.9-7.9

* Base on Nitro Nobel's Dynamex B explosive, charge per unit length of hole.

** The burden is assumed to be infinite since the pre-split charge is fired before the main charge

Smooth-wall Blasting

- Smooth-wall or post-spit blasting is similar to pre-split blasting except that the line of holes of fired after the main blast. This means that a free face exist close to the line of charged holes and hence a burden and spacing design has to be specified for this blast.

- Smooth-wall blasting is sometimes used as a clean-up operation to minimize the danger of rockfalls from a face which has been heavily blasted or where jointing has created loose blocky conditions on the face

Blast damage

Type of blasting damage are identified

1. Structural damage due to vibration induced in the rock mass
2. Damage due to fly rock or boulders ejected from the blast area
3. Damage due to airblast
4. Damage due to noise

Blast damage control

- Control of fly rock : Fly rock problem are caused by catering as a result of inadequate stemming or too small a front row burden.

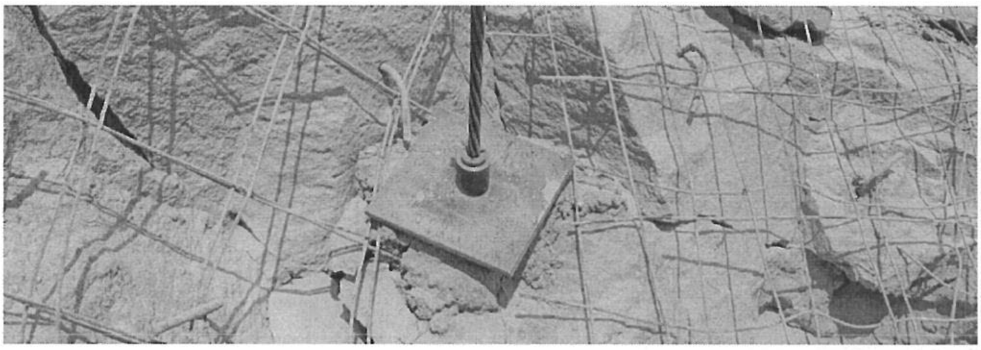
  Eliminated by

  - reducing the power factor to 0.2 kg/m^3,
  - increase the front row burden,
  - increase the stemming column length of 40 blast hole diameters and the optimum stemming column length of 0.67-2 time the burden.

- Airblast and noise problem associated with production blasts : Factors contributing to the development of an airblast and to noise include

  - overcharged blasthole
  - poor stemming
  - uncovered detonating cord,
  - venting of developing cracks in the rock
  - and the use of inadequate burdens giving rise to cratering.

- The propagation of the pressure wave depends upon atmospheric condition including temperature, wind and pressure-altitude relationship.



434422 Surface Excavation & Design

Topic 13 Stabilization of Rock Slopes

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

Design of Rock Slope Support System

Rock slope stabilization :

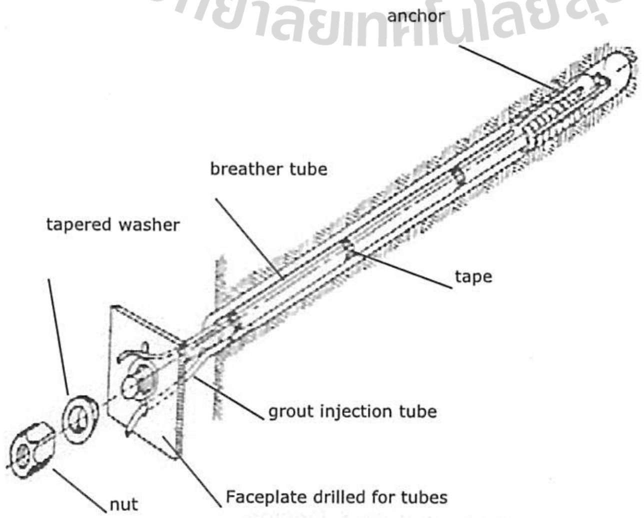
1) Removal of Unstable Rock
2) Catchments
3) Flattening of Slope
4) Buttresses
5) Surface Protection
6) Reinforcement
7) Drainage

Types of Rock Slope Supports (Stabilization)

1. Rockbolts
2. Dowels
3. Cable bolts
4. Shotcrete
5. Wire mesh
6. Pre-cast Concrete
7. Retaining Wall
8. Gabions

1. Rockbolts

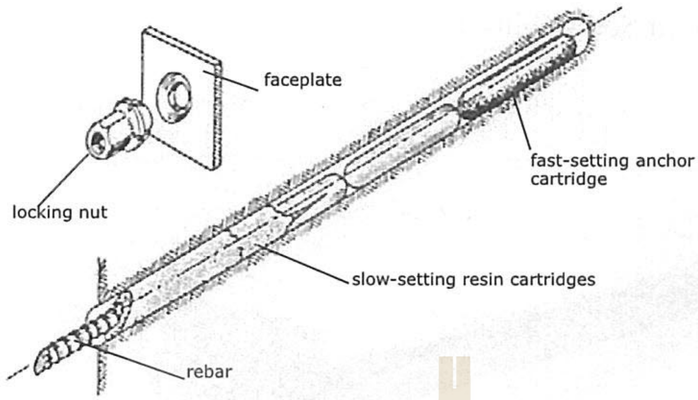
1.1) Mechanically anchored rockbolts



Grout injection arrangements for a mechanically anchored rockbolts.

