<u>Instability of tunnels in rock.</u>

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ABSTRACT.

The analysis of the magnitude and condition of the stress state in the ground mass surrounding the excavation will help to minimise project costs by optimising design. This paper will deal with application of the knowledge of stress fields surrounding the excavation to help understand the failure mechanism within the rock mass. This paper aims to develop an understanding of the Hoek-Brown failure criterion and investigate its application in the calculation of tunnel instability using closed form methods.

Rock Failure Surrounding Tunnels

In hard rock failure is governed not only by the stress distribution and rock strength but also the characteristics of the rock mass. The continuity and distribution of natural rock fractures are key to the stress distribution and propagation of stress fields into the surrounding rock mass. Discontinuities and their interactions with the induced stress fields may be modelled by applying boundary conditions. The induced stresses can then be compared to the rock strength-failure criterion to perform a stability analysis (Martin et al, 1999).
The Hoek-Brown failure criterion is a widely accepted method of analysis for rock failure and is primarily a modification of the proven Mohr-Coulomb criterion, using empirical relationships derived from extensive field research. The derivation has been updated in 2002 proceeding further research. The latest Hoek-Brown criterion developed in 2002 is stated as:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \qquad \textbf{Equation 1}$$

Where:

- σ_1' and σ_3' are the maximum and minimum effective principle stress
- σ_{ci} is the uniaxial compressive strength of the intact rock
- m_b is the modified Hoek-Brown constant for the rock mass
- s and a are property dependant constants

As the Hoek-Brown criterion modifies the Mohr-Coulomb theory according to the properties of the rock mass through which the excavation is made, the values of m_b, s and a are determined according to a rock classification index developed for characterising the rock mass. The *Geological Strength Index* (GSI), was developed to provide for local rock properties by Hoek and Marinos (2000) and Hoek, Wood and Shah (1992). The GSI replaces the previous Rock Mass Rating (RMR) by Breniawski (1976).
m_b is a modification of the existing Hoek-Brown constant m_i. m_i and σ_{ci} are obtained from undrained triaxial testing using standard laboratory equipment. For consistency with research conducted by Hoek and Brown resulting in the derivation of the empirical constants, the triaxial tests should be conducted a minimum of five times over a range of 0< σ_{ci} < 0.5σ_{ci}. The values of σ_{ci} and m_i may then be determined in the methods detailed Hoek (2000).

To apply the Hoek-Brown failure criterion the value of m_i can be used to determine the modified values m_b, s and a as given by the equations of Hoek 2002:

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \qquad \textbf{Equation 2}$$

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

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{\frac{-20}{3}}\right) \qquad \textbf{Equation 4}$$

In the above equations, the factor *D* is known as the disturbance parameter where in an undisturbed rock mass, *D* approaches 0 and *D* approaches 1 in extremely disturbed ground (such as blast damaged ground etc). An indication of how to select *D* and for GSI evaluation readers should refer to Hoek (2000).

It should be noted from the equations that as the value of GSI and *D* increase, that is, approach 100 and 1 respectively, the strength of the rock mass, σ_{cm}, approaches the uniaxial compressive strength of the intact rock, σ_{ci} (Martin et al, 1999).

Using the classification systems and failure criterion, we are able to determine the stability of the rock mass through which the tunnel is being driven with a level of confidence.

1.1.1 Tunnelling In Weak Rock

Tunnelling in weak rock mass presents many difficulties for design engineers. Firstly compared to massive and relatively intact rock masses, weak rock masses contain large quantities of rock structure that instigate weakness. These structures include bedding planes, fault planes discontinuities and joints. In highly stressed rock masses the extent of jointing will lead to instability of rock wedges around the tunnel periphery. In certain instances where extremely high in-situ stresses have caused the surrounding rock to become completely pulverised, little or no stability can be expected and support systems need to be installed immediately upon excavation.

To understand instability in tunnels, the analysis of rock mass behaviour (not limited to weak rock) and deformation response to excavation must first be conducted. As shown in Figure 1, deformation of the rock mass begins approximately 0.5D in front of the excavation face and reaches its maximum value of approximately 1.5D behind it (assuming $\sigma_h = \sigma_v$). This deformation may lead to instability.

