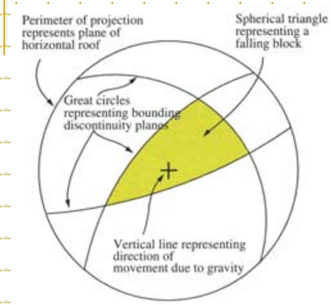


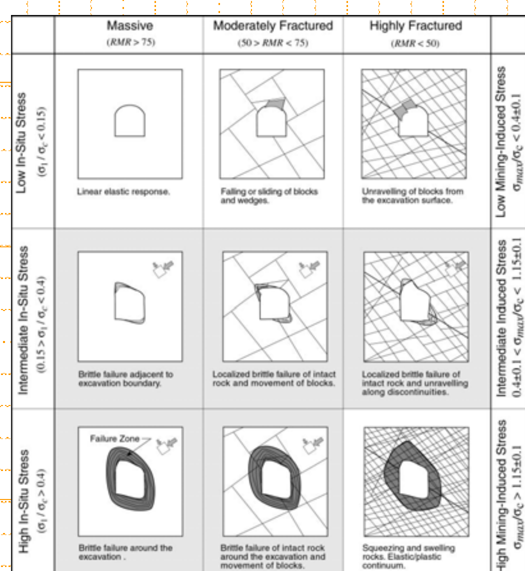
EOSC433:

Geotechnical Engineering Practice & Design

Lecture 4: Kinematic Analysis (Wedge Failure)

Underground Instability Mechanisms

When considering different rock mass failure mechanisms, we generally distinguish between those that are primarily structurally-controlled and those that are stress-controlled. Of course some failure modes are composites of these two conditions, and others may involve the effect of time and weathering on excavation stability.

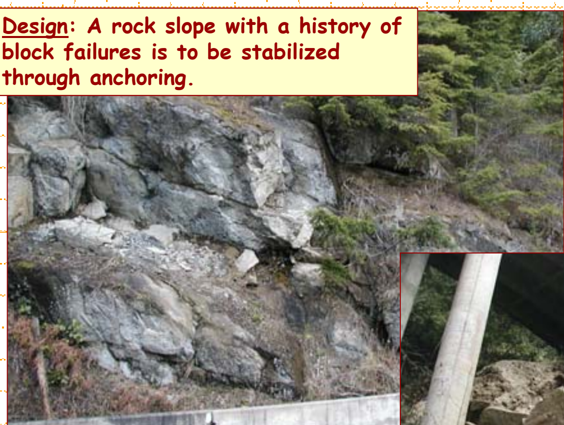


Martin *et al.* (1999)

Structurally-Controlled Instability Mechanisms

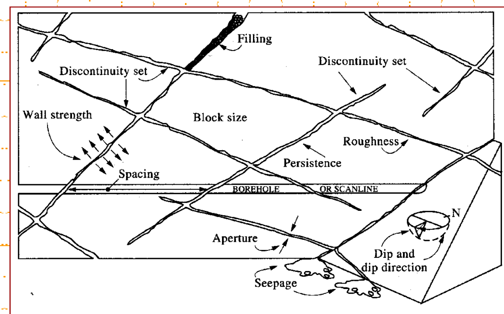
Design Challenge: Rock Slope Stabilization

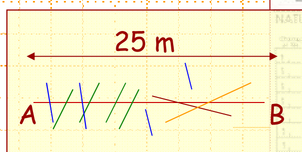
Design: A rock slope with a history of block failures is to be stabilized through anchoring.

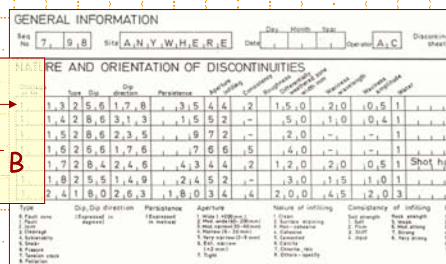


Rock Mass Characterization - Discontinuities

The main features of rock mass geometry include spacing and frequency, orientation (dip direction/dip angle), persistence (size and shape), roughness, aperture, clustering and block size.





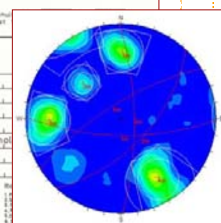

GENERAL INFORMATION

Seq No. 7, 9,8 Site A,N,Y,W,H,E,R,E Date (Day, Month, Year) Operator A,C Discontinuity sheet

NATURE AND ORIENTATION OF DISCONTINUITIES

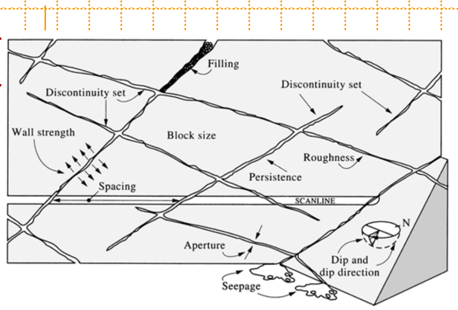
Chainage	Type	Dip	Dip direction	Persistence	Aperture	Infilling	Consistency	Roughness	Differentially weathered zone width mm	Waviness wavelength	Waviness amplitude	Water	
1,3	2	5,6	1,7,8	3,5	4	4	2		1,5,0	2,0	0,5	1	
1,4	2	8,6	3,1,3	1,5	5	2	-		5,0	1,0	0,4	1	
1,5	2	8,6	2,3,5	9	7	2	-		2,0	-	-	1	
1,6	2	6,6	1,7,6	7	6	6	5		4,0	-	-	1	
1,7	2	8,4	2,4,6	4,3	4	4	2		1,2,0	2,0	0,5	1	Shot hol
1,8	2	5,5	1,4,9	2,4	5	2	-		3,0	1,5	1,0	1	
2,4	1	8,0	2,6,3	1,8,0	3	4	4		2,0,0	4,5	2,0	3	

Type | Dip, Dip direction (Expressed in degrees) | Persistence (Expressed in metres) | Aperture | Nature of infilling | Consistency of infilling



Discontinuity Mapping

Hudson & Harrison (1997)

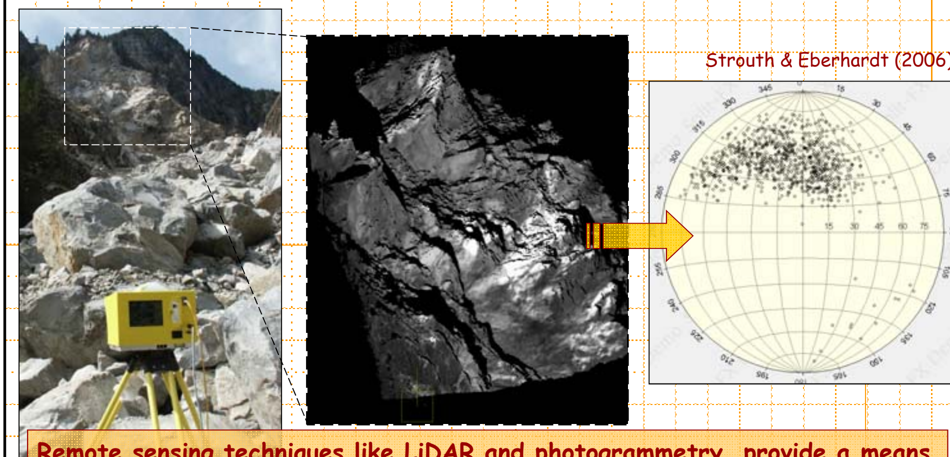


Scanline mapping

Window mapping

Discontinuity Mapping – Remote Sensing

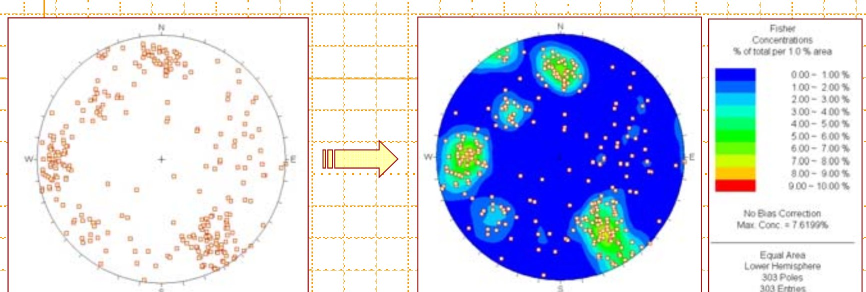
Strouth & Eberhardt (2006)



Remote sensing techniques like LiDAR and photogrammetry, provide a means to collect discontinuity data from a safe distance from rock faces that would otherwise be inaccessible or dangerous. Millions of high accuracy 3D data points are acquired in Cartesian space and processed, leading to robust estimates of joint dip, dip direction, spacing and persistence.