1. Rockbolts

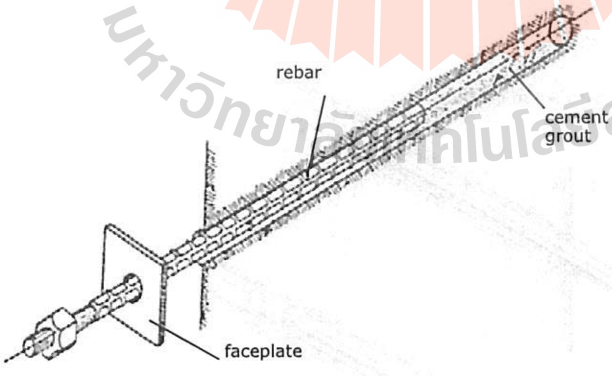
1.2) Resin anchored rockbolts



Typical set-up for creating a resin anchored and grouted rockbolt.

2. Dowels

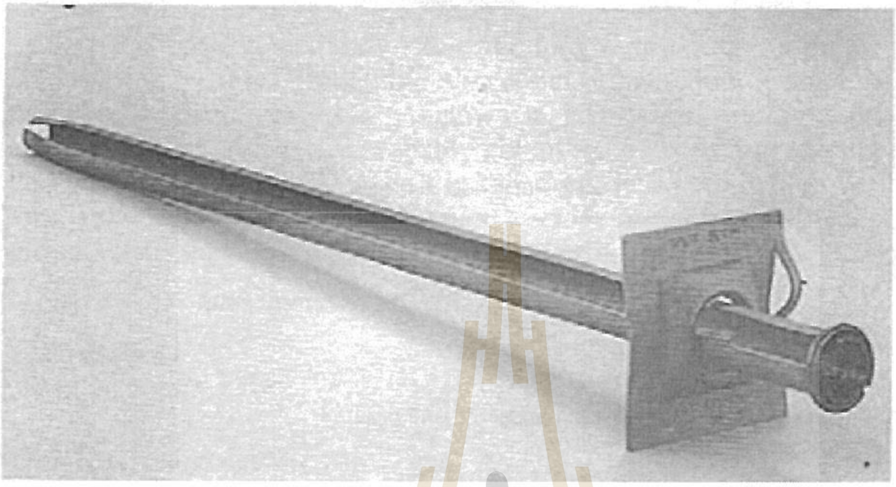
2.1) Grouted dowels



Grouted dowel using a deformed bar inserted into a grout-filled hole.

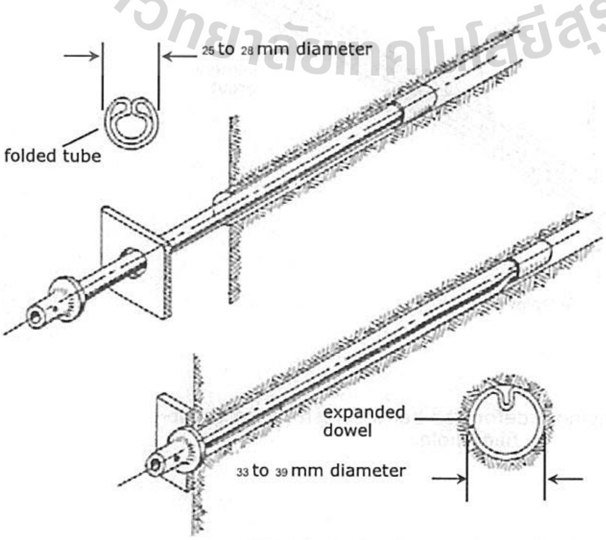
2. Dowels

2.2) Friction dowels or 'Split Set' stabilizers



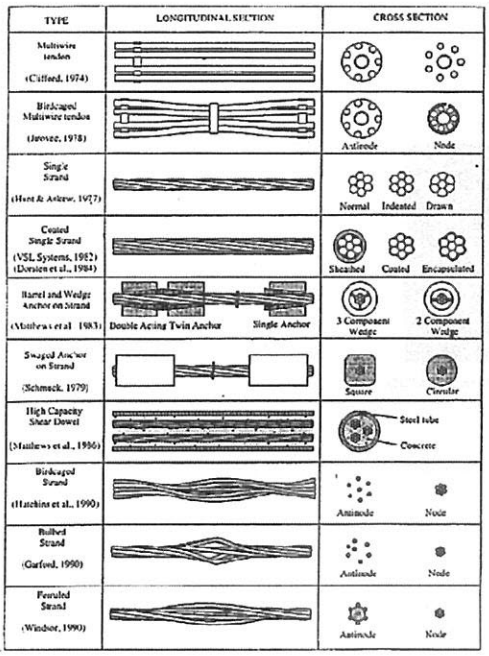
2. Dowels

2.3) 'Swellex' dowels



Atlas Copco Swellex dowel.

3. Cablebolts

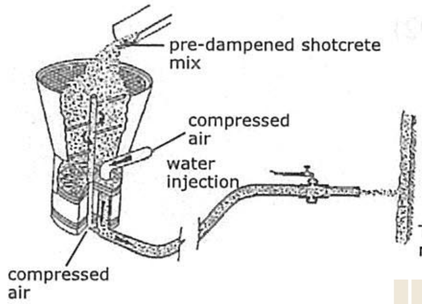


Summary of the development of cablebolt configurations. After Windsor (1992)

4. Shotcrete

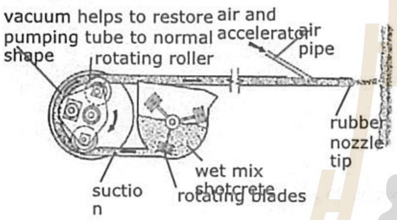
1) Dry mix shotcrete
2) Wet mix shotcrete
3) Steel fiber reinforced micro silica shotcrete
4) Mesh reinforced shotcrete