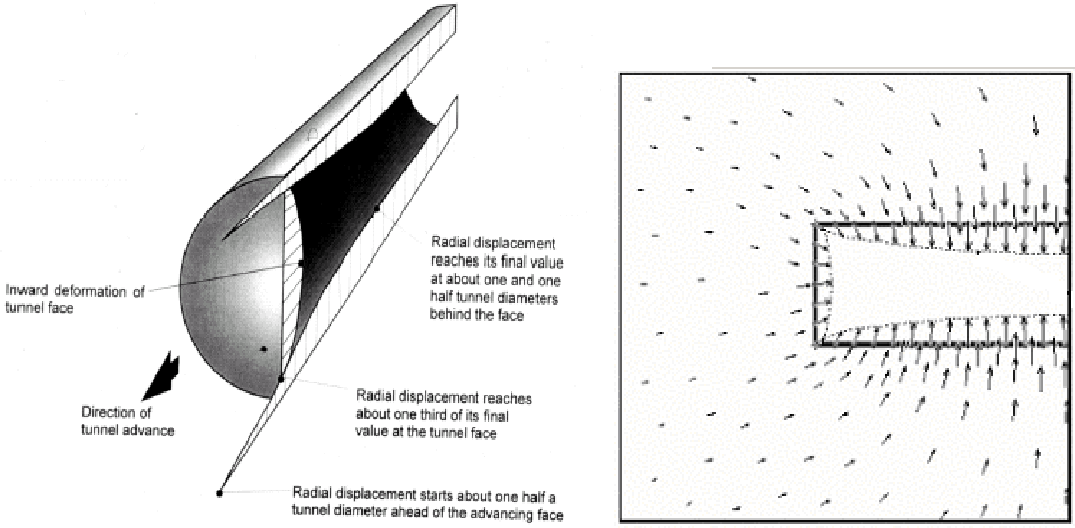

Figure 1: Deformation and induced stress in an advancing tunnel in weak rock. (After Hoek, 2000)

Rock deformation usually takes three forms:

1. **Stress Induced Instability**

The stress field induced by rearrangement of in-situ stresses upon excavation may lead to localised rock failure surrounding the opening. Using the Kirsch equations, it was assumed that the rock behaved as an continuous, homogeneous, isotropic, linear, elastic (CHILE) material. In reality however this is seldom the case and weak rock masses are abundant with structure which produces many inaccuracies in analysis.
Therefore in weak rocks the rock mass can assumed to be heavily jointed and that a ring of rock surrounding the excavation fails and instability is induced if internal support pressure is not provided immediately. For analysis of the development of the failed ring or rock, it is assumed that the rock mass behaves as an *elastic-perfectly plastic* material in which slip across the failure planes causes zero plastic volume change (Hoek, 2000).
The Mohr-Coulomb failure envelope is thus modified by the parameter *k* and becomes:

$$\sigma_1' = \sigma_{cm} + k\sigma_3' \qquad \textbf{Equation 5}$$

σ_{cm} is the uniaxial compressive strength of the rock mass. It should be noted that this value differs from σ_{ci} in the Hoek-Brown failure criterion in that σ_{cm} defines the entire rock mass and incorporates the effect of joints and discontinuities on the uniaxial compressive strength of the intact rock, σ_{ci}.
σ_{cm} may be derived from triaxial tests of rock samples containing these discontinuities and can be defined as:

$$\sigma_{cm} = \frac{2c'\cos\phi'}{(1-\sin\phi')} \qquad \textbf{Equation 6}$$

where *c' and* ϕ' are the effective Mohr-Coulomb parameters. In $\sigma_1' = \sigma_{cm} + k\sigma_3'$ Equation 55, *k is the slope of* σ_1' vs. σ_3' and is given as:

$$k = \frac{1-\sin\phi'}{1+\sin\phi'} \qquad \textbf{Equation 7}$$

If the stresses induced by the excavation are large enough, a ring of damaged rock, or 'plastic zone', will develop around the circumference of the excavation. The extent of the plastic zone is dependent on the distribution of the in-situ and induced stress fields and σ_{cm}. The development of a plastic region will induce displacements inward to the opening if an internal support pressure is not immediately applied. This phenomena is depicted in Figure 2 and the radius of the plastic ring is given by the expression:

$$r_p = r_o\left[\frac{2(p_o(k-1)+\sigma_{cm}}{(1+k)(k-1)p_i+\sigma_{cm}}\right] \qquad \textbf{Equation 8}$$

and the total radial displacement of the plastic zone is:

$$u_{ip} = \frac{r_o(1-v)}{E}\left[2(1-v)(p_o - p_{cr})\left(\frac{r_p}{r_o}\right)^2 - (1-2v)(p_o - p_i)\right] \qquad \textbf{Equation 9}$$