Stereonets – Pole Plots

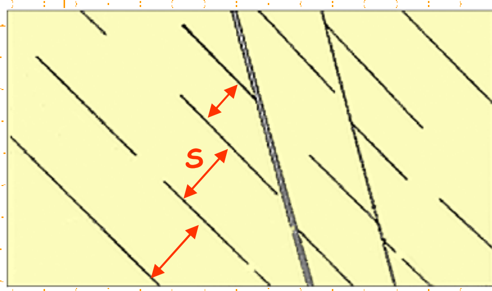
Plotting dip and dip direction, pole plots provide an immediate visual depiction of pole concentrations. All natural discontinuities have a certain variability in their orientation that results in scatter of the pole plots. However, by contouring the pole plot, the most highly concentrated areas of poles, representing the dominant discontinuity sets, can be identified.

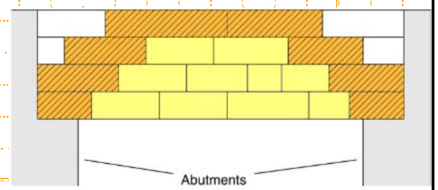


It must be remembered though, that it may be difficult to distinguish which set a particular discontinuity belongs to or that in some cases a single discontinuity may be the controlling factor as opposed to a set of discontinuities.

Discontinuity Spacing

Spacing is a key parameter in that it controls the block size distribution related to a potentially unstable roof or tunnel wall (i.e. kinematic failure of a wedge).

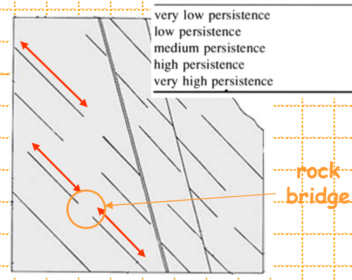


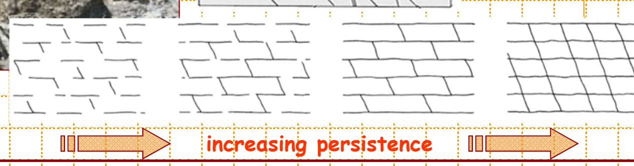


Discontinuity Persistence

Persistence refers to the areal extent or size of a discontinuity plane within a plane. Clearly, the persistence will have a major influence on the shear strength developed in the plane of the discontinuity, where the intact rock segments are referred to as 'rock bridges'.

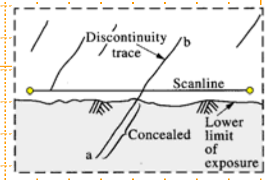
Description	Modal trace length (m)
very low persistence	< 1
low persistence	1–3
medium persistence	3–10
high persistence	10–20
very high persistence	20

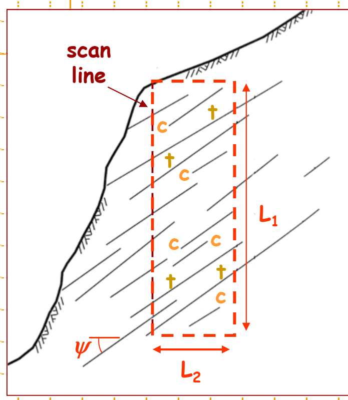




Discontinuity Persistence

Together with spacing, discontinuity persistence helps to define the size of blocks that can slide from a rock face. Several procedures have been developed to calculate persistence by measuring their exposed trace lengths on a specified area of the face.





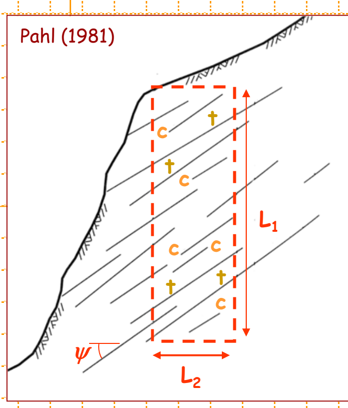
Step 1: define a mapping area on the rock face with dimensions L_1 and L_2.

Step 2: count the total number of discontinuities (N'') of a specific set with dip ψ in this area, and the numbers of these either contained within (N_c) or transecting (N_t) the mapping area defined.

For example, in this case:

$N'' = 14$
$N_c = 5$
$N_t = 4$

Discontinuity Persistence



Step 1: define a mapping area on the rock face with dimensions L_1 and L_2.

Step 2: count the total number of discontinuities (N'') of a specific set with dip ψ in this area, and the numbers of these either contained within (N_c) or transecting (N_t) the mapping area defined.

Step 3: calculate the approximate length, $\bar{l}$, of the discontinuities using the equations below.

$$m = \frac{(N_t - N_c)}{(N'' + 1)} \qquad H' = \frac{L_1 \cdot L_2}{(L_1 \cdot \cos\psi + L_2 \cdot \sin\psi)}$$

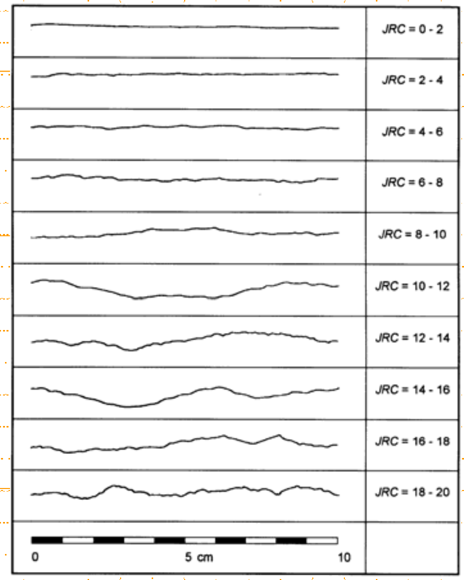
$$\bar{l} = H' \frac{(1+m)}{(1-m)}$$

Again, for this case:

If L_1 = 15 m, L_2 = 5 m and ψ = 35°, then H' = 4.95 m and m = -0.07. From this, the average length/persistence of the discontinuity set $\bar{l}$ = 4.3 m.