4. Shotcrete



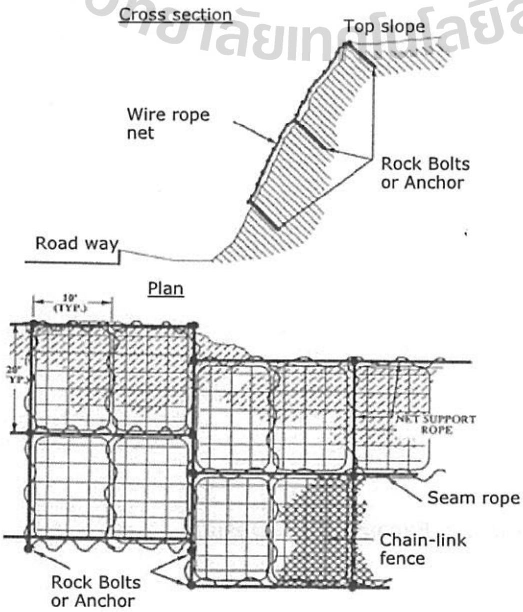
Typical dry mix shocrete machine.

Simplified sketch of a typical dry mix shotcrete system. After Mahar et al. (1975).

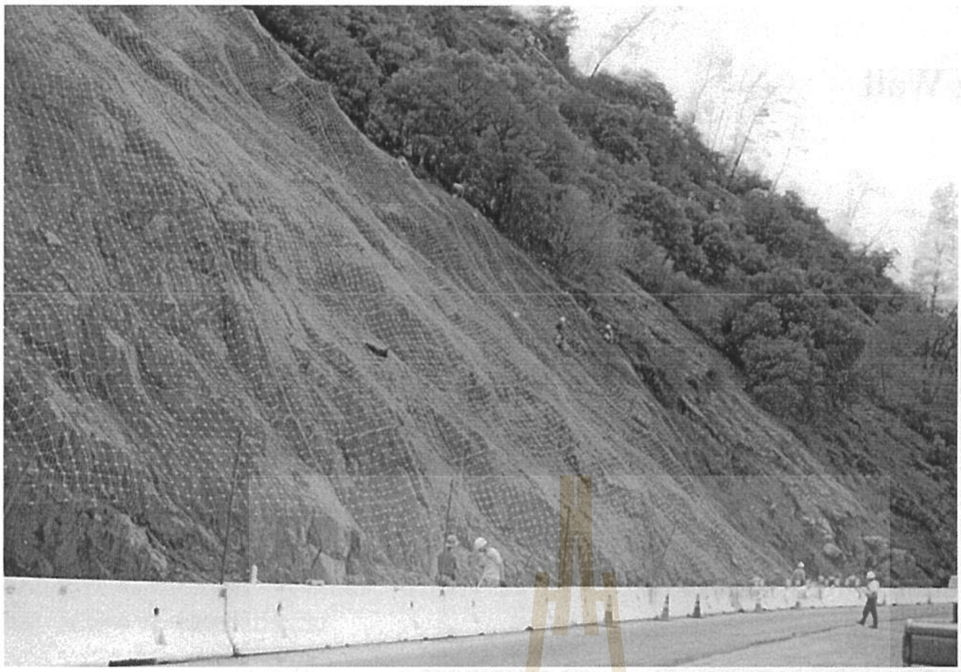


Typical wet mix shotcrete machine. After Mahar et al. (1975).

