

EOSC433/536:

**Geological Engineering
Practice I - Rock Engineering**

Lecture 4: Kinematic Analysis (Wedge Failure)



1 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Problem Set #1 - Debriefing

Learning Goals:

1. Yes, review of stress and strain... but also...
2. Problem solving using multiple resources
3. Notation and conventions
4. Problem solving verification








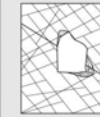


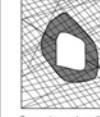
2 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.

	Massive ($RMR > 75$)	Moderately Fractured ($50 > RMR > 75$)	Highly Fractured ($RMR < 50$)	
Low In-Situ Stress ($\sigma_1 / \sigma_3 < 0.15$)	 Linear elastic response.	 Falling or sliding of blocks and wedges.	 Unravelling of blocks from the excavation surface.	Low Mining-Induced Stress $\sigma_{max}/\sigma_3 < 0.4 \pm 0.1$
Intermediate In-Situ Stress ($0.15 > \sigma_1 / \sigma_3 > 0.4$)	 Brittle failure adjacent to excavation boundary.	 Localized brittle failure of intact rock and movement of blocks.	 Localized brittle failure of intact rock and unravelling along discontinuities.	Intermediate Induced Stress $0.4 \pm 0.1 < \sigma_{max}/\sigma_3 < 1.15 \pm 0.1$
High In-Situ Stress ($\sigma_1 / \sigma_3 > 0.4$)	 Brittle failure around the excavation.	 Brittle failure of intact rock around the excavation and movement of blocks.	 Squeezing and swelling rocks. Elastic-plastic continuum.	High Mining-Induced Stress $\sigma_{max}/\sigma_3 > 1.15 \pm 0.1$

Martin et al. (1999)



3 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Wedge Failure & Rockbolting



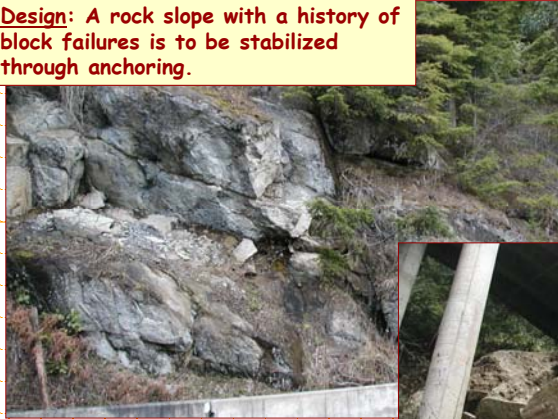
4 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Design Challenge: Rock Slope Stabilization

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



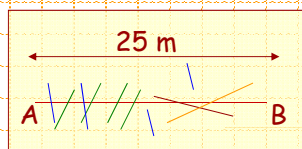
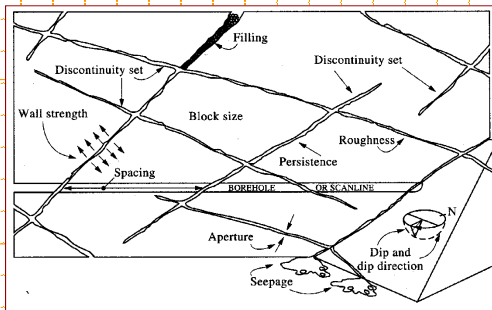
5 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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									
Site No.	7	S.B.	Site	A.N.Y.W.H.E.R.E.	Dist.	km	Scale	1:1000	Discontinuity date
Sheet No.	1	2	Drawn	A.C.	Sheet No.	2	of	1, 2	
NATURE AND ORIENTATION OF DISCONTINUITIES									
Change No.	Set	Dip direction	Dip angle	Persistence	Block size	Wall strength	Spacing	Roughness	Water
1	1	13	2	5	6	1	7	8	13
2	1	4	2	8	6	3	1	3	5
3	1	5	2	8	6	2	3	5	9
4	1	6	2	8	6	1	7	6	15
5	1	7	2	8	4	2	4	6	4
6	1	8	2	5	5	1	4	9	2
7	1	2	4	1	8	0	2	6	3
8	1	3	2	5	6	1	7	8	13
9	1	4	2	8	6	3	1	3	5
10	1	5	2	8	6	2	3	5	9
11	1	6	2	8	6	1	7	6	15
12	1	7	2	8	4	2	4	6	4
13	1	8	2	5	5	1	4	9	2
14	1	2	4	1	8	0	2	6	3
15	1	3	2	5	6	1	7	8	13
16	1	4	2	8	6	3	1	3	5
17	1	5	2	8	6	2	3	5	9
18	1	6	2	8	6	1	7	6	15
19	1	7	2	8	4	2	4	6	4
20	1	8	2	5	5	1	4	9	2
21	1	2	4	1	8	0	2	6	3
22	1	3	2	5	6	1	7	8	13
23	1	4	2	8	6	3	1	3	5
24	1	5	2	8	6	2	3	5	9
25	1	6	2	8	6	1	7	6	15
26	1	7	2	8	4	2	4	6	4
27	1	8	2	5	5	1	4	9	2
28	1	2	4	1	8	0	2	6	3
29	1	3	2	5	6	1	7	8	13
30	1	4	2	8	6	3	1	3	5
31	1	5	2	8	6	2	3	5	9
32	1	6	2	8	6	1	7	6	15
33	1	7	2	8	4	2	4	6	4
34	1	8	2	5	5	1	4	9	2
35	1	2	4	1	8	0	2	6	3
36	1	3	2	5	6	1	7	8	13
37	1	4	2	8	6	3	1	3	5
38	1	5	2	8	6	2	3	5	9
39	1	6	2	8	6	1	7	6	15
40	1	7	2	8	4	2	4	6	4
41	1	8	2	5	5	1	4	9	2
42	1	2	4	1	8	0	2	6	3
43	1	3	2	5	6	1	7	8	13
44	1	4	2	8	6	3	1	3	5
45	1	5	2	8	6	2	3	5	9
46	1	6	2	8	6	1	7	6	15
47	1	7	2	8	4	2	4	6	4
48	1	8	2	5	5	1	4	9	2
49	1	2	4	1	8	0	2	6	3
50	1	3	2	5	6	1	7	8	13
51	1	4	2	8	6	3	1	3	5
52	1	5	2	8	6	2	3	5	9
53	1	6	2	8	6	1	7	6	15
54	1	7	2	8	4	2	4	6	4
55	1	8	2	5	5	1	4	9	2
56	1	2	4	1	8	0	2	6	3
57	1	3	2	5	6	1	7	8	13
58	1	4	2	8	6	3	1	3	5
59	1	5	2	8	6	2	3	5	9
60	1	6	2	8	6	1	7	6	15
61	1	7	2	8	4	2	4	6	4
62	1	8	2	5	5	1	4	9	2
63	1	2	4	1	8	0	2	6	3
64	1	3	2	5	6	1	7	8	13
65	1	4	2	8	6	3	1	3	5
66	1	5	2	8	6	2	3	5	9
67	1	6	2	8	6	1	7	6	15
68	1	7	2	8	4	2	4	6	4
69	1	8	2	5	5	1	4	9	2
70	1	2	4	1	8	0	2	6	3
71	1	3	2	5	6	1	7	8	13
72	1	4	2	8	6	3	1	3	5
73	1	5	2	8	6	2	3	5	9
74	1	6	2	8	6	1	7	6	15
75	1	7	2	8	4	2	4	6	4
76	1	8	2	5	5	1	4	9	2
77	1	2	4	1	8	0	2	6	3
78	1	3	2	5	6	1	7	8	13
79	1	4	2	8	6	3	1	3	5
80	1	5	2	8	6	2	3	5	9
81	1	6	2	8	6	1	7	6	15
82	1	7	2	8	4	2	4	6	4
83	1	8	2	5	5	1	4	9	2
84	1	2	4	1	8	0	2	6	3
85	1	3	2	5	6	1	7	8	13
86	1	4	2	8	6	3	1	3	5
87	1	5	2	8	6	2	3	5	9
88	1	6	2	8	6	1	7	6	15
89	1	7	2	8	4	2	4	6	4
90	1	8	2	5	5	1	4	9	2
91	1	2	4	1	8	0	2	6	3
92	1	3	2	5	6	1	7	8	13
93	1	4	2	8	6	3	1	3	5
94	1	5	2	8	6	2	3	5	9
95	1	6	2	8	6	1	7	6	15
96	1	7	2	8	4	2	4	6	4
97	1	8	2	5	5	1	4	9	2
98	1	2	4	1	8	0	2	6	3
99	1	3	2	5	6	1	7	8	13
100	1	4	2	8	6	3	1	3	5