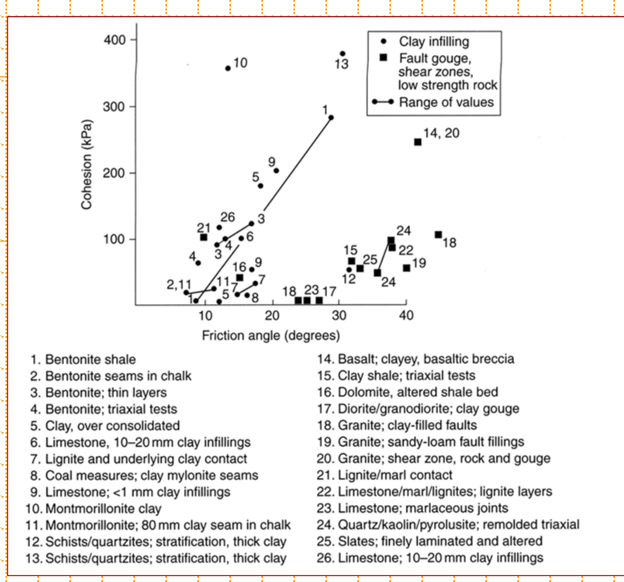
Discontinuity Roughness

From the practical point of view of quantifying joint roughness, only one technique has received some degree of universality – the Joint Roughness Coefficient (JRC). This method involves comparing discontinuity surface profiles to standard roughness curves assigned numerical values.



Barton & Choubey (1977)

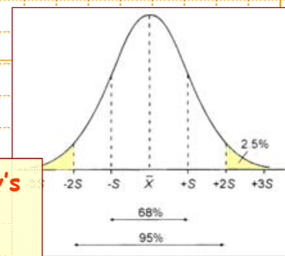
Mechanical Properties of Discontinuities



Wyllie & Mah (2004)

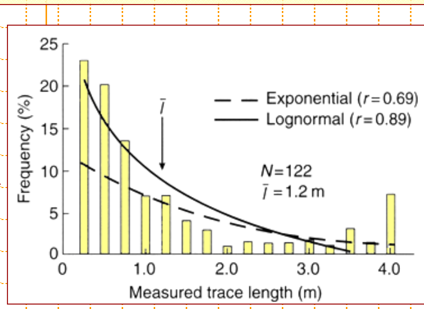
Discontinuity Data – Probability Distributions

Discontinuity properties can vary over a wide range, even for those belonging to the same set. The distribution of a property can be described by means of a probability distribution function.



A normal distribution is applicable where a property's mean value is the most commonly occurring. This is usually the case for dip and dip direction.

Wyllie & Mah (2004)

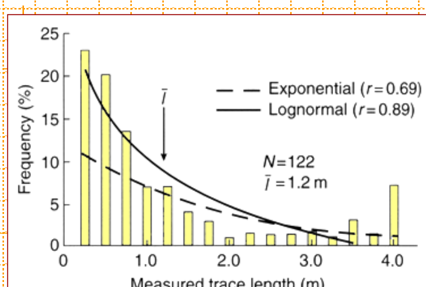


A negative exponential distribution is applicable for properties, such as spacing and persistence, which are randomly distributed.

Negative exponential function: $f(x) = \frac{1}{\bar{x}}\,(e^{-x/\bar{x}})$

Discontinuity Data - Probability Distributions

Wyllie & Mah (2004)



Negative exponential function: $f(x) = \frac{1}{\bar{x}}\,(e^{-x/\bar{x}})$

From this, the probability that a given value will be less than dimension *x* is given by: $F(x) = (1 - e^{-x/\bar{x}})$

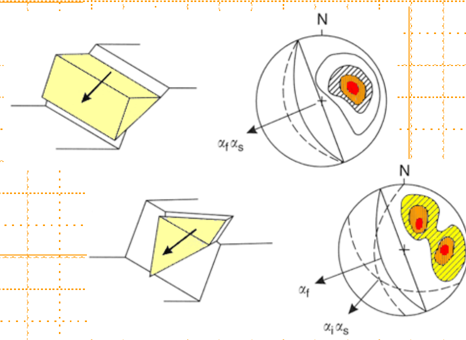
For example, for a discontinuity set with a mean spacing of 2 m, the probabilities that the spacing will be less than:

1 m ⇨ $F(x) = (1 - e^{-1/2}) = 40\%$

5 m ⇨ $F(x) = (1 - e^{-5/2}) = 92\%$

Structurally-Controlled Instability Mechanisms

Structurally-controlled instability means that blocks formed by discontinuities may be free to either fall or slide from the excavation periphery under a set of body forces (usually gravity). To assess the likelihood of such failures, an analysis of the kinematic admissibility of potential wedges or planes that intersect the excavation face(s) can be performed.



Kinematic Analysis – Planar Rock Slope Failure

To consider the kinematic admissibility of plane instability, five necessary but simple geometrical criteria must be met:

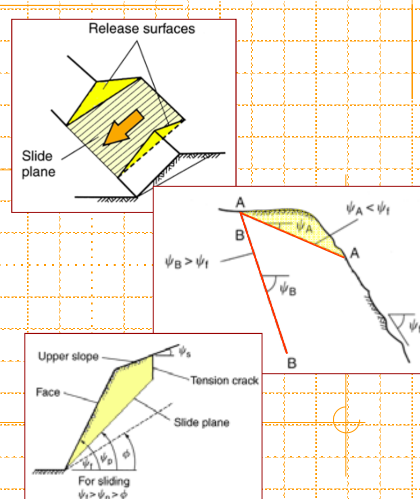
(i) The plane on which sliding occurs must strike near parallel to the slope face (within approx. ±20°).

(ii) Release surfaces (that provide negligible resistance to sliding) must be present to define the lateral slide boundaries.

(iii) The sliding plane must "daylight" in the slope face.

(iv) The dip of the sliding plane must be greater than the angle of friction.

(v) The upper end of the sliding surface either intersects the upper slope, or terminates in a tension crack.



Wyllie & Mah (2004)

Kinematic Analysis – Rock Slope Wedge Failure

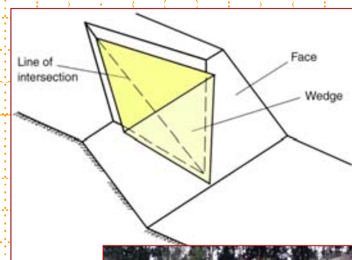
Similar to planar failures, several conditions relating to the line of intersection must be met for wedge failure to be kinematically admissible :

(i) The dip of the slope must exceed the dip of the line of intersection of the two wedge forming discontinuity planes.

(ii) The line of intersection must "daylight" on the slope face.

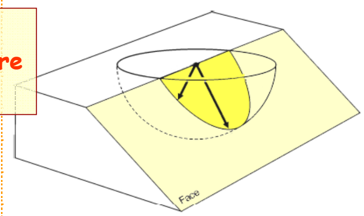
(iii) The dip of the line of intersection must be such that the strength of the two planes are reached.

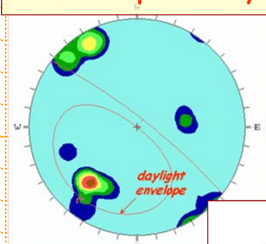
(iv) The upper end of the line of intersection either intersects the upper slope, or terminates in a tension crack.

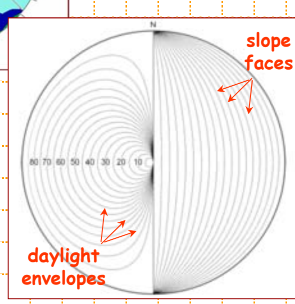


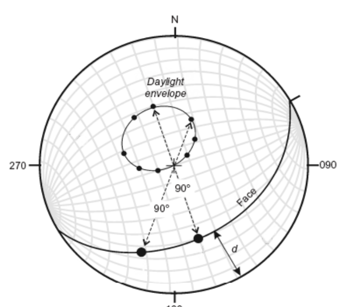
Kinematic Analysis – Daylight Envelopes

Daylight Envelope: Zone within which all poles belong to planes that daylight, and are therefore potentially unstable.