5. Wire mesh

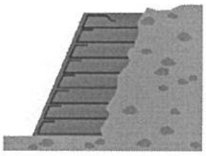


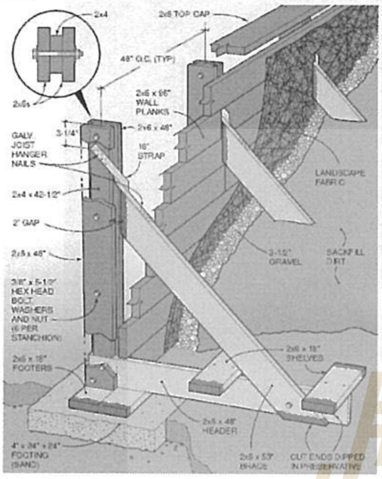
5. Wire mesh

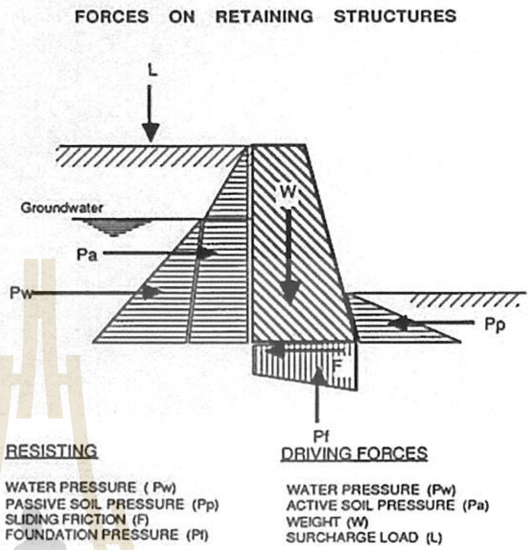


6. Pre-cast Concrete

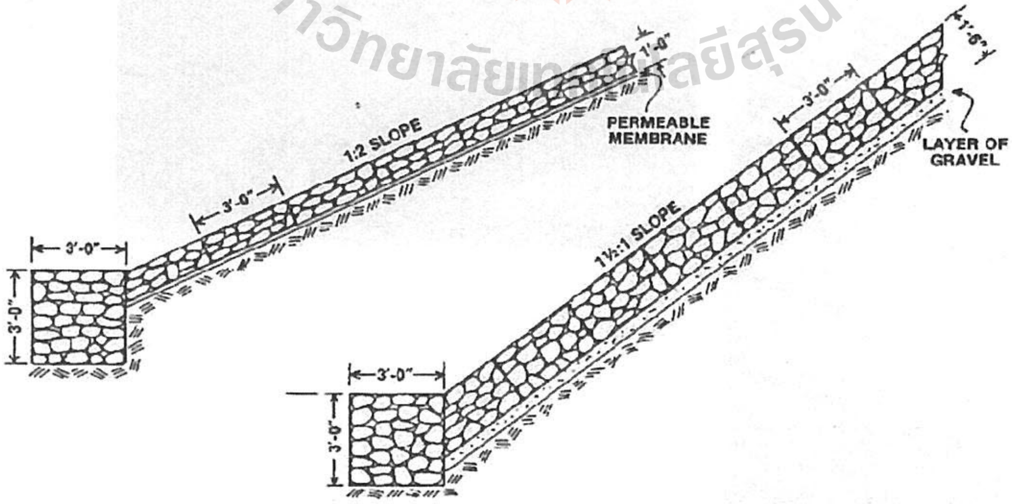
7. Retaining Wall



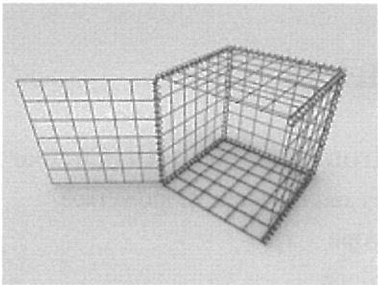


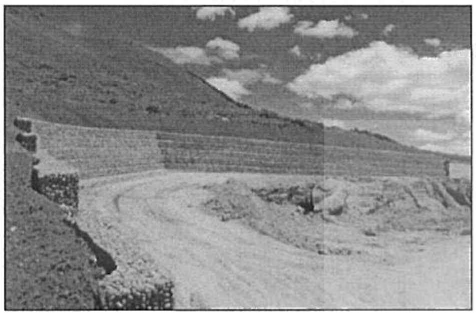


8. Gabion



8. Gabion





Safety Requirements

Slope Type

Type A is for the slope toe nearby the residential structures or power plant facilities.

Type B is for the slopes along he main highways, railroads, and large bridges.

Type C is for the slopes along the small roads and reservoirs.

Type D is defined for the temporary access or small roads in open pit mines.

Design Parameters

1. Slope failure from rock strength (Circular Failure)
   - Orientations and dip angle of slope face
   - Height of slope
   - Length of Slope
   - Unit weight of rock

2. Slope failure from rock fracture (Plane, Wedge, and Toppling Failure)
   - Orientations and dip angle of slope face
   - Height of slope
   - Length of Slope
   - Unit weight of rock
   - Orientations and dip angle of failure surface
   - Joint spacing

Design Methodology and Criterion

1. Stabilization Method
   - Rockbolt / Cablebolt
   - Rockbolt / Cablebolt + Wire Mesh
   - Rockbolt / Cablebolt + Wire Mesh + Shotcrete + Drainage
   - Spot of Rockbolt / Cablebolt
   - Drainage

2. Slope Modification

3. Combined Methods
   - Slope Modified + Rockbolt / Cablebolt
   - Slope Modified + Rockbolt / Cablebolt + Wire Mesh
   - Slope Modified + Rockbolt / Cablebolt + Wire Mesh + Drainage

Design and Selection of Support Types

Parameters Considered	Functional Requirements	Design Solutions	Design Components	Constraints	Design Specifications:
σ_c = 0.25-1 & 1-5 MPa				None	1. 5-7 m / 35°* 2. 7-10 m / 30° 3. > 10 m / bench width ≥ 4 m & working face = 30°
σ_c = 5-25 MPa				A & B	1. 5-10m / 50° 2. 10-15m / 45° 3. 15-20m / 40° 4. > 20m / bench width ≥ 4 m & working face = 40°
σ_c = 5-25 MPa	Reduce driving force	Solution : 5 Modify slope shape	1. Slope height 2. Slope face angle	C & D	1. 5-10 m/ 60° 2. 10-15m / 55° 3. 15-20m / 50° 4. > 20m / bench width ≥ 4 m & working face = 50°
σ_c = 25-50 MPa				A & B	1. 5-7m / 65° 2. 7-10m / 60° 3. 10-15m / 50° 4. 15-20m / 45° 5. > 20m / bench width ≥ 4 m & working face = 45°
σ_c = 25-50 MPa				C & D	1. 5-7m / 75° 2. 7-10m / 70° 3. 10-15m / 60° 4. 15-20m / 55° 5. > 20m / bench width ≥ 4 m & working face = 55°