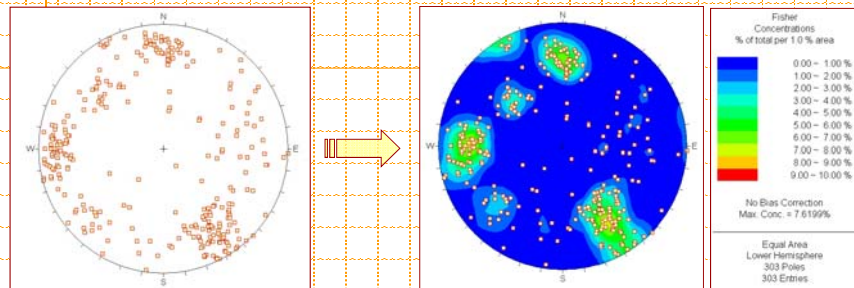
6 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.



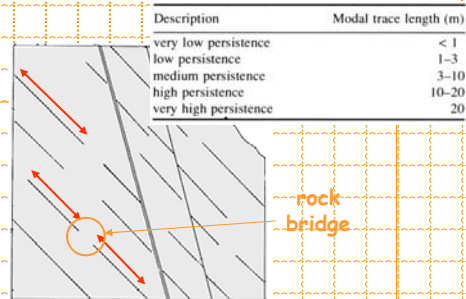
7 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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'.



increasing persistence



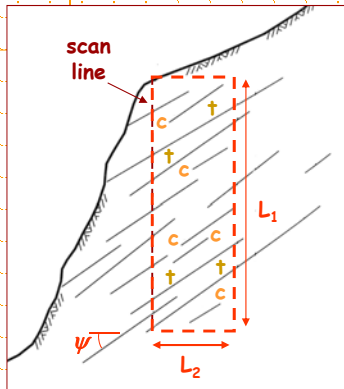
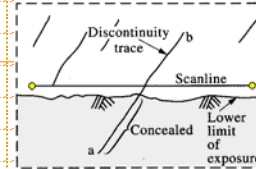
8 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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:

$$N'' = 14$$

$$N_c = 5$$

$$N_t = 4$$



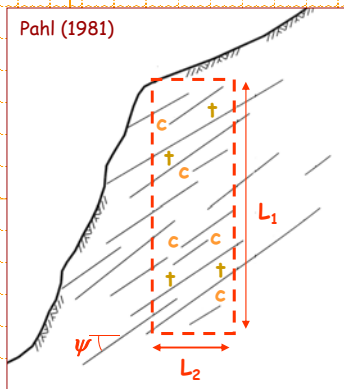
9 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Discontinuity Persistence

Pahl (1981)



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.

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

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

Again, for this case:

If $L_1 = 15$ m, $L_2 = 5$ m and $\psi = 35^\circ$, then $H' = 4.95$ m and $m = -0.07$.

From this, the average length/persistence of the discontinuity set $\bar{l} = 4.3$ m.



