Strength of Rock and Rock Mass

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Designing with rocks and rock masses allow many similarities to techniques that have been developed for soils. There is however a number of major differences:

•*The scale effect is great in rocks*. Rock strength varies widely with sample size. At one end, we have the intact rock (homogenous, isotropic, solid, and continuous with no obvious structural defects) which really exists only at the hand-specimen scale. At the other end is the rock mass that is heterogeneous and anisotropic carrying all the defects that is characteristic of the rock mass at the field scale. In the design of engineering structures in rock, the size of interest is determined by the size of the rock mass that carries the stresses that are imposed on it.

•*Rock has tensile strength*. It may have substantial tensile strength at the intact rock scale, but much smaller at the scale of the rock mass. Even then only in exceptional circumstances can the rock mass be considered as a "tensionless" material. Intact rock fails in tension along planes that are perpendicular to maximum tension (or minimum compression) and not along shear planes as suggested by the Coulomb theory).

•*In compression, intact rock does not fail according to the Coulomb theory*. It is true, that its strength increases with confining pressure, but at failure there is no evidence for the appearance of a shear fracture as predicted by the Coulomb theory. Furthermore the (envelope) is usually nonlinear following a $y^2=x$ type of parabolic law. Interestingly, shear fractures do form, but not at peak stress; they form as part of the collapse mechanism, usually quite late in the post-failure history.

•*The effect of water on the rock mass is more complex.*

1. Pore space in most *intact rocks is very small* and so is the permeability. The water contained in the pore space is *not necessarily free water*. The truly free water exists only in the rock mass, in fractures, where water may flow at high rates.
2. In contrast to soils, water is more compressible (by about one order of magnitude) than intact rock. The *difference would be smaller* when compared with the compressibility of the *rock mass* (especially close to the free surface where loose rock commonly found).

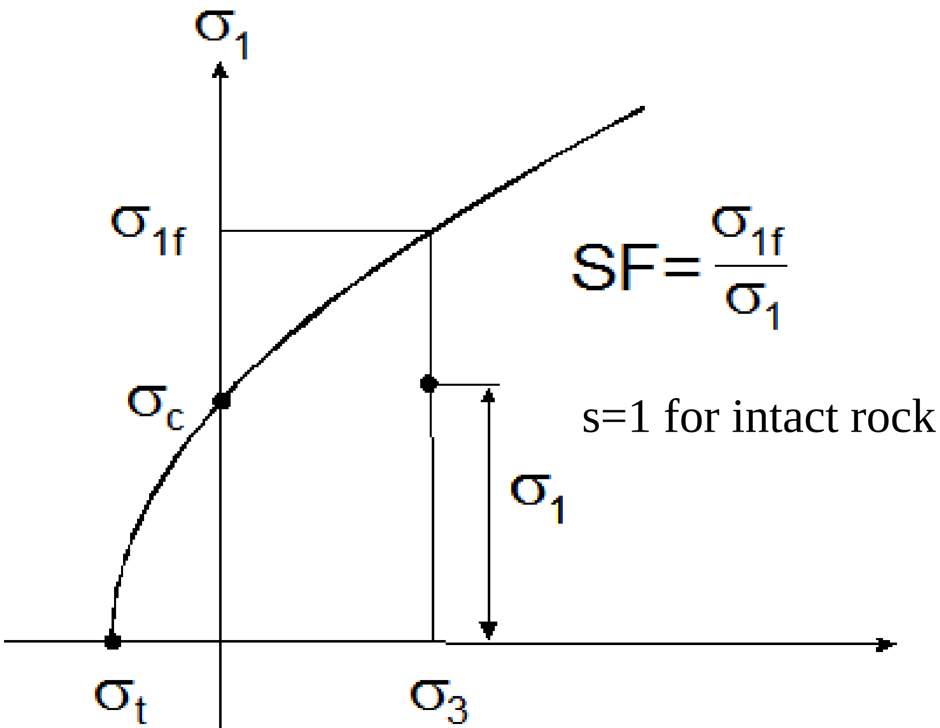
Note that in the derivation of the effective stress theory for soils, the assumption is made that in stressed soils the *grain to grain contact is negligibly small*. This is not so for rocks where the pore space, if it exists, is filled with *cement* increasing the grain-to grain contact to a significant degree. The assumption of "incompressible" water controls the way an applied stress (total stress) is distributed among the solid and the liquid phases. Under undrained conditions, the effective stress (strength) concept passes the whole external stress increment to the water. This does not happen in rock, because water is more compressible than rock. In summary, view the application of the effective stress concept to intact rock with great suspicion.