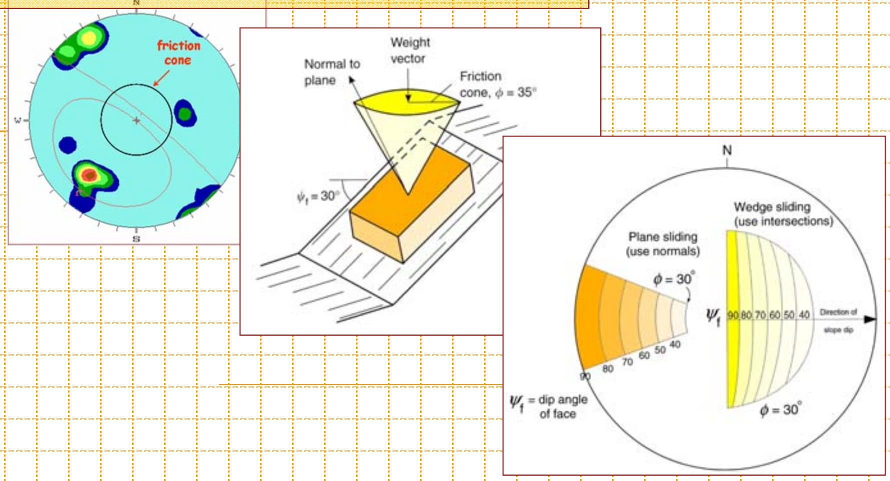




Lisle (2004)

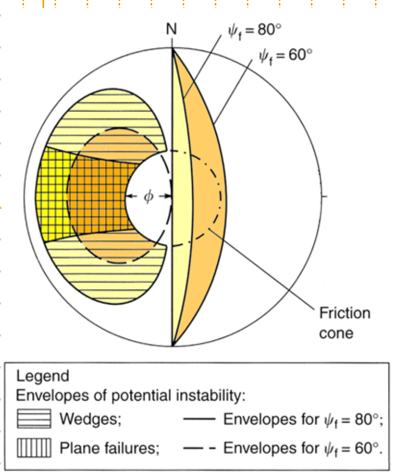
Kinematic Analysis – Friction Cones

Friction Cone: Zone within which all poles belong to planes that dip at angles less than the friction angle, and are therefore stable.

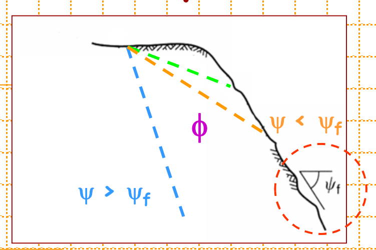


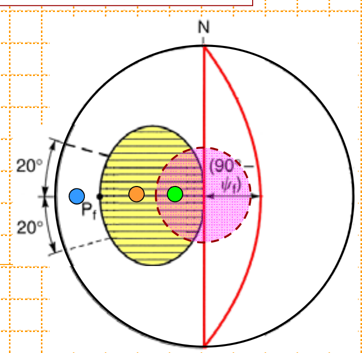
Harrison & Hudson (2000)

Pole Plots - Kinematic Admissibility

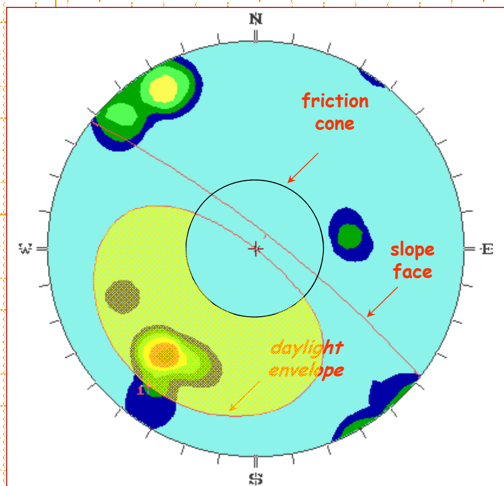


Wyllie & Mah (2004)





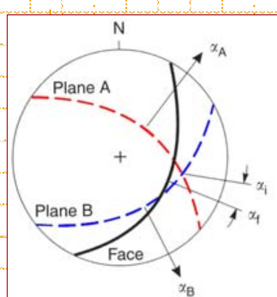
Pole Plots - Kinematic Admissibility



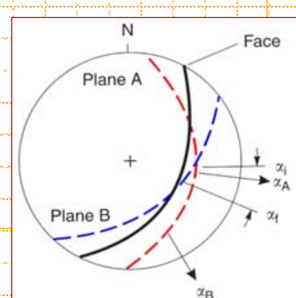
Having determined from the daylight envelope whether block failure is kinematically permissible, a check is then made to see if the dip angle of the failure surface (or line of intersection) is steeper than the with the friction angle.

Thus, for poles that plot **inside** the **daylight envelope**, but **outside** the **friction circle**, translational sliding is possible.

Wedge Failure – Direction of Sliding



Scenario #1: If the dip directions of the two planes lie outside the included angle between α_i (trend of the line of intersection) and α_f (dip direction of face), the wedge will slide on both planes.

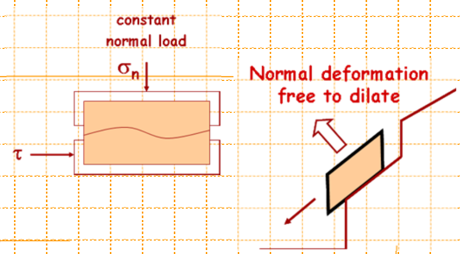


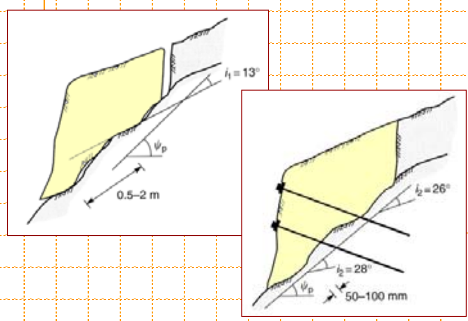
Wylie & Mah (2004)

Example scenario #2: If the dip directions of one plane (e.g. Plane A) lies within the included angle between α_i (trend of the line of intersection) and α_f (dip direction of face), the wedge will slide on only that plane.

Dilatancy and Shear Strength

In the case of sliding of an unconstrained block of rock from a slope, dilatancy will accompany shearing of all but the smoothest discontinuity surfaces. If a rock block is free to dilate, then the second-order asperities will have a diminished effect on shear strength.



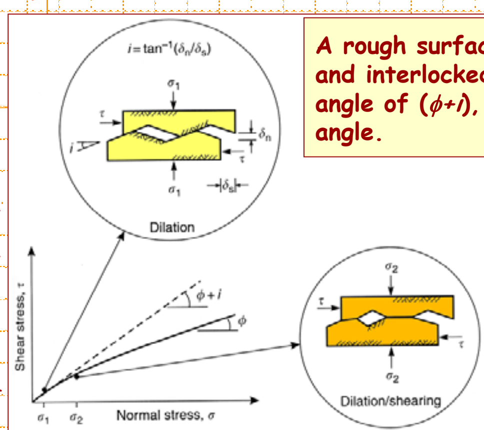


Wyllie & Mah (2004)

Thus, by increasing the normal force across a shear surface by adding tensioned rock bolts, dilation can be limited and interlocking along the sliding surface maintained, allowing the second-order asperities to contribute to the shear strength.

Discontinuity Shear Strength

Strength along a discontinuity surface is mostly provided by asperities. For shear failure to occur, the discontinuity surfaces must either dilate, allowing asperities to override one another, or shear through the asperities.



A rough surface that is initially undisturbed and interlocked will have a peak friction angle of (ϕ+*i*), where *i* is the roughness angle.

As stresses increase and shear displacements occur, the asperities will shear off, and the friction angle will progressively diminishe to a minimum value of the basic, or residual, friction angle of the rock.