Where:

- r_p is the radius of the plastic zone
- r_o is the theoretical radius of excavation
- p_o is the value of the in-situ stress σ'_1
- p_i is the internal support pressure
- E is the deformation modulus of the rock mass given as:

$$E = \sqrt{\frac{\sigma_{ci}}{100}}10^{\left(\frac{GSI-10}{40}\right)} \qquad \textbf{Equation 10}$$

It should be noted that failure of the surrounding rock mass will occur when the internal pressure p_i is less than the critical required support pressure, p_{cr} , given by

$$p_{cr} = \frac{2p_o - \sigma_{cm}}{1+k} \qquad \textbf{Equation 11}$$

Using these relationships it is possible to calculate the required support pressures to minimise deformations and extensive formation of the plastic ring.

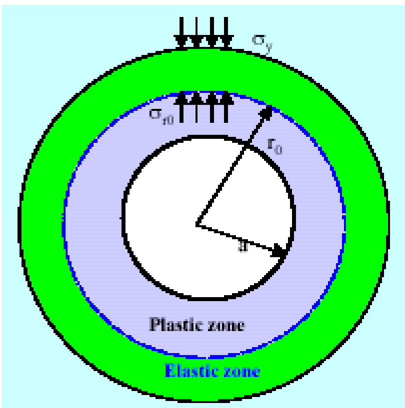

Figure 2: Plastic Zone surrounding a circular tunnel (Morgan and Yang, 2002)

Empirical relationships using the above equations have been derived for assessing the likely damage to the surrounding rock. It has been shown that for unsupported excavations the plastic zone increases rapidly as the value of σ_{cm} falls below 20% of the in-situ stress field with deformations also increase considerably at this point. Therefore it becomes very difficult to control stability and collapse is imminent unless support is provided simultaneously with the excavation (Singh et al, 1998).
The following equations which are graphically depicted in Figure 3 and Figure 4, can be used to establish required support pressures to limit plastic zone propagation and excessive deformations:

$$\frac{r_p}{r_o} = \left(1.25 - 0.625\frac{p_i}{p_o}\right)\frac{\sigma_{cm}}{p_o}^{\left(\frac{p_i}{p_o} - 0.57\right)} \quad \textbf{Equation 12}$$

$$\frac{u_i}{r_o} = \left(0.002 - 0.0025\frac{p_i}{p_o}\right)\frac{\sigma_{cm}}{p_o}^{\left(2.4\frac{p_i}{p_o} - 2\right)} \quad \textbf{Equation 13}$$

Once again, while these empirical relationships are useful for providing an outline of the expected behaviour and preliminary estimates for support requirements, engineers should utilise one of the computer based numerical methods available to more accurately model the results.

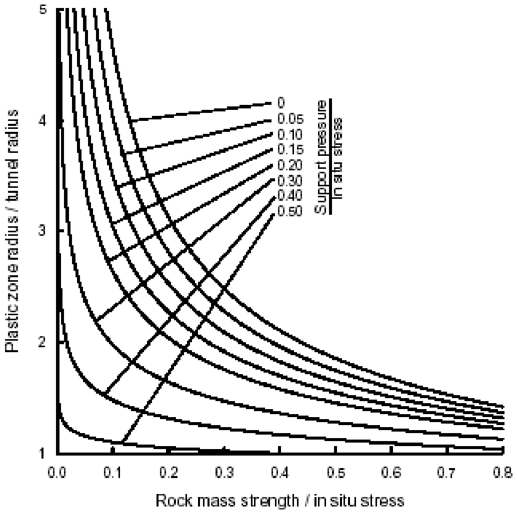

Figure 3: r_p/r_o *vs.* σ_{cm}/σ_v (Hoek, 2000).