Design and Selection of Support Types

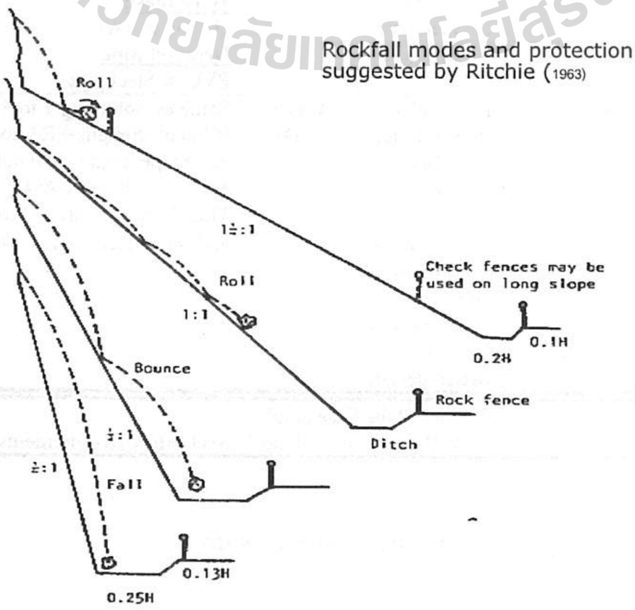
1. Dip direction of failure plane 2. Average joints spacing 3. Slope height 4. Slope length 5. Slope dip direction 6. Slope dip angle 7. Rock unit weight 8. Groundwater level 9. Intact strength	1. Increase resisting force 2. Reduce driving force	Solution : 1	Rock bolts	(A, B, C or D)	**Rock bolts** Fully grout steel rebar (A &B) Rock anchored (C & D) **Grout materials** Resin (A) Cement (B) **Wire mesh** Galvanize (A) **Drained pipe** PVC or Steel pipe
		Solution : 2	Rock bolts Wire mesh		
		Solution : 3	Rock bolts Wire mesh Drained pipe		
		Solution : 4	Drained pipe		
		Solution : 6	Rock bolts Bench design		Same as solution : 1 to 5 but If Intact strength = R3 to R4 and Slope height > 30 m (A & B) or > 40 m (C & D) Then Bench width ≥ 4 m and Slope face angle < 60°
		Solution : 7	Rock bolts Wire mesh Bench design		
		Solution : 8	Rock bolts Wire mesh Drained pipe Bench design		
		Solution : 9	Drained pipe Bench design		

σ_c = Uniaxial Compressive Strength, * Slope Height / Slope Face Angle,
** Williams Form Engineering Corp (2002), A, B, C and D = Slope Types (Safety Requirements)

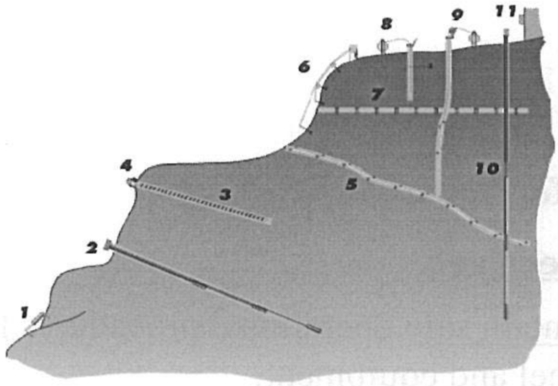
Design Specifications

1. Rockbolts
2. Grout Material
3. Wire Mesh
4. Drain Pipe
5. Ditch
6. Safety Area

Control of Rock Fall



Rockfall modes and protection system suggested by Ritchie (1963)



434422 Surface Excavation & Design
Topic 14 Slope Movement Monitoring

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

Objectives for Slope Monitoring

Typical measurement objectives

- To determine absolute lateral and vertical movements of a sliding surface.
- To determine the rate of sliding (accelerating or decelerating) and thus warn of impending dangers.
- To determine the depth and shape of the sliding surfaces.
- To determine the relative movements within a slope.
- To monitor groundwater levels and pore pressures so that analyses can be made.

Objectives for Slope Monitoring

overall objectives

- To maintain safe operational procedures for the protection of personnel and equipment.
- To provide advance notice of instability so that mine plans can be modified to minimize the impact of slope displacement.
- To provide geotechnical information for analyzing the slope failure mechanism, for designing appropriate remedial measures, and for conducting future redesign of the slope.

Instruments suitable for examining slope stability during excavation

Measurement	Suitable Instruments
Surface deformation	Surveying methods Crack gages Tiltmeters Multipoint liquid level gages
Subsurface deformation	Inclinometers Fixed borehole extensometers Slope extensometers Shear plane indicators Multiple deflectometers In-place inclinometers Combined piezometer–inclinometer system Acoustic emission monitoring
Groundwater pressure	Single piezometers Multipoint piezometers Combined piezometer–inclinometer system

Overview of routine and special monitoring

Application	Measurement
Routine monitoring	Surface deformation Groundwater pressure
Special applications	Subsurface deformation Load in rockbolts Temperature

Criteria for Selecting Site-Specific Instruments

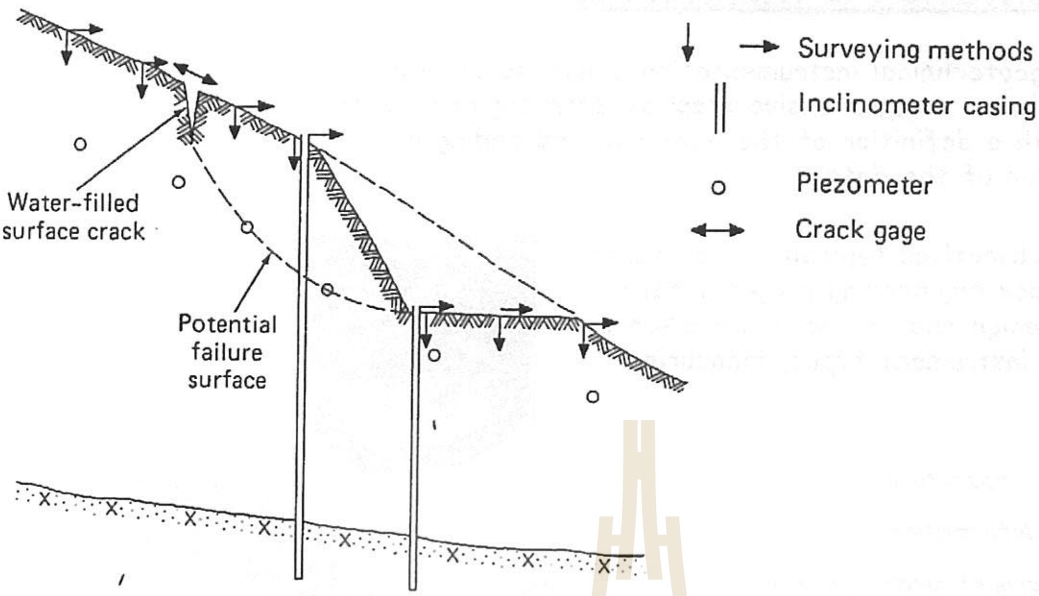
- Measure the obvious things first
- Simpler is better
- Precision costs money
- Redundancy is required
- Timely reporting is essential