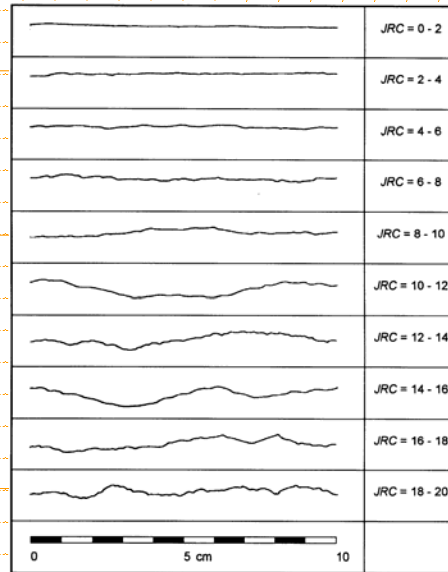
10 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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)

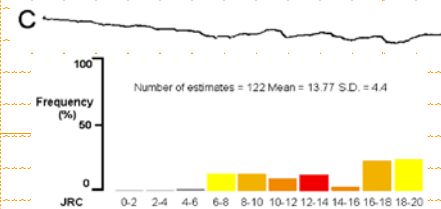
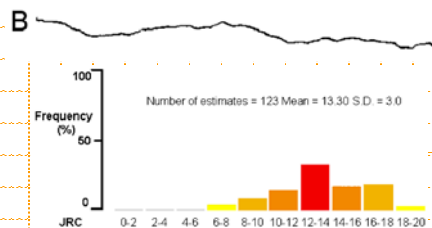
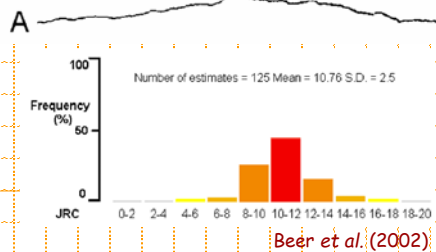
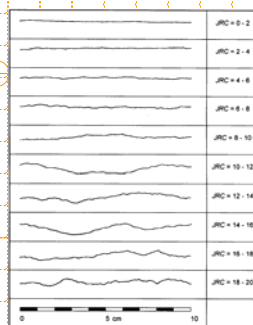


11 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Subjectivity in Joint Roughness

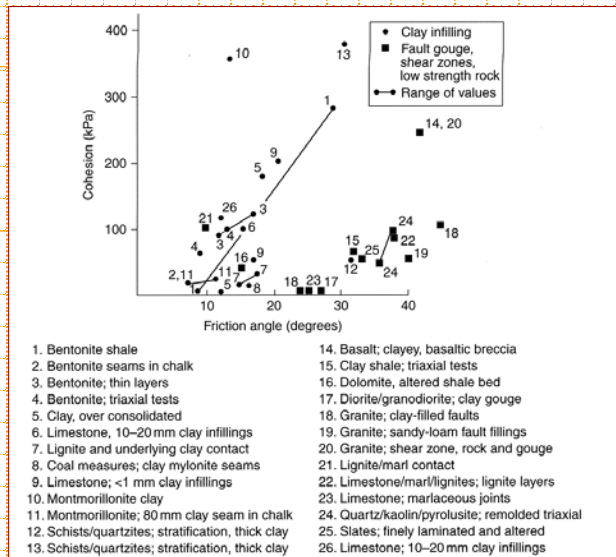


12 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Mechanical Properties of Discontinuities



Wyllie & Mah (2004)



13 of 57

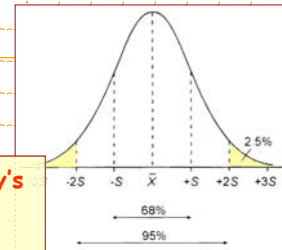
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

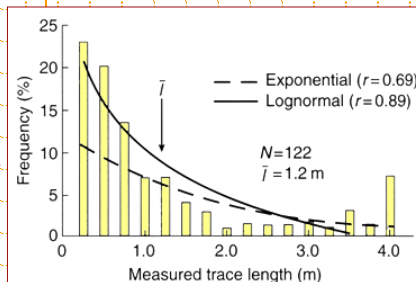
Discontinuity Data - Probability Distributions

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:

$$f(x) = \frac{1}{\bar{x}} (e^{-x/\bar{x}})$$



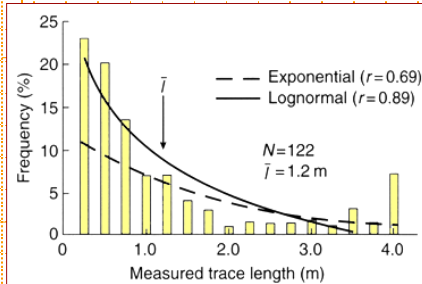
14 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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 $\Rightarrow F(x) = (1 - e^{-1/2}) = 40\%$

5 m $\Rightarrow F(x) = (1 - e^{-5/2}) = 92\%$



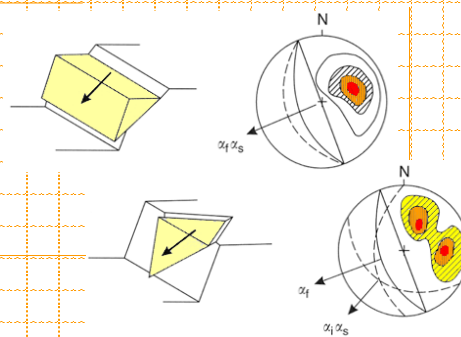
15 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.



16 of 57

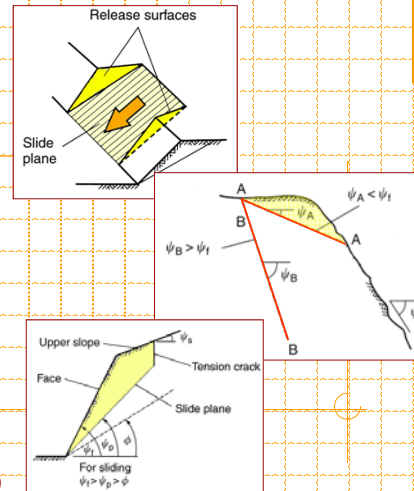
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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. $\pm 20^\circ$).
- (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)



17 of 57

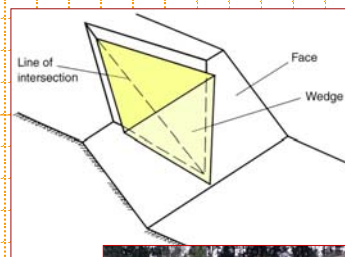
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.