For the rock mass, water will influence stability as "joint or fracture" water. It is preferable to include its effect separately as a "water force" rather than mix its effect with the rock response (as in the effective stress theory).

Intact rock has both tensile and compressive strength, but the compressive to tensile strength ratio is quite high, about 20. In uniaxial tension, failure follows the maximum principal stress theory: $\sigma_3 = T_o$

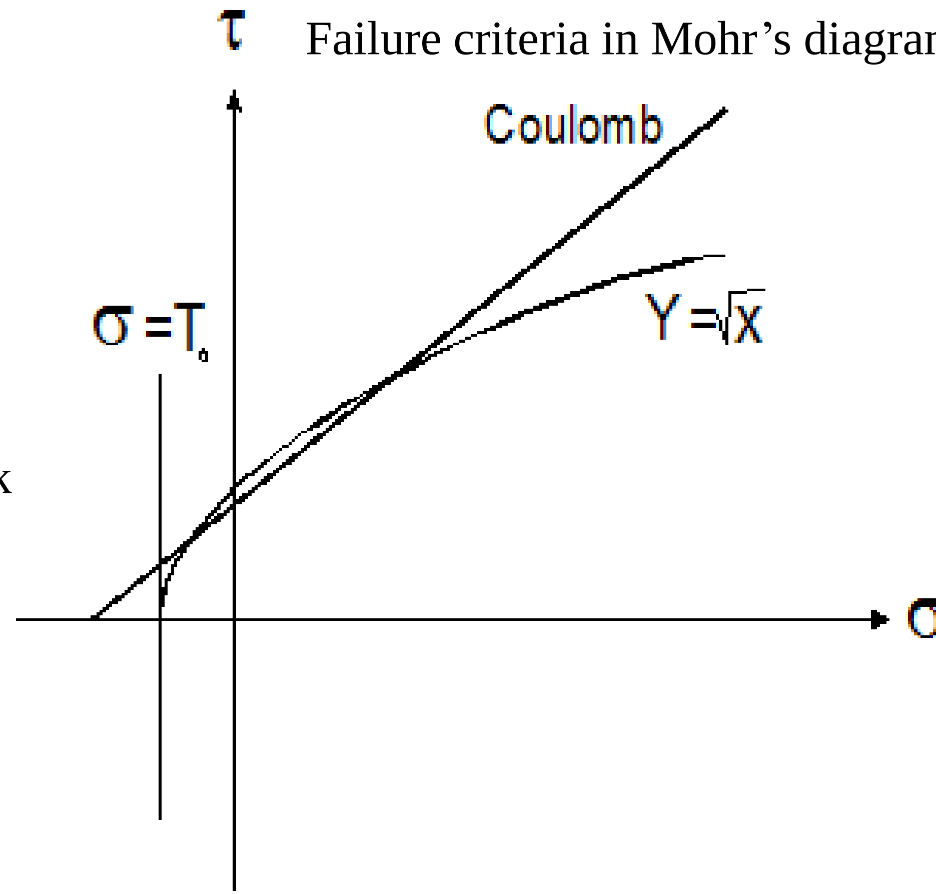
maximum principal stress theory: suggest that the other two principal stresses have no influence