Wyllie & Norrish (1996)

Discontinuity Shear Strength - Example

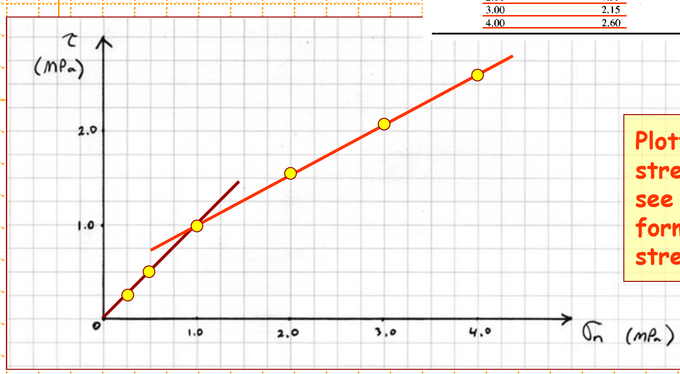
The following tests were obtained in a series of direct shear tests carried out on 100 mm square specimens of granite containing clean, rough, dry joints.

Normal stress σ_n (MPa)	Peak shear strength τ_p (MPa)	Residual shear strength τ_r (MPa)	Displacement at peak shear strength: Normal v(mm)	Displacement at peak shear strength: Shear u(mm)
0.25	0.25	0.15	0.54	2.00
0.50	0.50	0.30	0.67	2.50
1.00	1.00	0.60	0.65	3.20
2.00	1.55	1.15	0.45	3.60
3.00	2.15	1.70	0.30	4.00
4.00	2.60	–	0.15	4.20

Direct shear tests give normal and shear values which may be plotted directly.

Discontinuity Shear Strength - Example

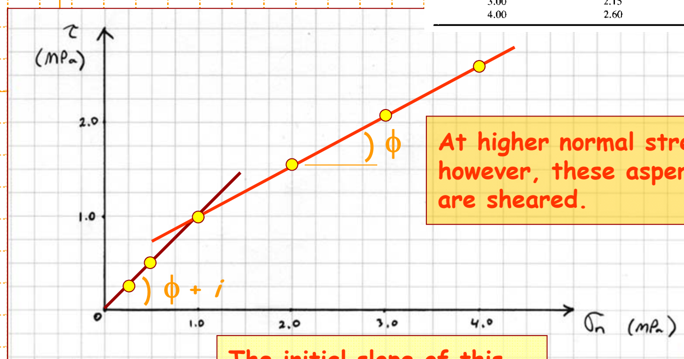
Normal stress σ_n (MPa)	Peak shear strength τ_p (MPa)	Residual shear strength τ_r (MPa)	Displacement at peak shear strength: Normal v(mm)	Displacement at peak shear strength: Shear u(mm)
0.25	0.25	0.15	0.54	2.00
0.50	0.50	0.30	0.67	2.50
1.00	1.00	0.60	0.65	3.20
2.00	1.55	1.15	0.45	3.60
3.00	2.15	1.70	0.30	4.00
4.00	2.60		0.15	4.20



Plotting the peak strength data we can see that it takes the form of a bilinear strength envelope.

Discontinuity Shear Strength - Example

Normal stress	Peak shear strength	Residual shear strength	Displacement at peak shear strength	
			Normal	Shear
σ_n (MPa)	τ_p (MPa)	τ_r (MPa)	v(mm)	u(mm)
0.25	0.25	0.15	0.54	2.00
0.50	0.50	0.30	0.67	2.50
1.00	1.00	0.60	0.65	3.20
2.00	1.55	1.15	0.45	3.60
3.00	2.15	1.70	0.30	4.00
4.00	2.60		0.15	4.20

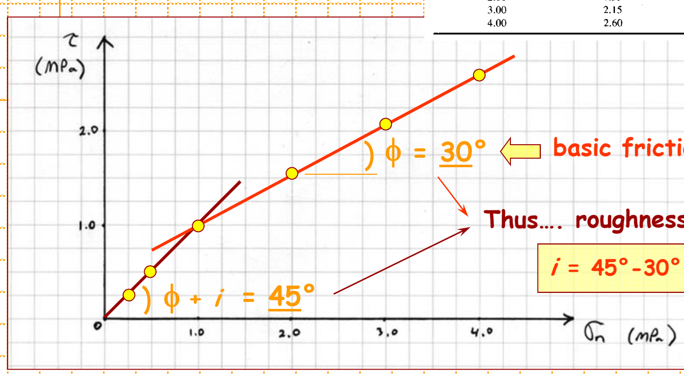


At higher normal stresses, however, these asperities are sheared.

The initial slope of this envelope has an apparent friction angle of (ϕ+i).

Discontinuity Shear Strength - Example

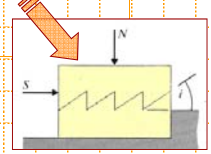
Normal stress	Peak shear strength	Residual shear strength	Displacement at peak shear strength	
			Normal	Shear
σ_n (MPa)	τ_p (MPa)	τ_r (MPa)	v(mm)	u(mm)
0.25	0.25	0.15	0.54	2.00
0.50	0.50	0.30	0.67	2.50
1.00	1.00	0.60	0.65	3.20
2.00	1.55	1.15	0.45	3.60
3.00	2.15	1.70	0.30	4.00
4.00	2.60		0.15	4.20



basic friction angle

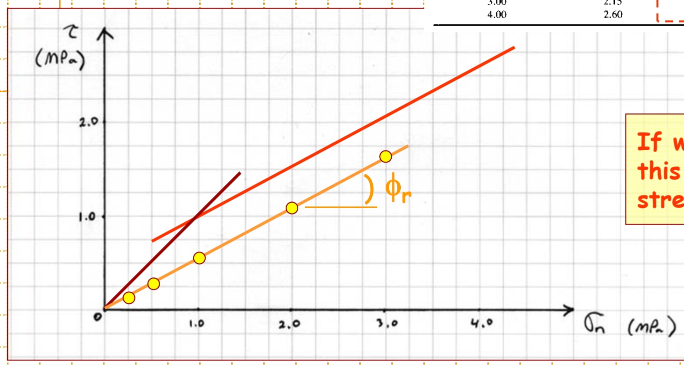
Thus…. roughness angle

i = 45°-30° = 15°



Discontinuity Shear Strength - Example

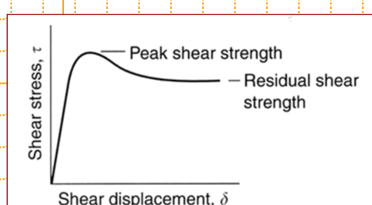
Normal stress σ_n (MPa)	Peak shear strength τ_p (MPa)	Residual shear strength τ_r (MPa)	Displacement at peak shear strength: Normal v(mm)	Displacement at peak shear strength: Shear u(mm)
0.25	0.25	0.15	0.54	2.00
0.50	0.50	0.30	0.67	2.50
1.00	1.00	0.60	0.65	3.20
2.00	1.55	1.15	0.45	3.60
3.00	2.15	1.70	0.30	4.00
4.00	2.60		0.15	4.20

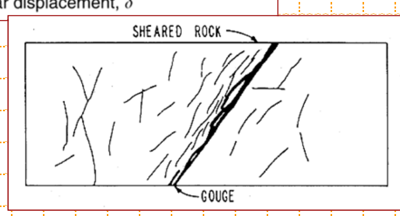


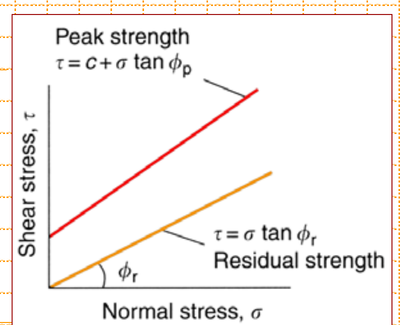
If we were to repeat this for the residual strength values...