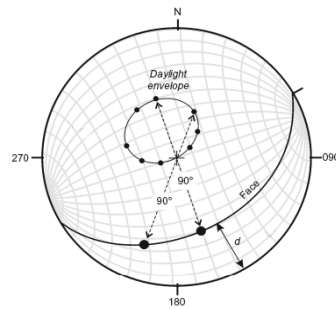
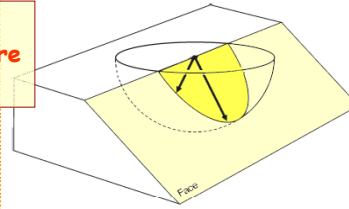
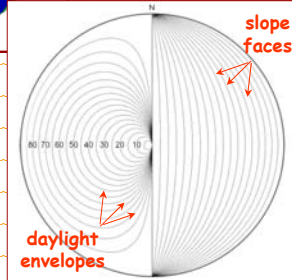
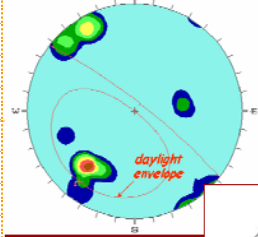
18 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Kinematic Analysis - Daylight Envelopes

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



Lisle (2004)



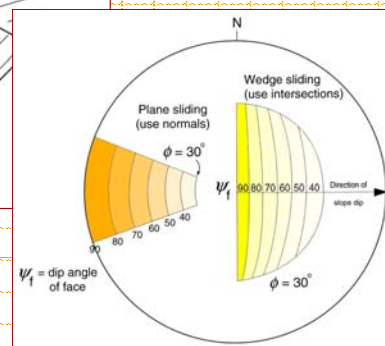
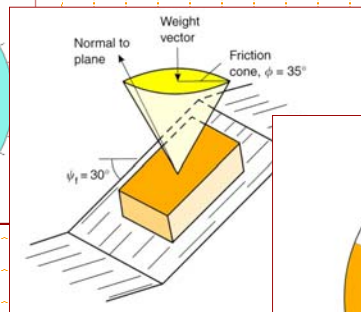
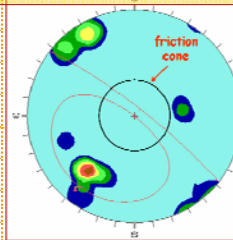
19 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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)

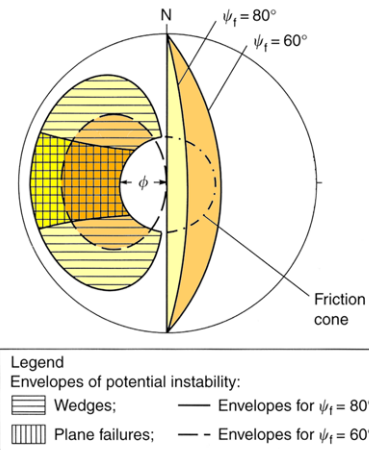


20 of 57

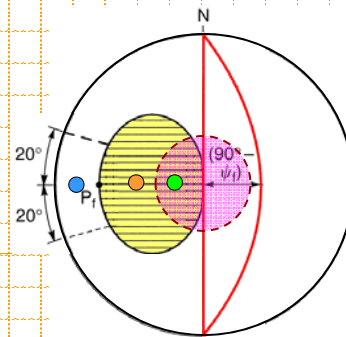
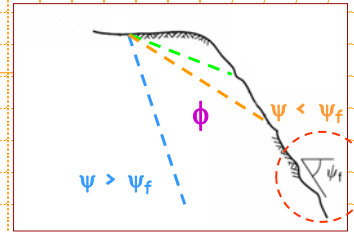
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Pole Plots - Kinematic Admissibility



Wyllie & Mah (2004)

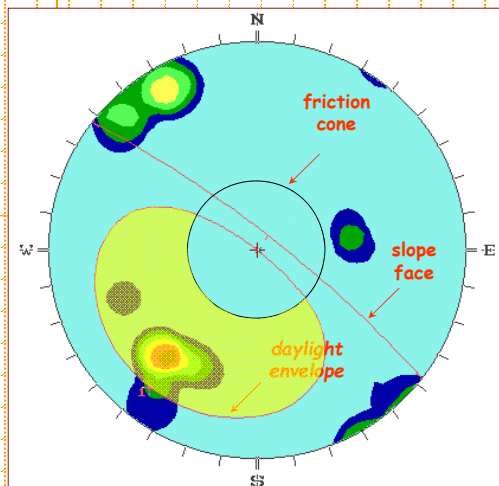


21 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.

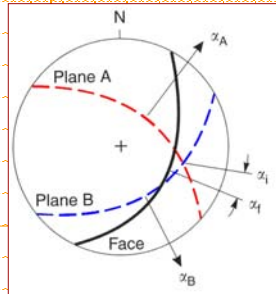


22 of 57

Erik Eberhardt - UBC Geological Engineering

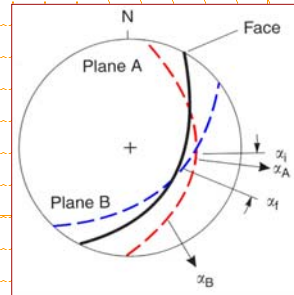
EOSC 433 (2017)

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.

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.