The Hoek and Brown rock strength function



$$\sigma_{1f} = \sigma_3 + \sqrt{\sigma_3 \sigma_c m + s\sigma_c^2}$$

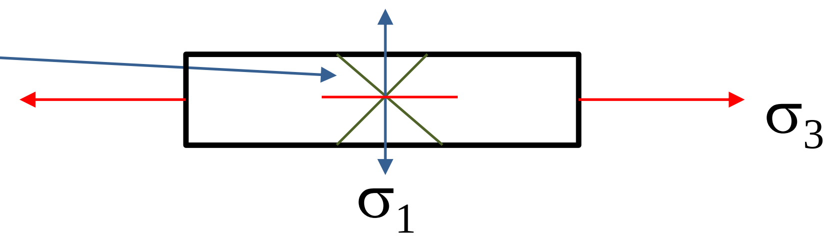
Failure criteria in Mohr's diagram



Finding the parameters m and s from classification parameters

	Limestone Dolomite	Shale Siltstone	Sandstone	Volcanics	Granitic rocks
Intact Rock CSIR=100 NGI=500	M=7 S=1	M=10 S=1	M=15 S=1	M=17 S=1	M=25 S=1
Very Good CSIR=85 NGI=100	M=3.5 S=0.1	M=5 S=0.1	M=7.5 S=0.1	M=8.5 S=0.1	M=12.5 S=0.1
Good CSIR=65 NGI=10	M=0.7 S=0.04	M=1 S=0.04	M=1.5 S=0.04	M=1.7 S=0.04	M=2.5 S=0.04
Fair CSIR=44 NGI=1	M0.14 S=.0001	M=0.2 S=.0001	M=0.3 S=.0001	M=.34 S=.0001	M=0.5 S=.0001
Poor CSIR=23 NGI=0.1	M=.04 S=.00001	M=.05 S=.00001	M=.08 S=.00001	M=.09 S=.00001	M=.13 S=.00001
Very Poor CSIR=3 NGI=0.01	M=.007 S=0	M=.01 S=0	M=.015 S=0	M=.017 S=0	M=.025 S=0

At failure a fracture plane forms that is oriented perpendicular to the σ_3 (Note that the Coulomb theory would predict shear failure in uniaxial tension at 45-ϕ/2 with σ_3).



There was a suggestion to combine the Coulomb theory with the maximum stress theory (the tension cutoff) which would predict the proper orientation of the failure plane for both tension and compression.

Others would rather replace both with a y^2=x type of parabola (Figure 1).

As discussed earlier, the shear fracture does not appear at point of failure, so that this aspect of the Coulomb theory is meaningless.

In fact, there is little point in using the Mohr's diagram. In rock mechanics, failure conditions are more meaningfully presented in the σ_3-σ_1 space using a nonlinear function for strength.

Strength of the Rock Mass

The strength of the rock mass is only a **fraction** of the strength of the intact strength. The reason for this is that failure in the rock mass is a **combination** of both **intact rock** strength and separation or sliding along **discontinuities**. The latter process usually dominates. Sliding on discontinuities occurs against the **cohesional** and/or **frictional** resistance along the discontinuity. The cohesional component is only a very small fraction of the cohesion of the intact rock.

What procedures are used in designing with the rock mass?

•When a rock block is well defined, its stability is best evaluated through a standard **(rigid-body)** analysis technique. All the **forces** on the block are **vector-summed** and the resultant is resolved into **tangential and normal components** with respect to the sliding plane. The safety factor becomes the ratio of the available resistance to sliding to the tangential (driving) force. This is the technique used in **slope stability analysis**.

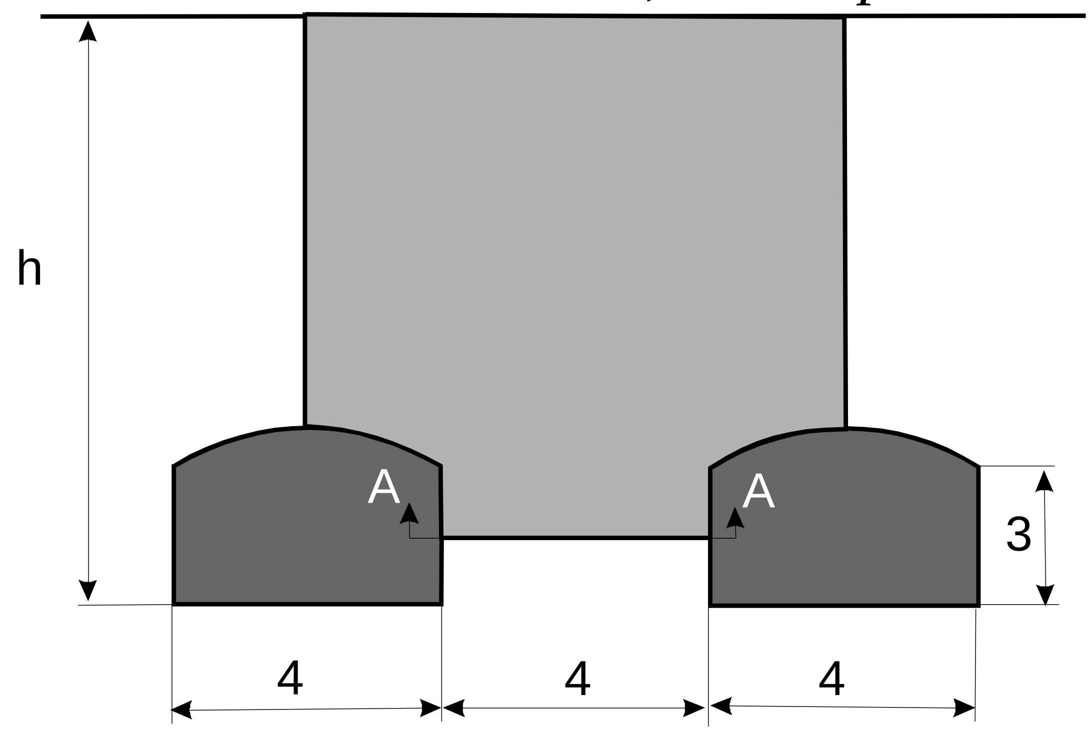
•The second technique is **stress** rather than force-based. Here the stresses are evaluated (usually modeled through numerical procedures) and compared with **available strength**. The latter is expressed in terms of the **Hoek and Brown rock mass strength function**. This is where the s parameter becomes useful. $s=1$ for intact rock and $s<1$ for the rock mass.