Residual Strength

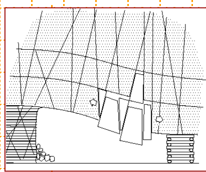
For the residual strength condition, any cohesion is lost once displacement has broken the cementing action. Also, the residual friction angle is less than the peak friction angle because the shear displacement grinds the minor irregularities on the rock surface and produces a smoother, lower friction surface.

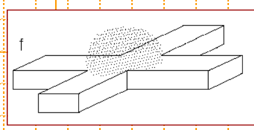


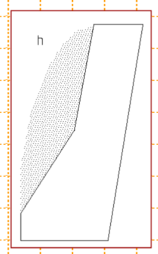


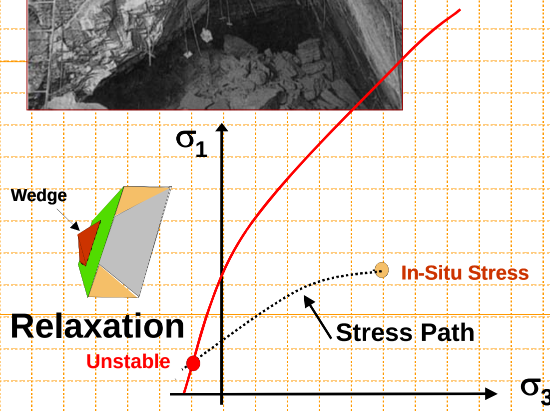


Underground Instability Mechanisms







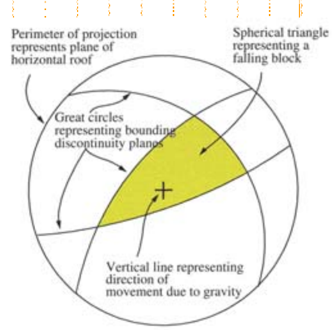


Kaiser et al. (2000)

Kinematic Analysis – Underground Wedges

The minimum requirement to define a potential wedge is four non-parallel planes; the excavation periphery forms one of these planes. On a hemispherical projection, these blocks may be identified as spherical triangles where the plane of projection represents the excavation surface.

If a tetrahedral block/wedge exists, there are three kinematic possibilities to be examined: the block falls from the roof; the block slides (either along the line of maximum dip of a discontinuity, or along the line of intersection of two discontinuities); or the block is stable.

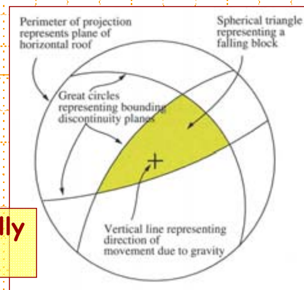


Hudson & Harrison (1997)

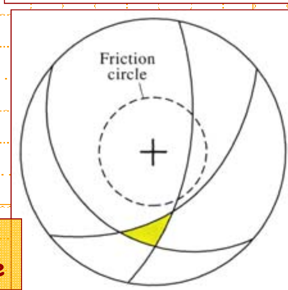
Analysis of Kinematic Admissibility - Falling

Falling occurs when a block detaches from the roof of an excavation without sliding on any of the bounding discontinuity planes. In the case of gravitational loading, the direction of movement is vertically downwards.

This is represented on the projection as a line with a dip of 90°, i.e. the centre of the projection. Thus, if this point falls within the spherical triangle formed by the bounding discontinuities, falling is kinematically admissible.



kinematically admissible



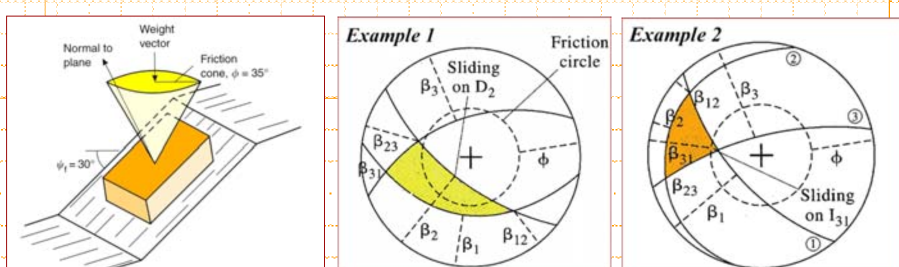
kinematically inadmissible = stable

Hudson & Harrison (1997)

Analysis of Kinematic Admissibility - Sliding

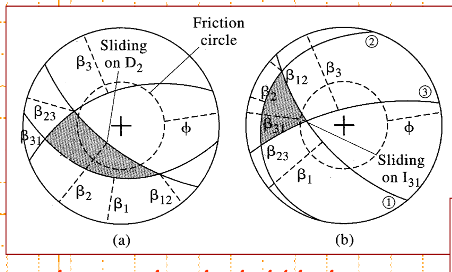
Kinematic analyses first assess whether sliding from the roof will occur along either a single discontinuity plane (planar failure) or a line of intersection (wedge failure). The analyses then considers whether these have a dip greater than the angle of friction.

Hudson & Harrison (1997)

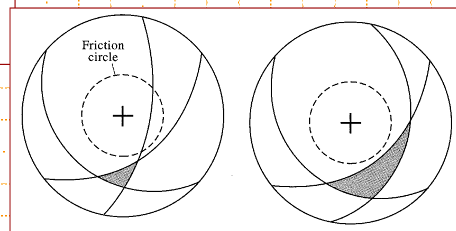


Assuming that each discontinuity plane has the same friction angle, the sliding direction will occur along a line of maximum dip (either that of a plane or a line of intersection of two planes). No other part of the spherical triangle represents a line of steeper dip than these candidates.

Analysis of Kinematic Admissibility - Sliding



… hence, the shaded blocks above represent (a) planar sliding along β_2; and (b) wedge sliding along β_{31}.



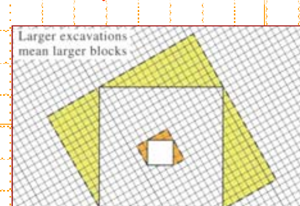
… of course, if the spherical triangles fall completely outside the friction circle, then the blocks are identified as being stable.

Geometrical Analysis of Maximum Wedge Volume

Once a series of joint sets have been identified as having wedge forming potential, several questions arise :

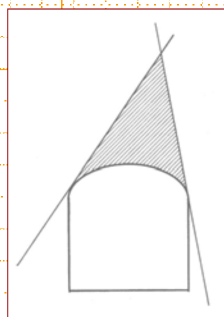
- ⇨ in the case of a falling wedge, how much support will be required to hold it in place (what kind of loads on the added support can be expected, how dense will the bolting pattern have to be, etc.);
- ⇨ in the case of a sliding wedge, do the shear stresses exceed the shear strength along the sliding surface, i.e. that provided by friction and sometimes cohesion (in the form of intact rock bridges or mineralized infilling), and if so, how much support will be required to stabilize the block, how dense will the bolting pattern have to be, etc..

In both cases, the volume/weight of the maximum wedge that may form is required. This can be determined through further geometrical constructions.



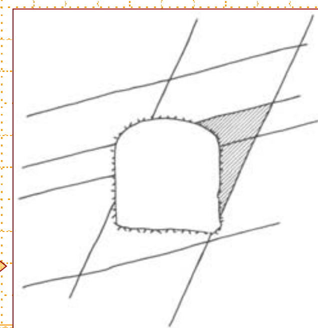
Maximum Wedge and Key Block Theory

Key block theory tries to build on wedge analysis by establishing a complete list of multiple blocks that may fail and a relative block failure likelihood distribution whose modes define the critical blocks.