All instruments have certain requirements in common

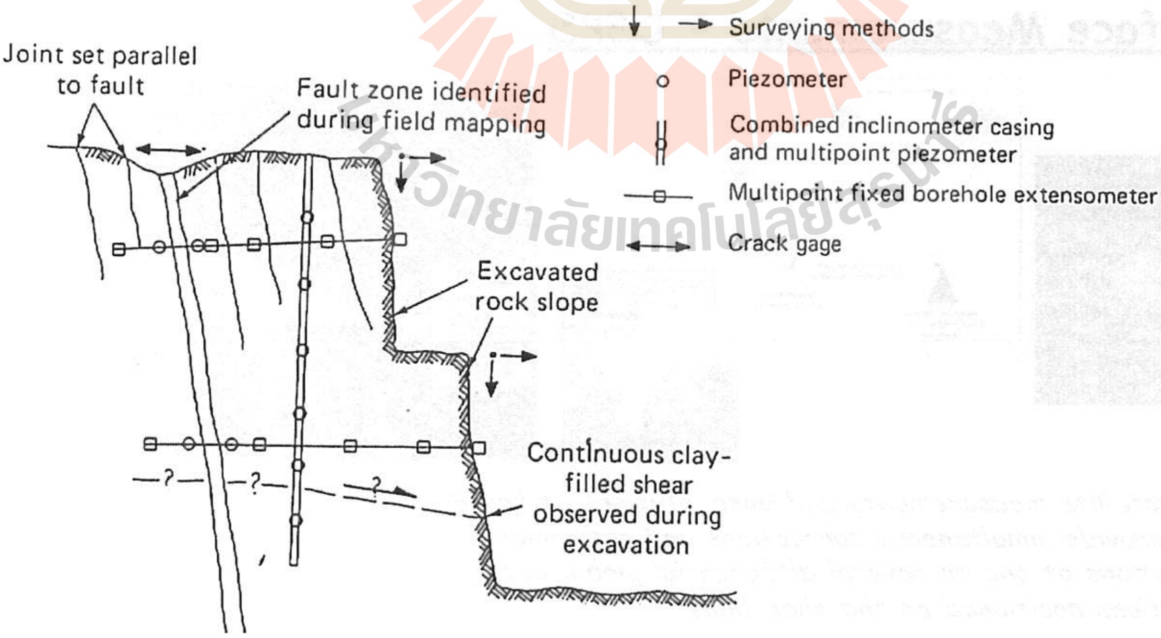
- Range
- Resolution
- Repeatability
- Accuracy
- Survivability

Accuracy and Precision.

Possible layout of instrumentation for monitoring an excavated slope in soil.



Possible layout of instrumentation for monitoring an excavated slope in rock.



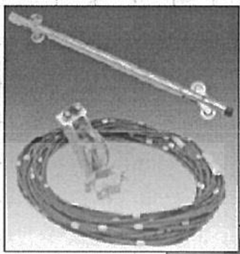
Instrumentation & Monitoring

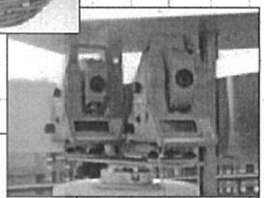
The use of geotechnical instrumentation is not merely the selection of instruments but a comprehensive step-by-step engineering process beginning with a definition of the objective and ending with implementation of the data.

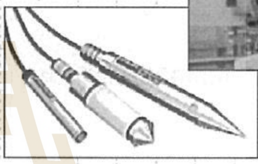
Engineering objectives typically encountered in soil and rock engineering projects have led to the design and commercial marketing of numerous instrument types, measuring for example:

decreasing reliability ↓

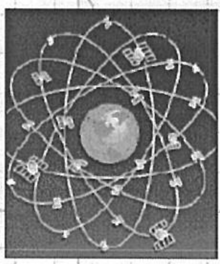
- *temperature*
- *deformation*
- *groundwater/pore pressures*
- *total stress in soil and stress change in rock*