What we are doing simply to discount the intact rock strength? What value to assign to s?

There is no test that will define this value. In theory, it is possible to do field tests of the rock mass, but it is expensive and not necessarily very reliable. Hoek and Brown however have compiled a list of s values depending on the rock type and the rock classification ratings. A simplified version of this is presented in the table above. To make use of this table, one needs only the rock type and one of the ratings from either the CSIR (classification of jointed rock mass) or the NGI (Q-System) classification. Ratings that are not listed will have to be interpolated. User's of this table are however warned that this approach is given here as a guide and its reliability is open to question. Nevertheless, the given s values are so small that they would tend to under rather than overestimate the strength of the rock mass. Problems however could arise when failure occurs along a single weak discontinuity (slope stability), in which case the stress-based approach is obviously invalid.

How engineers design engineering structures in rocks using the Hoek and Brown approximation for strength?

Let us assume that we are going to build a twin-tunnel road system at some depth in the worked rock mass. The plan is two make two inverted-U shaped tunnels, each tunnel to be 3 m high and 4 m wide. The tunnels are to be separated by a pillar (rock left in place), preferably no more than 4 m wide. The safety factor for the pillar should be 1.5 or better. The depth of siting for the roadway has not been established yet, but it could range anywhere between 100 and 300 m, the *deeper the better.*