… area of the maximum key-block for an underground opening.

… "maximum wedge" formed by multiple key-blocks intersecting an excavation.

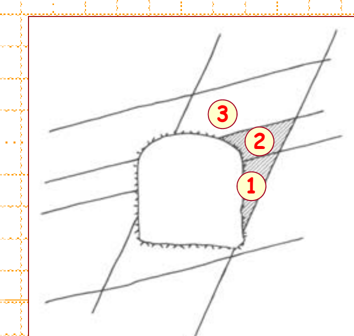


Goodman & Shi (1995)

Key Block Analysis

The underlying axiom of block theory is that the failure of an excavation begins at the boundary with the movement of a block into the excavated space. The loss of the first block augments the space, possibly creating an opportunity for the failure of additional blocks, with continuing degradation possibly leading to massive failure.

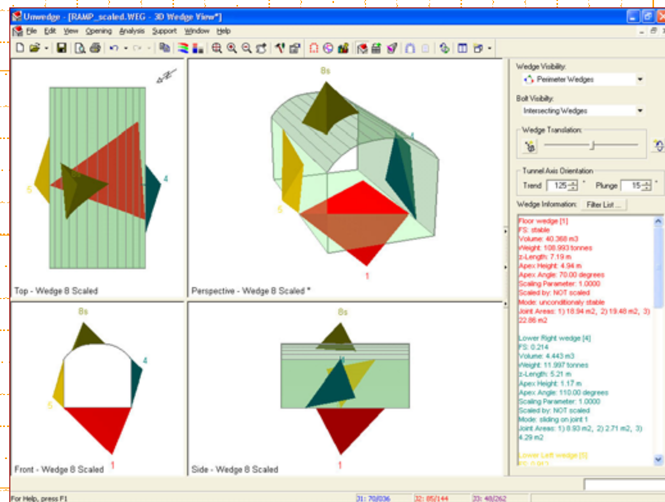
As such, the term key-block identifies any block that would become unstable when intersected by an excavation. The loss of a key-block does not necessarily assure subsequent block failures, but the prevention of its loss does assure stability.



Key-block theory therefore sets out to establish procedures for describing and locating key blocks and for establishing their support requirements.

Wedge Analysis – Computer-Aided

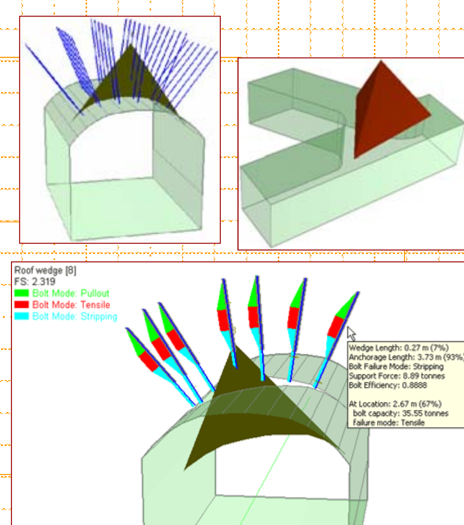
The 3-D nature of wedge problems (i.e. size and shape of potential wedges in the rock mass surrounding an opening) necessitates a set of relatively tedious calculations. While these can be performed by hand, it is far more efficient to utilise computer-based techniques.



(Rocscience – Unwedge)

Computer-Aided Wedge Analysis in Design

The speed of computer-aided wedge analyses allow them to be employed within the design methodology as a tool directed towards "filter analysis". This is carried out during the preliminary design to determine whether or not there are stability issues for a number of different problem configurations (e.g. a curving tunnel, different drifts in the development of an underground mine, etc.).



(Rocscience – Unwedge)

Case History: Rock Slope Stabilization

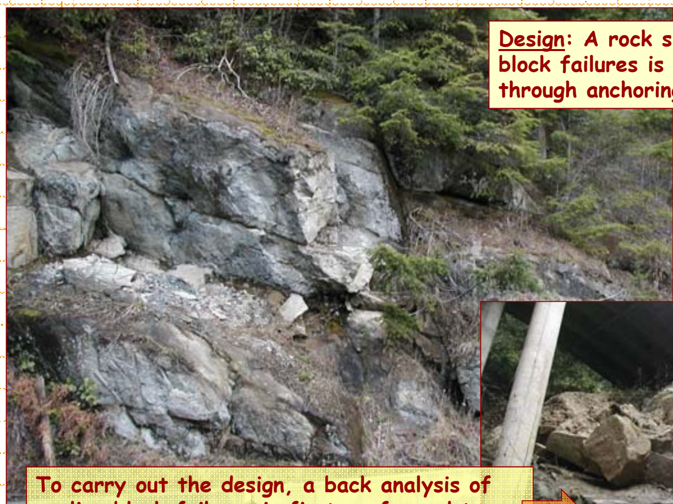
Project Details:

Existing bridge (to be replaced)
- built in 1947
- three-span; reinforced concrete
- structurally deficient
- large rock slide in 1998

New bridge
- 230-ft; two span steel bridge

Courtesy - B. Fisher (Kleinfelder Inc.)

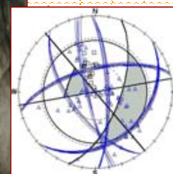
Case History: Rock Slope Stabilization



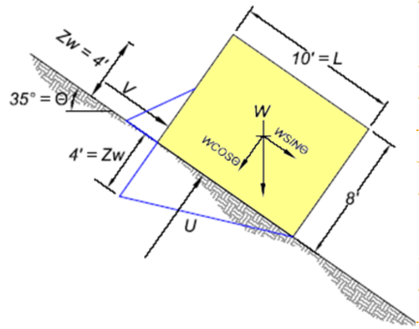
Design: A rock slope with a history of block failures is to be stabilized through anchoring.

To carry out the design, a back analysis of earlier block failures is first performed to obtain joint shear strength properties.

Case History: Rock Slope Stabilization

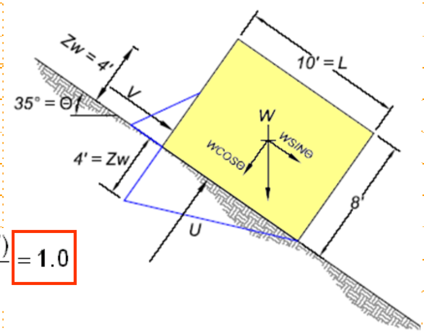


Courtesy - B. Fisher (Kleinfelder)



Case History: Rock Slope Stabilization

Assume: Water in tension crack @ 50% the tension crack height & water along discontinuity.



$$F = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{(W\cos\theta - U)\,Tan(\phi' + i)}{W\sin\theta + V} = 1.0$$

$$V = \frac{1}{2}\gamma_w z_w^{\ 2} = \frac{1}{2}(62.4pcf)(4)^2 = 0.5\ kips$$

$$U = \frac{1}{2}L\gamma_w z_w = \frac{1}{2}(10')(4')(62.4\,pcf) = 1.25\ kips$$

$$F = \frac{(12.8^K\ \cos 35° - 1.25^K)\,Tan(\phi'+i)}{12.8^K\ \sin 35° + 0.5^K} = \frac{9.24^K\ Tan(\phi'+i)}{7.84^K} = 1.0$$