Wyllie & Mah (2004)



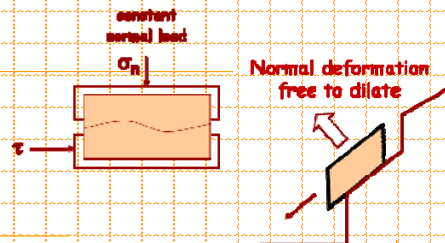
23 of 57

Erik Eberhardt - UBC Geological Engineering

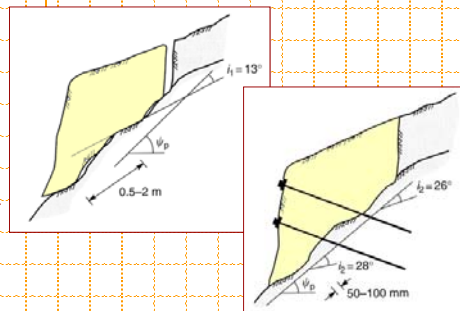
EOSC 433 (2017)

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.



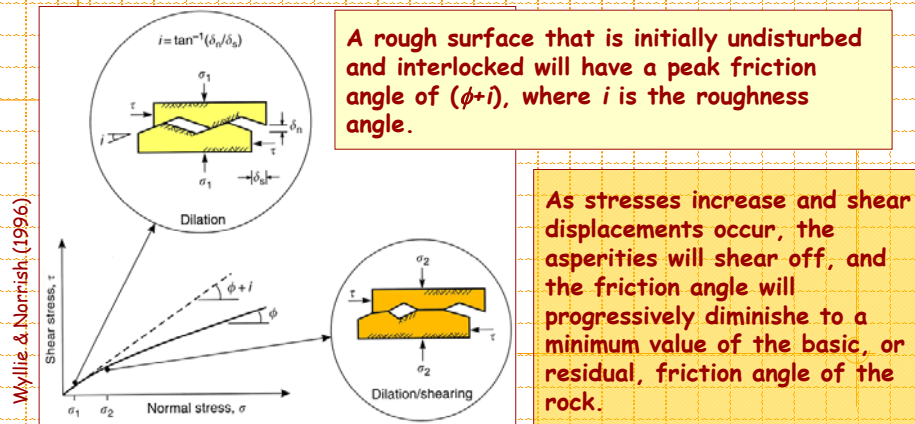
24 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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**.



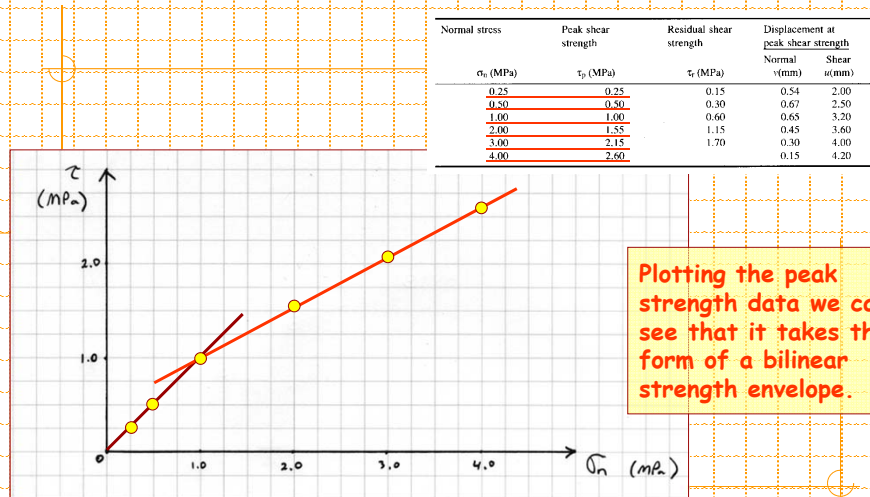
Discontinuity Shear Strength - Example

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	Peak shear strength	Residual shear strength	Displacement at peak shear strength	
σ_n (MPa)	τ_p (MPa)	τ_r (MPa)	Normal v (mm)	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

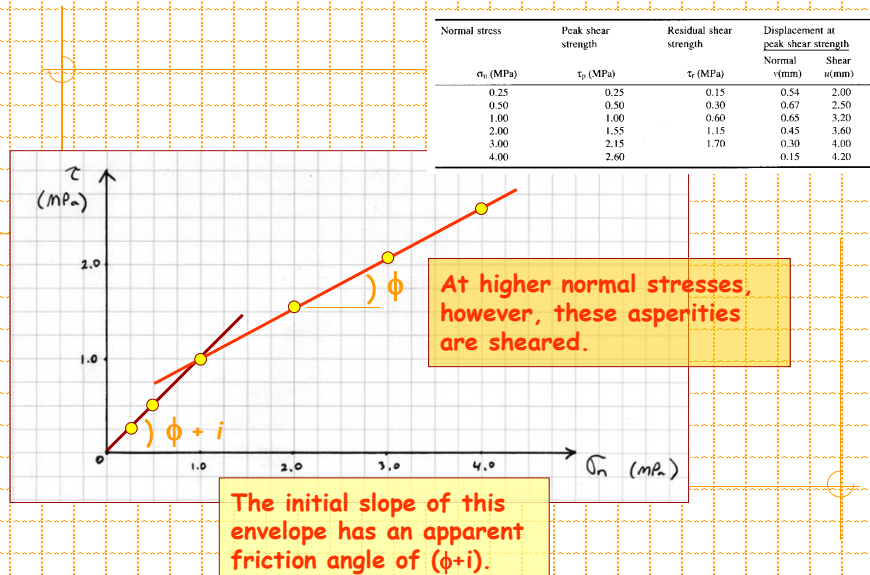


27 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Discontinuity Shear Strength - Example