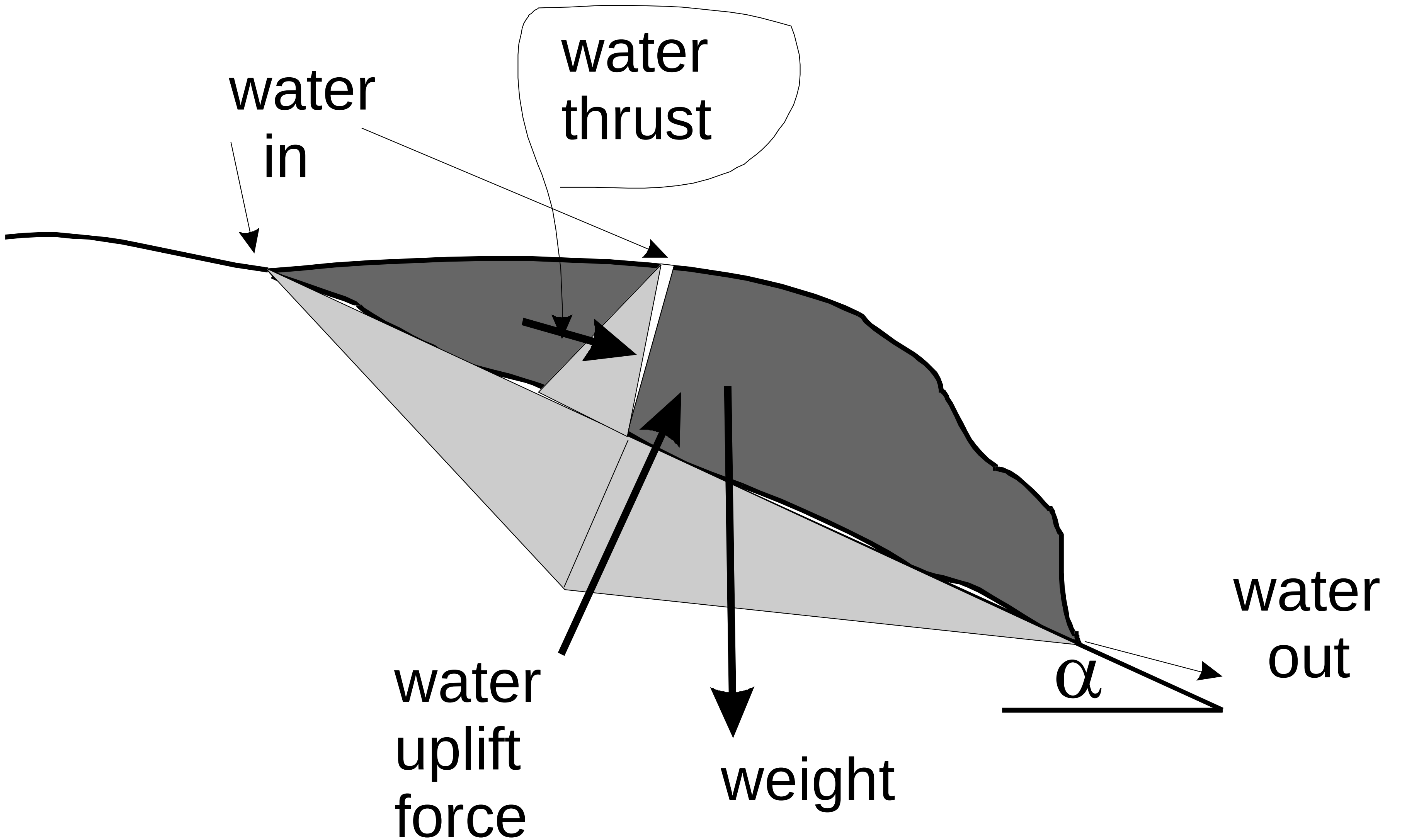
The job is to find the appropriate depth within this range. This is an example for pillar design. The loading condition is determined by assuming that the weight of the overlying rock mass, as shown in Figure 1, is distributed evenly across the width of the pillar at AA (this is not quite true, the stresses are usually higher at the tunnel perimeter than at the center, but the high safety factor should take care of this). You follow this procedure now:

•Find the rock mass strength using your classification and the strength table give above.
•Find the volume and the weight of the overlying rock using 100 m for depth (check if you have a unit weight for the rock in the report).
•Distribute the total weigh over the cross sectional area AA. This is the average vertical stress on the cross section
•Formulate the safety factor as

$$Sf = \frac{Strength}{VerticalSress}$$

•Check the safety factor at 300 m
•See if you can get an algebraic expression for the safety factor using *h* as a variable.
•What is the story you are going to tell the boss?

How the engineers can attempt to find the safety factor for the problem shown?



We are looking at the stability of the dark-shaded mass of rock. There is the possibility of sliding down along joint plane sloping at angle α. First, we should establish the forces that act on this block of rock. Weight is an obvious one. The water forces are based on the assumption that water flows along the slide plane and perhaps along other joints or as in this case in a tension crack as well. If there was no tension crack, we would have an uplift force alone arising from the fact that water would normally flow in at the high-elevation end and flow out at the low elevation. The head of water at the intake and discharge points is zero. It would normally maximize between. Here we assume a triangular distribution, assuming that the maximum head occurs at midpoint and its value is one half of the elevation difference between intake and the discharge points. The uplift force itself is equal to the area of the pressure distribution diagram (light-shaded area) and acts perpendicular to the slide surface. With a tension crack, there could be a slope-parallel water thrust due to water accumulating in the tension crack. Its value would be calculated from the upper (small) light-shaded triangle. For this the maximum head would occur at the base, with the maximum head being equal to the elevation difference between the top and the bottom of the tension crack.