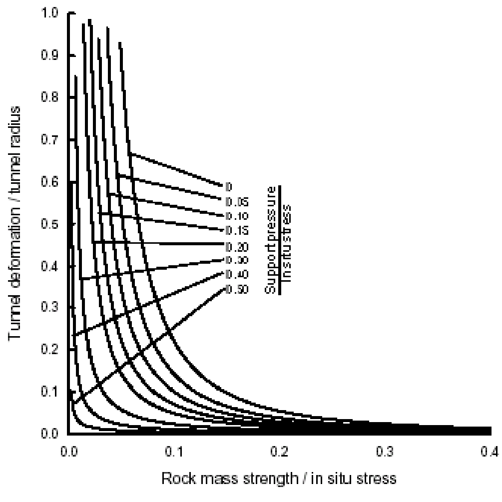

Figure 4: u_i/r_o *vs.* σ_{cm}/σ_v (Hoek, 2000)

2. <u>Instability resulting from rock structure.</u>

Excavation of highly jointed rock masses may produce instability by mobilising blocks or wedges of rock freed from confinement pressures that kept them in static equilibrium. This phenomenon is termed *structurally controlled instability.*
Intersecting discontinuities form tetrahedral blocks with a discrete block being formed by four or more non-parallel discontinuities. Three failure planes may be the result of intersecting

joints and bedding planes etc and the fourth is formed by the excavation periphery. The formation of the final failure plane of the block results in the wedge either falling from the crown under gravity or sliding off the sidewalls along one of its failure planes.
In order to determine the kinematic feasibility of the wedges, the following data should be assessed (Eberhardt, 2002):

a. Average dip direction and angle of significant discontinuities
b. Plunge of the tunnel in relation to (a)
c. Trend of the tunnel in relation to (a)

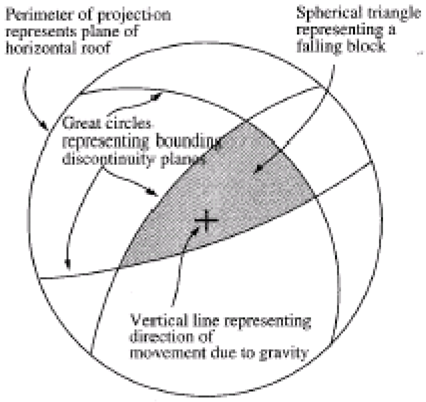

Figure 5: Greater circle representation of discontinuity boundaries forming a tetrahedral block (Eberhardt, 2002).

Figure 5 represents the plane of excavation and shows the formation of the wedges due to excavation.
There are three distinct possibilities when a block is freed. Firstly the block may fail by falling, secondly failure may take the form of sliding and finally that the block remains stable.
Falling usually occurs at or very close to the crown. In this condition the cohesion at the discontinuities is assumed to be zero and the block may become dynamic immediately upon excavation of the final failure plane. In instances where this is likely to occur, immediate support, in the form of rockbolts, as close as possible to the face may be provided.
The second scenario for mobilisation comes from sliding. Sliding is usually seen at the sidewalls as the block moves along the plane of the discontinuity of greatest slope or the line of intersection of two failure planes. Sliding failure may provide some stand up time as the block must overcome the angle of friction before moving. Depending on the inclination of the failure plane to horizontal, support may not need be provided instantaneously but as soon as practical after excavation.
The final scenario of the block remaining stable usually occurs at the invert or close to and does not require any mention.
Wedge or block failure in tunnels may induce overall instability of the excavation if not controlled in time. In jointed rock mass stability of the blocks is primarily due to the interlocking of the discontinuities, thus loosening of one block may open up planes of movement for adjacent blocks making them unstable. This can lead to a chain reaction effect of wedge failure which will only cease when sufficient interlocking of discontinuities is established to attain static equilibrium, when a natural arch is formed or when the excavation is filled with rock.
Therefore care should be taken during the investigation and design phase to predict potential zones of wedge formation. Further care during the excavation process should be undertaken, in the form of geological mapping. The produced geological maps can then be used to identify potential failure planes and estimate support requirements.

Geological boreholes at regular intervals along the tunnel alignment are useful for modelling the physical geology or the tunnel route. There are also graphical methods that are appropriate in rock which are not discussed.