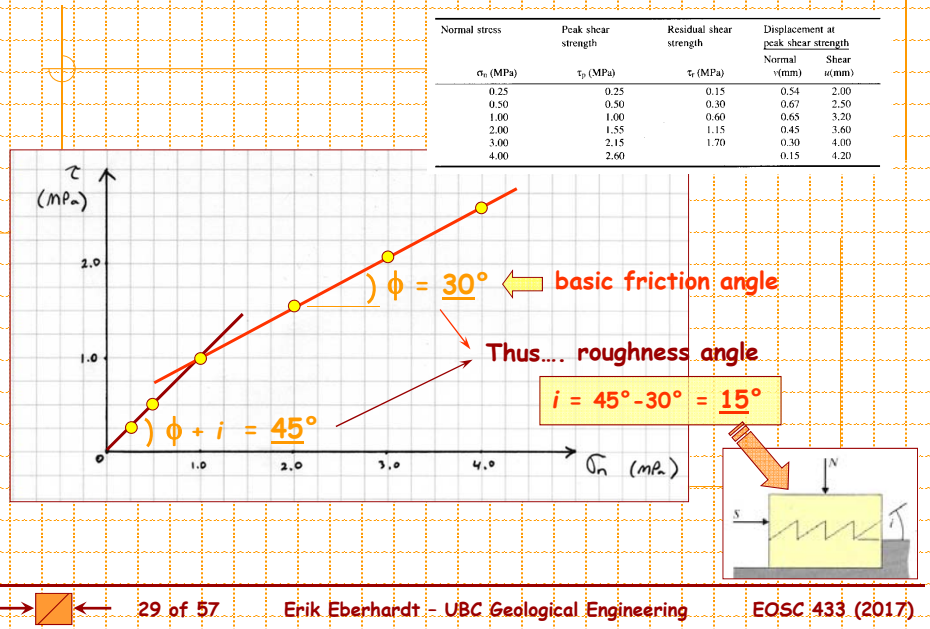


28 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Discontinuity Shear Strength - Example

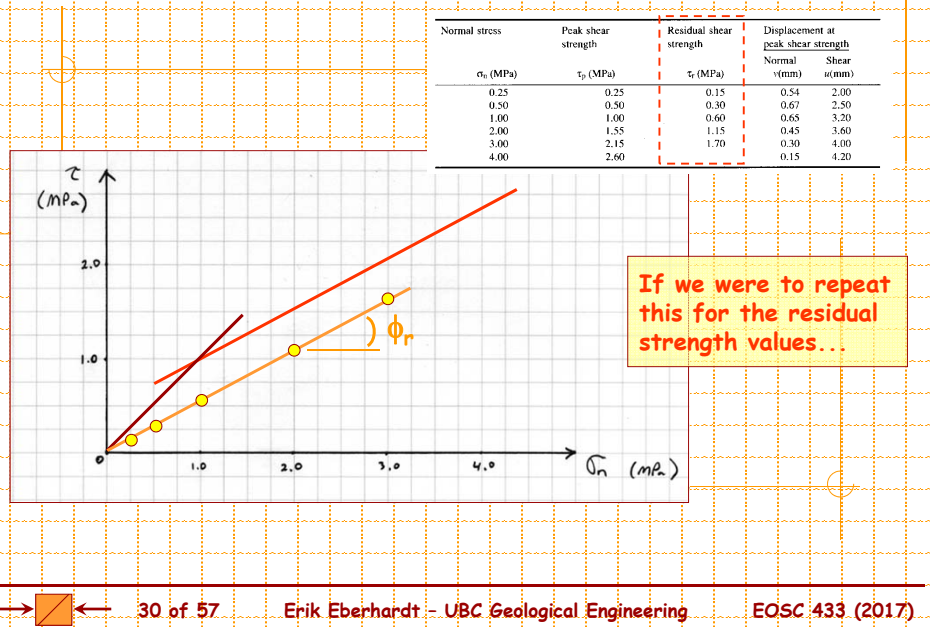


29 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Discontinuity Shear Strength - Example



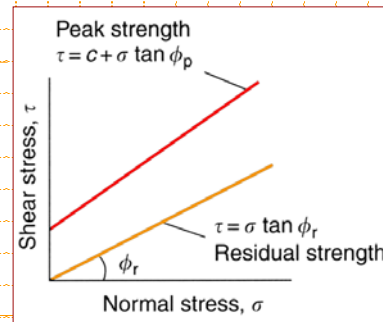
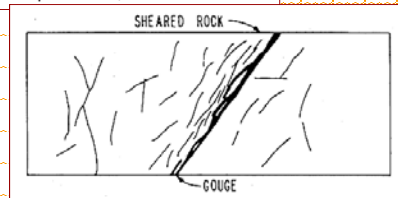
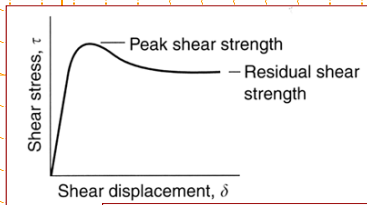
30 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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**.

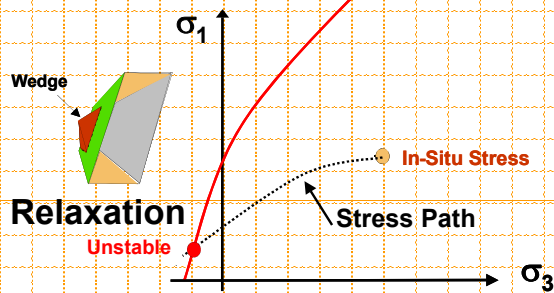
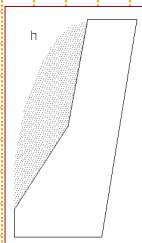
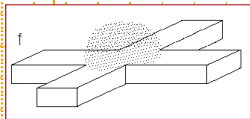
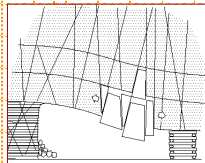


31 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Underground Instability Mechanisms



Kaiser et al. (2000)



32 of 57

Erik Eberhardt - UBC Geological Engineering

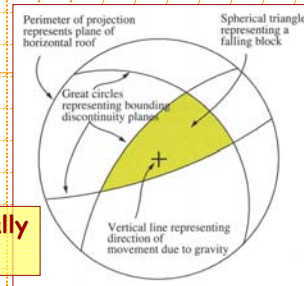
EOSC 433 (2017)

Analysis of Kinematic Admissibility - Falling

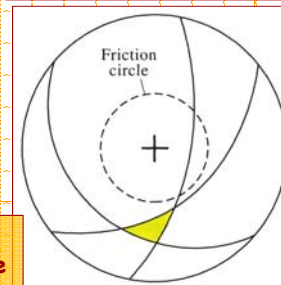
Falling occurs when a block detaches from the roof of an excavation **without sliding** on any of the bounding discontinuity planes. In the case of gravitational loading, the direction of movement is **vertically downwards**.