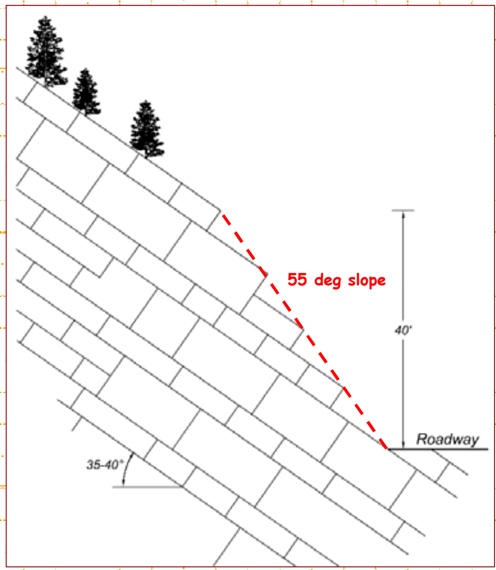
$$Tan(\phi'+i) = \frac{7.84^K}{9.24^K} = 0.748 \Rightarrow (\phi'+i) = 40.3° \text{ or } 40°$$

Case History: Rock Slope Stabilization

Given:

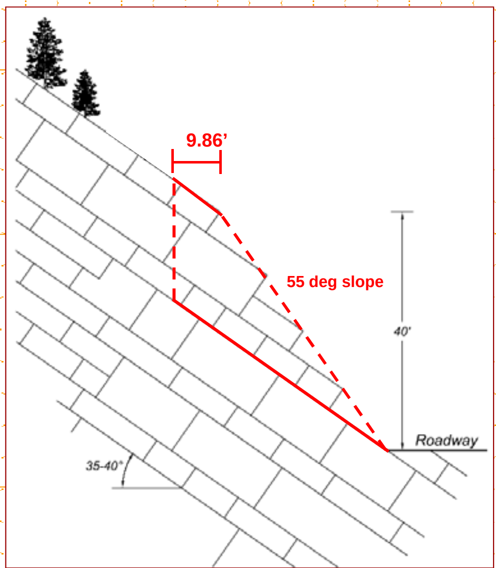
- Unstable Rock Slope
- 40 ft tall
- About 55 degrees
- Joint Set Dips 38 degrees
- $\phi' + i \sim$ 38 - 40 degrees

From previous back analysis of failed block below bridge abutment.



Case History: Rock Slope Stabilization

1. "Worst case" tension crack distance is 8.6 ft for a "dry" condition.
2. Assume 50% saturation for tension crack.
3. Estimate "super bolt" tension given desired bolt inclination.
4. Distribute "super bolt" tension over slope face based on available bolts.
5. Make sure and "bolt" all unstable blocks.



Courtesy - B. Fisher (Kleinfelder Inc.)

Case History: Rock Slope Stabilization

Results:

1. 22 kips tension/ft required at 5 deg downward angle for F = 1.5
2. Slope face length is equal to:

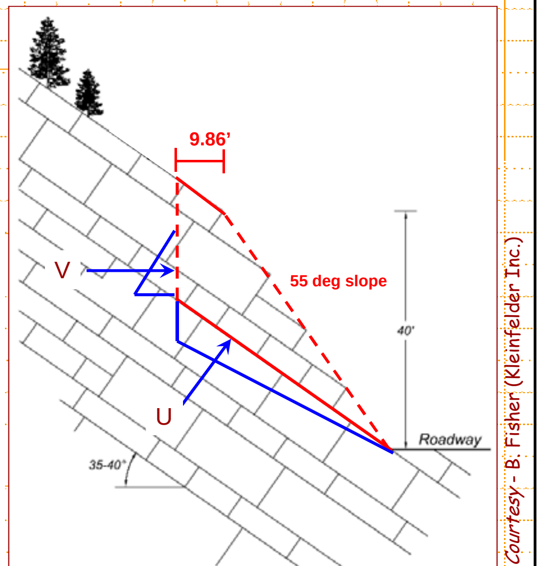
$$L_{Face} = \frac{(H)}{\cos\psi_f} = \left(\frac{40}{\cos 35}\right) = 48.83ft$$

$$22^K / 48.83\text{ft face} = 0.5ksf/\text{ft face}$$

$$25^K / 0.5ksf = 50\ ft^2$$

$$\sqrt{50}\ ft = 7.1\ ft \sim 7.0\ ft$$

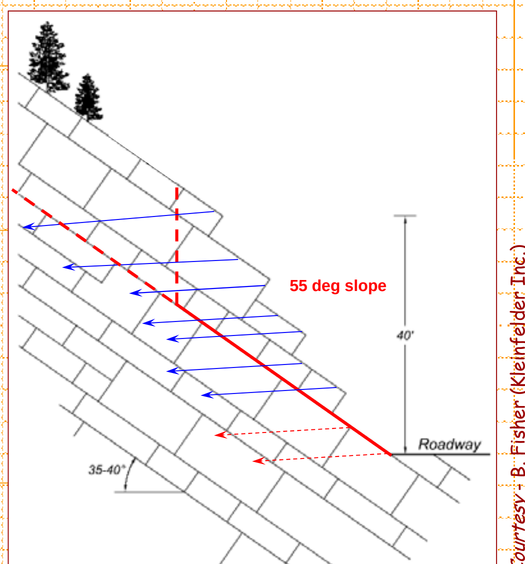
$$7.0\ ft(\cos 35) = 5.5\ ft\ \text{O.C. using elevation}$$



Case History: Rock Slope Stabilization

Recommendations:

1. 8 rows of bolts (40/5 = 8)
2. Try to bolt every block
3. Grout length determined by contractor
4. Rule of thumb, grout length; UCS/30 < 200 psi adhesion
5. Contractor responsible for testing or rock bolts
6. Engineer responsible to "sign off" on Contractors tests



Case History: Rock Slope Stabilization

Courtesy - B. Fisher (Kleinfelder Inc.)

Case History: Rock Slope Stabilization

Courtesy - B. Fisher (Kleinfelder Inc.)

Lecture References

Barton, NR & Choubey, V (1977). The shear strength of rock joints in theory and practice. *Rock Mechanics* **10**: 1-54.

Goodman, RE & Shi, GH (1995). Block theory and its application. *Géotechnique* **45**(3): 383-423.

Hoek, E, Kaiser, PK & Bawden, WF (1995). Support of Underground Excavations in Hard Rock. Balkema: Rotterdam.

Hudson, JA & Harrison, JP (1997). Engineering Rock Mechanics – An Introduction to the Principles . Elsevier Science: Oxford.

Kaiser, PK, Diederichs, MS, Martin, D, Sharpe, J & Steiner, W (2000). Underground works in hard rock tunnelling and mining. In *Proceedings, GeoEng2000, Melbourne*. Technomic Publishing: Lancaster, pp. 841-926.

Lisle, RJ (2004). Calculation of the daylight envelope for plane failure of rock slopes. *Géotechnique* **54**: 279-280.

Martin, CD, Kaiser, PK & McCreath, DR (1999). Hoek-Brown parameters for predicting the depth of brittle failure around tunnels. *Canadian Geotechnical Journal* **36**(1): 136-151.

Pahl, PJ (1981). Estimating the mean length of discontinuity traces. *International Journal of Rock Mechanics & Mining Sciences & Geomechanics Abstracts* **18**: 221-228.

Priest, SD (1985). Hemispherical Projection Methods in Rock Mechanics. George Allen & Unwin: London.

Lecture References

Strouth, A & Eberhardt, E (2006). The use of LiDAR to overcome rock slope hazard data collection challenges at Afternoon Creek, Washington. In *41st U.S. Symposium on Rock Mechanics: 50 Years of Rock Mechanics, Golden*. American Rock Mechanics Association, CD: 06-993.

Wyllie, DC & Mah, CW (2004). Rock Slope Engineering (4th edition). Spon Press: London.

Wyllie, DC & Norrish, NI (1996). Rock strength properties and their Measurement. In *Landslides: Investigation and Mitigation – Special Report 247*. National Academy Press: Washington, D.C., pp. 372-390.