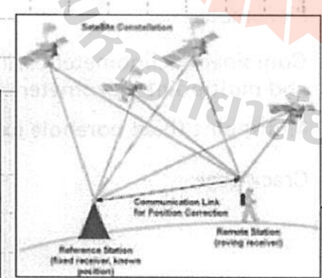


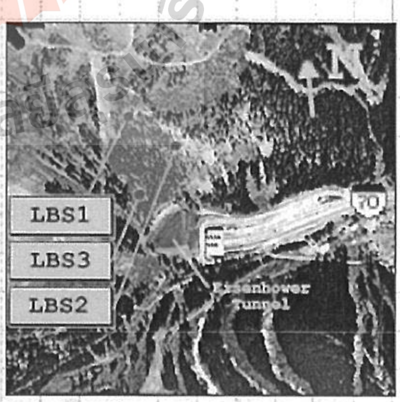




Surface Measurements - DGPS



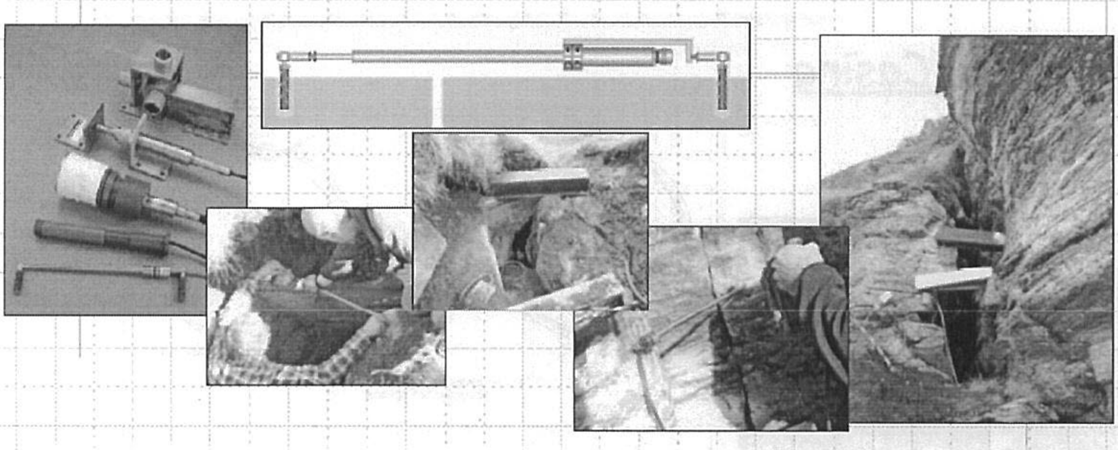




... satellite measurements and base-stations at known locations are used to provide simultaneous corrections and refinements to the computed locations of one or several differential global positioning system (DGPS) stations positioned on the slide body.

Advantages: automated, economical (especially over large areas).
Sensitivity: better than 1 cm in ideal conditions

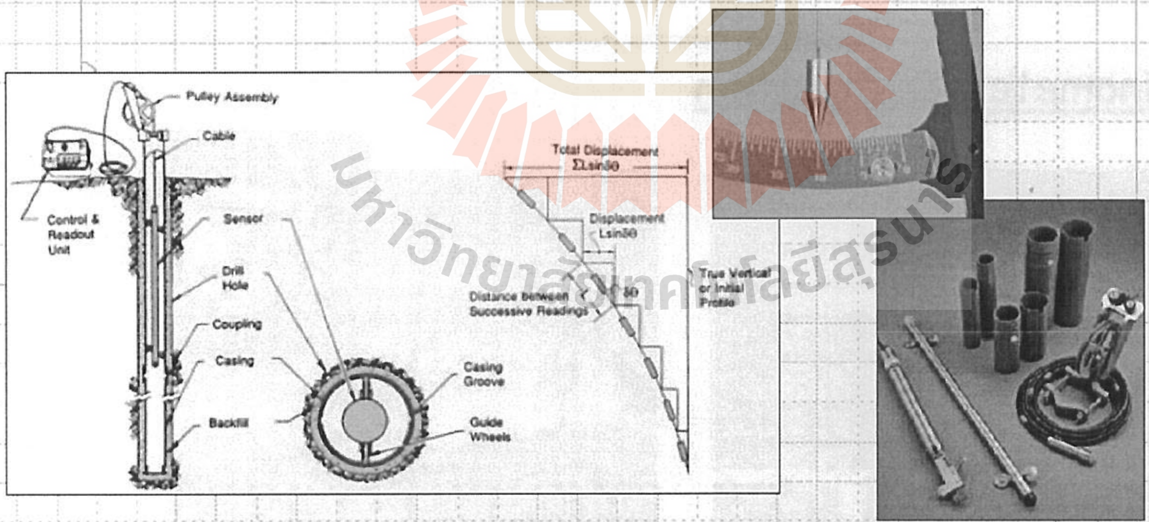
Surface Measurements - Crackmeters



... used to measure and monitor the opening of surface fractures and tension cracks.

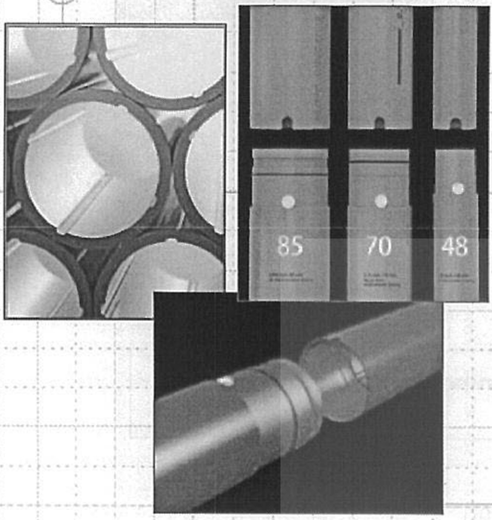
Advantages: simple, ideally suited for early warning systems.
Sensitivity: <0.01mm with 50-100 mm range