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)



33 of 57

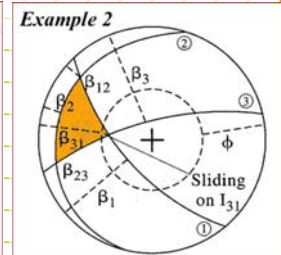
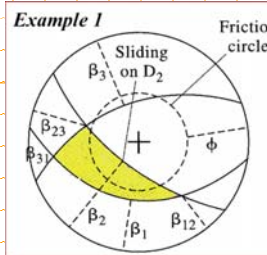
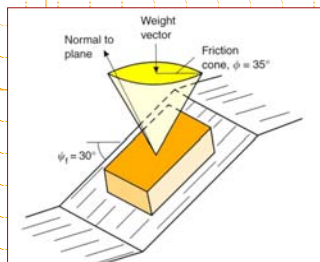
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.

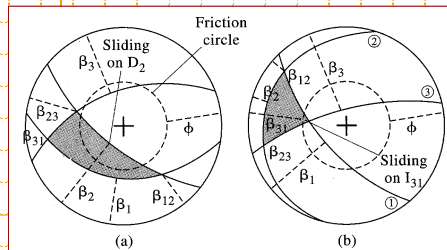


34 of 57

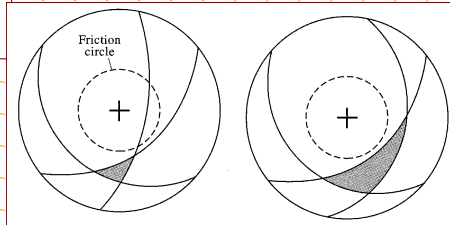
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.



35 of 57

Erik Eberhardt - UBC Geological Engineering

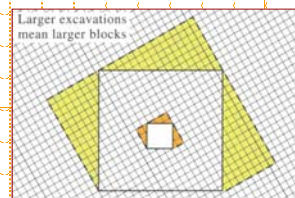
EOSC 433 (2017)

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.



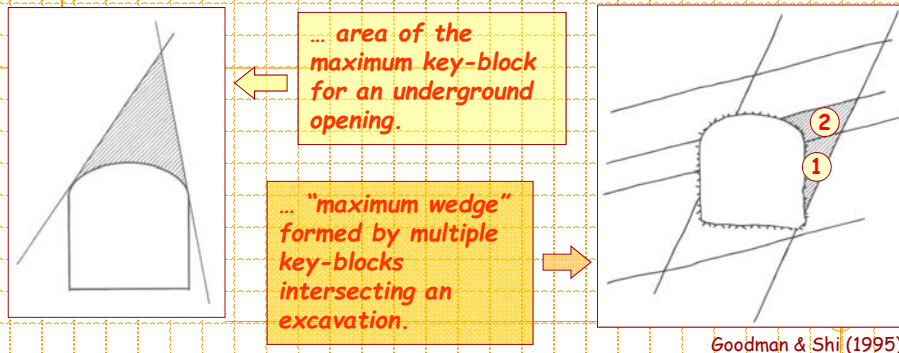
36 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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**.



37 of 57

Erik Eberhardt - UBC Geological Engineering

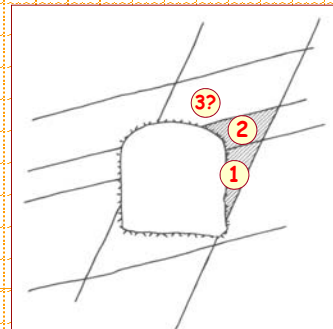
EOSC 433 (2017)

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**.



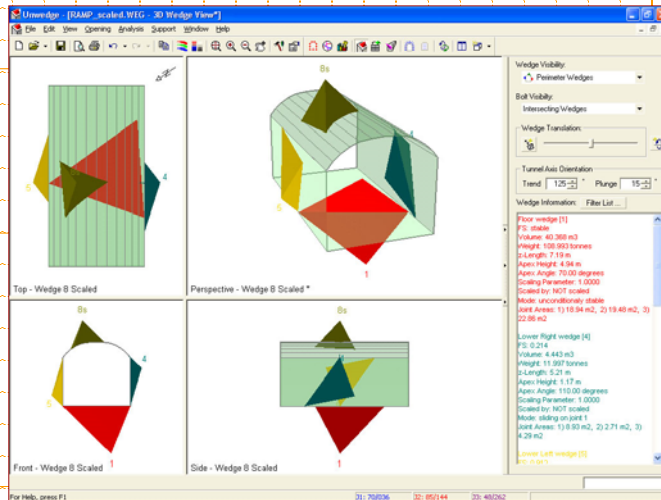
38 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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)



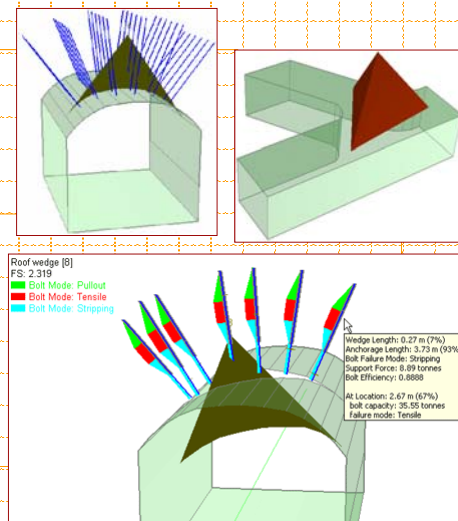
39 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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)



40 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Case History: Rock Slope Stabilization Design

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.)