3. **<u>Pre existing state of ground</u>**

In highly fractured, weathered, jointed, faulted and tectonically disturbed rock masses the rock may be considered to be in a semi-failed state and therefore even when the level of induced stress is significantly lower than σ_{ci} , excessive ground deformations can be expected. An example of this is *squeezing ground* in which a combination of the weak rock mass and the high in-situ stress state cause the rock mass to plastically deform into the tunnel, particularly in the sidewalls. This condition makes tunnelling extremely difficult as the squeezing ground places high stresses on the shield (if so used) and the final tunnel lining. Thus the stress along the surface area of the shield increases and therefore higher jacking forces are required. In extreme cases the shield may become stuck causing a significant delay as remedial work is undertaken. High support pressures may be required at the face along with a large, stiff, reinforced concrete final lining to limit deformations and thus surface settlements.
Another example when the ground conditions may cause deformation is where *swelling* rock masses are encountered. In this instance the excavation allows a phreatic surface to be formed as the excavation will be at atmospheric pressure, thus a path for water to flow due to the net pressure (head) differential is set up. The result is the surrounding ground absorbing the ground water, increasing in volume and expanding into the tunnel. Rock masses with high clay contents are likely to experience this problem. Similar consequences to squeezing ground may result.
In the case of highly stressed, massive and relatively intact rock mass under high in-situ stresses, a phenomenon known as *rockburst* may result. The excavation allows an avenue for release of the high in-situ stress; the result is a violent failure of the rock mass similar to an explosion. This situation is likely in alpine regions where excess folding and large overburdens cause high in-situ stresses.
In extremely weathered conditions where the rock mass has taken a 'granular' form, the presence of high water head may lead to a flowing or running phenomenon of the ground similar to that of soils (Singh, 1998).
The last example that will be considered is that of openings in stratified rock. A similar process to that of wedge failure may occur when interlayer slip caused by the induced stress field results in cracks propagating between bedded layers, producing the final plane resulting in a layer block mobilising into the excavation (See Figure 6). This phenomenon is commonly referred to as the 'slab effect'.

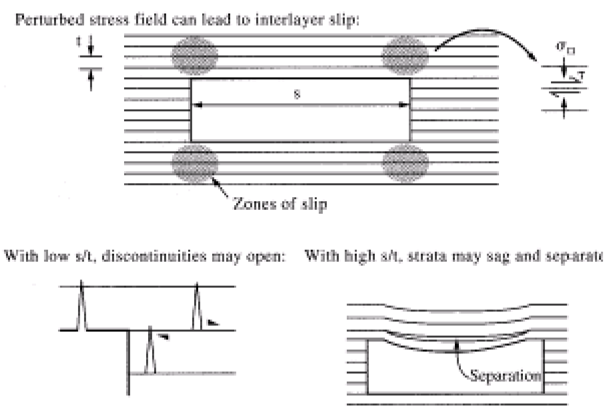

Figure 6: Induced instability in stratified rock (Eberhardt, 2002).

Conclusion.

It can be seen that tunnelling in weak rock conditions may cause significant deformations around the excavation leading to increased tunnel lining/support requirements and high surface settlements. Table 1 shows the primary methods of deformations encountered in rock conditions. Due to the significantly varying geological conditions encountered, all leading to a separate form of instability, the Mohr-Coulomb soil mechanic principles have to be monitored for individual rock conditions.

By successfully predicting and analysing the ground structure, stress state and geology design engineers can then model and assess the support and lining requirements for the drive, and this may eventually assist in final machine selection or method of construction employed for the project.

Table 1: Classification of ground conditions (Singh, 1998).

Sr. no.	Ground classification	Sub-class	Rock behaviour
1	Elastic	Self-supporting	Massive or competent rock mass requiring no support for tunnel stability
		Non-squeezing	Massive and competent rock mass requiring supports for tunnel stability
2	Ravelling		Chunks or flakes of rock mass begin to drop out of the arch or walls due to loosening, sometimes after the rock mass is excavted
3	Squeezing	Mild squeezing ($u/a = 1-3\%$) Moderate squeezing ($u/a = 3-5\%$) High squeezing ($u/a > 5\%$)	Rock mass squeezes plastically into the tunnel. Rate of squeeze depends upon the degree of overstress. Occurs at shallow depths in weak rock masses like clay (after Singh et al., 1973), etc. Hard rock masses under high cover may move in combination of ravelling of face and squeezing behind the face.
4	Swelling		Rock mass absorbs water, increases in volume and expands slowly into the tunnel, e.g., montmorillonite clay.
5	Running		Granular material becomes unstable when exposed to steeper slopes.
6	Flowing		A mixture of soil-like material and water flows into the tunnel. The material can flow from invert as well as from the face, crown, and wall and can flow for large distances completely filling the tunnel in some cases.
7	Rock burst		A violent failure in hard and massive rock masses when subjected to high overstress.

u = radial tunnel closure; a = tunnel radius.

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