Give an example to how the engineers use the technique of applying the block theory to designing rock slopes.

1. Assume that the elevation difference between the intake and discharge points is 20 m and the slope angle is 30°. Find the weight of the block of rock (*hint:* turn it into a triangle to ease the pain of calculation) using a width of 1 m in the third direction. Assume 25 kN/m^3 for the unit weight.
2. Compute the uplift force and the water thrust
3. Resolve all the forces into components, normal and parallel with the slide plane
4. Sum the parallel (tangential) forces to get the **Driving Force**
5. Sum the normal forces and get the total frictional resistance by multiplying it with tan ϕ (use 30°)
6. Define the cohesive force as unit cohesion times the total area of contact; the unit cohesion will stay as a variable now
7. Add the cohesive force to the total frictional force
8. Formulate the safety factor, equate it with 1 and compute the friction angle.

After this operation, you have all the strength parameters defined and are ready to redesign the slope. In practice, you would get rid of the water by drilling drainage holes to intersect and drain the slide plane. Assuming that the drainage works, do the last thing:

9. Find now the safety factor for the slope with the water effect gone! If it is greater than about 1.25, tell the people that the slope is safe as long as they have the drainage holes clean. Otherwise you would have to install and anchor system to increase the safety factor (changing the weight of the block by shaving it would result in a minor improvement only, you can try this analysis too.)

What are the effects of ground water in rock slope stability?

We must have a particular attention how the effect of ground water is incorporated into the stability analysis. The water is *increase the weight of block*, *decrease the normal force* on the failure plane and *decrease the frictional resistance*.

Which method the engineers use for rock slope and how?

We use block analysis when we expect the block to slide on a single or a combination of discontinuities and we have pretty good control over the geometry. This means that we have good knowledge of the size and through this the weight of the block and the geometry of the slope. In the simple two-dimensional case, the geometry is simply the slope angle.

Illustrate the shear resistance as in the Coulomb theory?

The discontinuity shear strength is made up of two components, cohesion and frictional resistance. The cohesion supposed to represent the strength of "solid rock bridges" that may exist at the base and will have to be sheared off to let the block move. This is the hardest part to estimate, because it may vary between zero and the strength of the solid rock. Usually, it is a very small fraction of the solid strength. The frictional part is simply the normal force times the tangent of the friction angle. We use *forces rather than stresses* here and the resistance force according to the Coulomb specification becomes:

$$Discontinuity\ shear\ strength = Cohesive\ \ force + N\tan\phi$$

When and how the Coulomb Theory is useful for rock slope stability?

The Coulomb type of specification is useful only in the "back analysis" of slope failures. In the consulting business, a common chore is to redesign slopes that have either failed or showed signs of instability (tension crack at the back of the slope). In cases like this, the safety factor should be very close to unity. If we have a good estimate of the friction angle and the loading condition then we can back-calculate and find the value of the discontinuity shear strength. Having this, the slope can be redesigned to a new safety factor with a lot more confidence. When no back analysis is possible, coming up with a reliable estimate of the available cohesive force is practically impossible. Most design situations fall into this category.

What are the alternative way of Coulomb theory for estimating discontinuity strength? Barton specification advances a non-linear law:

Joint Roughness Coefficient

Joint Compressive Strength

$$\tau = \sigma_n \tan\left(JRC \log_{10}\left(\frac{JCS}{\sigma_n}\right) + \phi_b \right)$$

What are the differences between Coulomb theory and Barton law?

Coulomb Theory use the force unit, while Barton law use the stress unit. The Coulomb law refers to no strength at zero normal stress. Essentially, the Barton specification is defined in terms of a friction angle that is adjusted for joint roughness and the strength of rock.

Which material parameters are Barton strength uses?

JRC (joint roughness coefficient), *JCS* (joint compressive strength) and ϕ_b (basic friction angle).

1. *JRC* varies between 0 (very smooth, planar joint) and 20 (rough undulating surface). *JCS* is a fraction of the compressive strength of the rock.
2. The compressive strength should be discounted depending on the condition of the rock walls on the two sides of the joint. Usually the surface is weathered and altered and may carry soft filling. In the latter case, the strength would be very small indeed.
3. The basic friction angle is what we would normally call the friction angle determined on a flat surface rubbing against another flat surface of the same rock.

THANK YOU