41 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Case History: Rock Slope Stabilization Design

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.



42 of 57

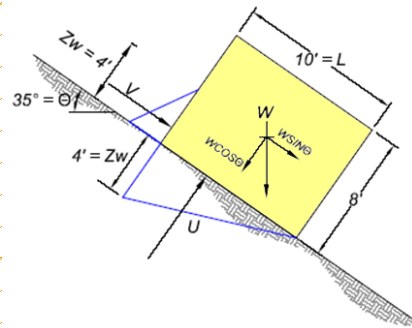
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Back Analysis for Forward Analysis



Courtesy: B. Fisher (Kleinfielder)



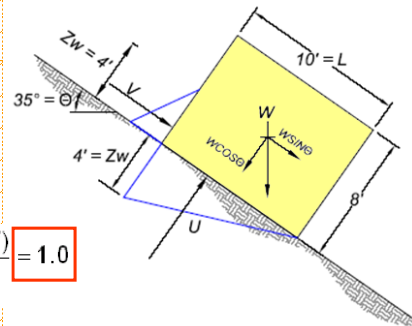
43 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Back Analysis for Forward Analysis

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



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

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

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

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

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



44 of 57

Erik Eberhardt - UBC Geological Engineering

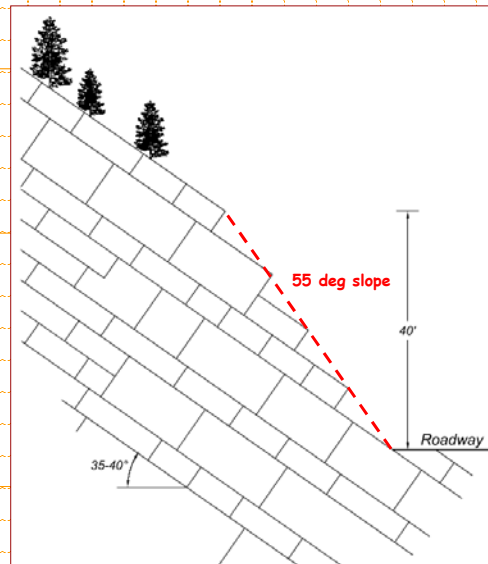
EOSC 433 (2017)

Case History: Rock Slope Stabilization Design

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.



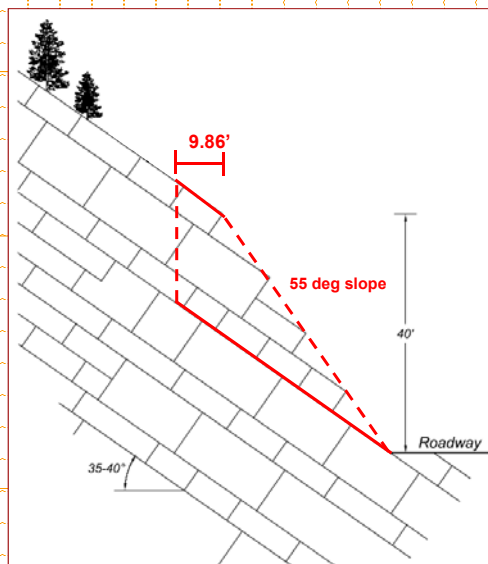
45 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Case History: Design Assumptions

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.)



46 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Case History: Rock Slope Stabilization Design

Results:

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

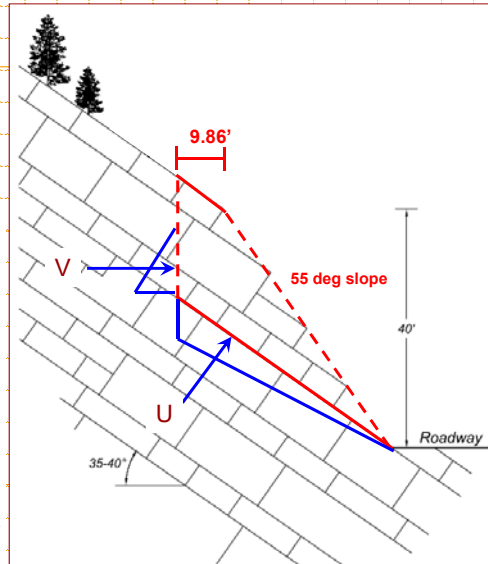
$$L_{Face} = \frac{(H)}{\cos \psi_f} = \left(\frac{40}{\cos 35} \right) = 48.83 \text{ ft}$$

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

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

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

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



Courtesy - B. Fisher (Kleinfelder Inc.)



47 of 57

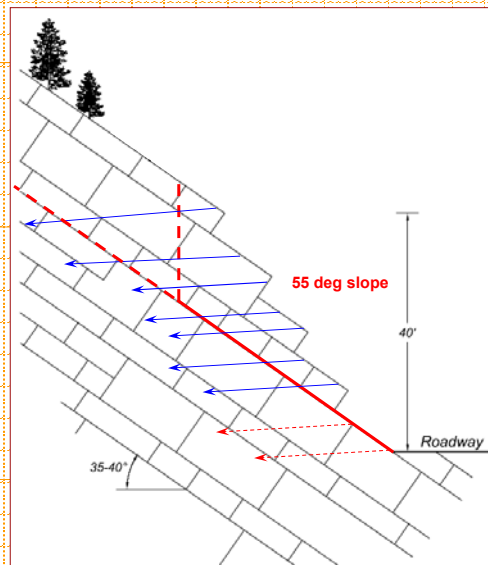
Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Case History: Rock Slope Stabilization Design

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 on rock bolts
6. Engineer responsible to "sign off" on Contractors tests



Courtesy - B. Fisher (Kleinfelder Inc.)



48 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

Case History: Rock Slope Stabilization



Courtesy - B. Fisher (Kleinfelder Inc.)



49 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.



50 of 57

Erik Eberhardt - UBC Geological Engineering

EOSC 433 (2017)

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.