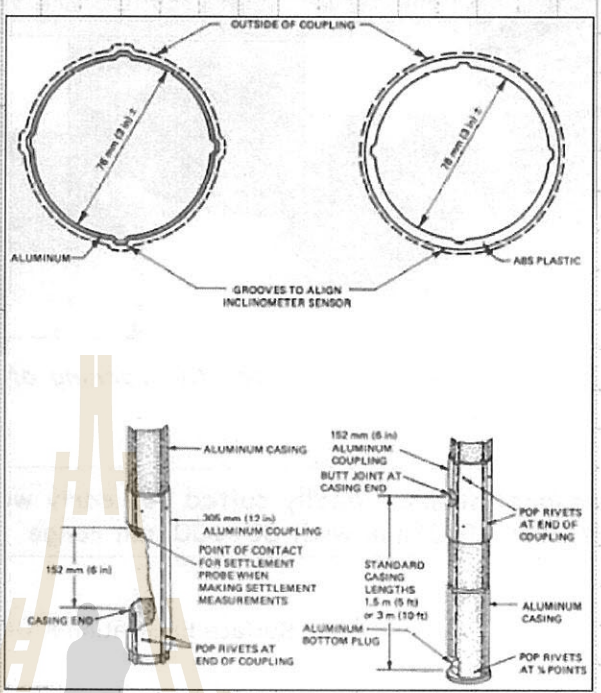
Subsurface Measurements - Inclinometers



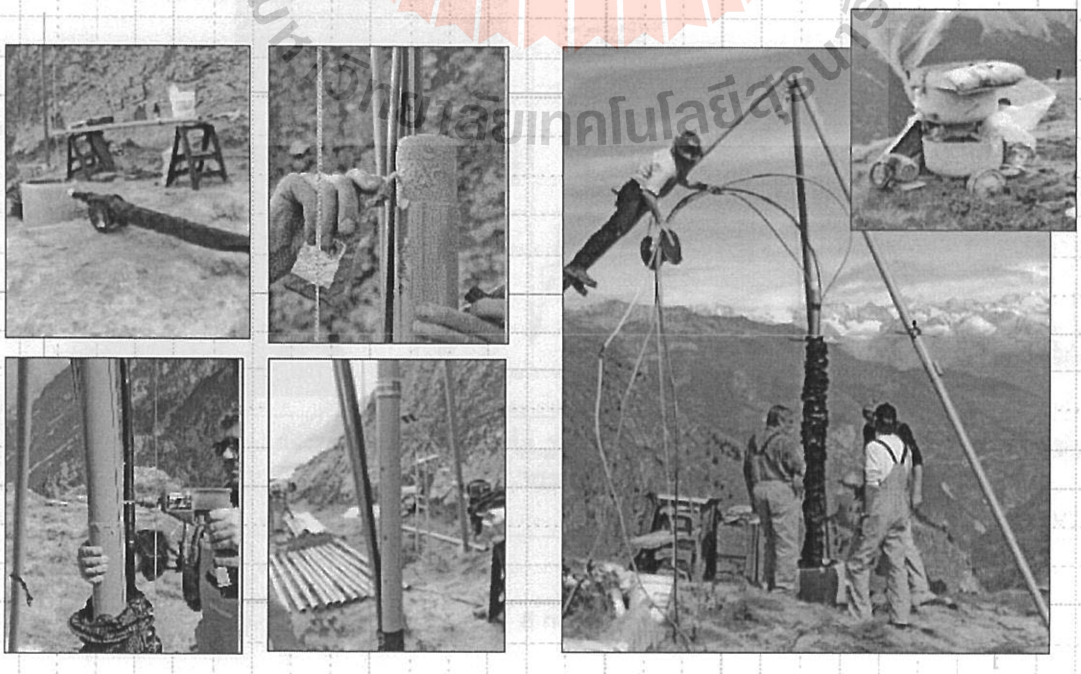
Advantages: can detect and monitor complex slope deformations and displacements along multiple shear planes.
Sensitivity: ±10 arc seconds (±0.05mm/m)

Inclinometer Casing

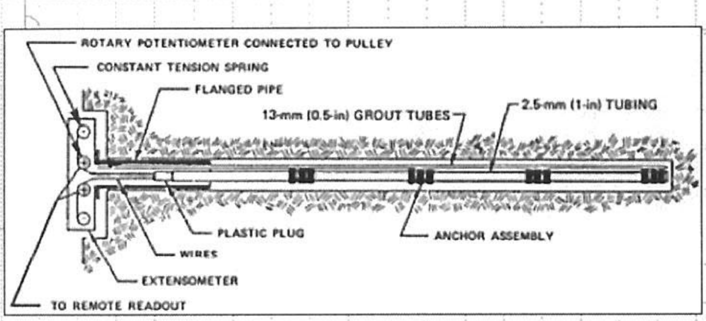




Inclinometer Installation



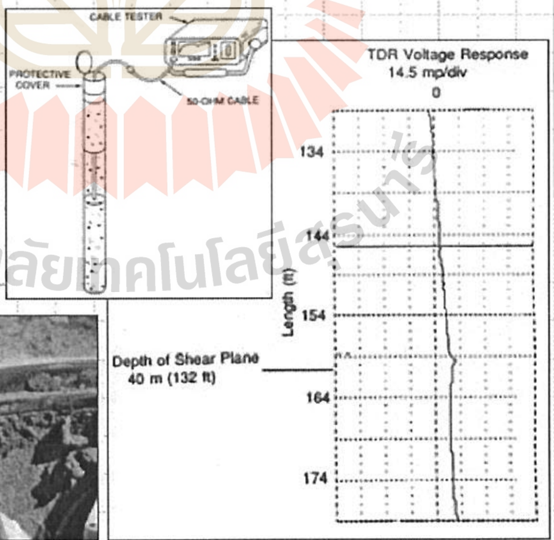
Subsurface Measurements - Extensometers



... extensometers measure the relative change in position between several fixed points.

Advantages: simple to install, inexpensive, can measure larger slope displacements than inclinometers.
Accuracy: ±0.01 mm/m

Subsurface Measurements - TDR



Dowding & Huang (1994)

Time Domain Reflectometry (TDR) - *uses characteristics of returned electrical pulses to determine the amount of strain, or the existence of a rupture, in a coaxial cable.*

Borehole Piezometers

Vibrating wire piezometers consist of a diaphragm, which when deflected by pore pressures, can be measured by an electrical transducer. These have the advantage of a negligible time lag and being extremely sensitive.