Geotechnical Aspects of Exploration

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ABSTRACT

This paper examines the multiple processes that should be followed to obtain geotechnical information during an exploration programme. Determination of rock properties by core logging, geophysics and material testing are considered. This work includes the selection of methods of testing that should be used to suit the rock type and the likely mine development.
The process of determining rock stress from paleo indicators, regional features, borehole breakout, overcoring and hydrofracture are considered as is the variability of rock stress.
The measurement and analysis of ground fluids for the determination of their effect on effective stress, outbursting and fluid inflow are presented. The section pays particular attention to the use of petroleum industry derived test techniques adapted to the mine exploration environment.
Finally, the paper looks at how this geotechnical information should be used in underground design. It also emphasises the importance of getting enough information to reduce risk to later mining. Examples of suitable measurements for coal and metalliferous mine exploration are presented to outline the potential complications.
A lot of what is presented emphasises the dangers of leaping into numerical modelling without adequate knowledge of stress, rock properties or their combined effect on effective stress. An understanding of what is going to happen on mining should be possible from adequate measurements without resort to complex numerical models with inadequate input data. These are tools that may be used to refine design, but not at the early exploration stage.

INTRODUCTION

Which techniques are used to gather geotechnical information during exploration is very much dependent on what is going to be mined or constructed. Increasingly in mining the wish is to use mass extraction techniques such as longwall mining or block caving. These require a completely different approach to exploration to that of a shallow surface mine where rock discontinuities or soil behaviour are likely to be far more important than stress and intact rock strength. The exploration needs have to be tailored to what work or mining is going to be done and the likely rock materials that will be encountered.

Ground fluids may also become critically important in excavation stability. The term fluids is used to indicate both water and gas, as the latter may become extremely important in coals and carbonaceous rocks. Where fluids exist in rock there is a need to gather adequate information to determine their importance~~.~~ The effect of fluids is dependent on fluid pressure and how this pressure acts within the rock mass. If it needs to be drained, there is a need to know permeability so that the quantity of fluid and spacing of drainage boreholes may be designed.

What measurements of rock properties need to be conducted is very much dependent on the rock type and the nature of the structure or mine being designed. There is a need to think about how the rock is and will be stressed so that the appropriate tests may be undertaken to determine its behaviour. These measurements extend through simple point load testing, complex laboratory tests to geophysically derived parameters. These tests need to take into account the inhomogeneity and anisotropy of the rock mass at all levels.

As with all parts of an investigation the geotechnical model will depend on the adequacy of the geological model.

The following matters need to be considered in designing a geotechnical investigation:

- Where do we need to create openings that are permanent?
- What do we want to fall in as part of the mining process?
- What are the consequences of it falling in?
- How much rock can we remove without regional stability problems?
- Will there be a rock burst or an outburst problem?
- What are the consequences for the regional groundwater behaviour?
- Will we encounter gas or other petroleum fluids?

STRESS

A concept of what stress is and the definitions that surround it are required to understand the rest of this paper and therefore these concepts are introduced here.

Stress is the internal force per unit area within the rock. It is a second order tensor with the units of force per unit area. It is described by two subscripts. The first subscript describes the vector that is normal to the plane being considered while the second describes the direction of the stress by vector. Figure 1 shows a cube of unit dimension with stresses acting on the various planes. Here the numbered red arrows indicate the three vectors used to describe direction. The blue arrows indicate traction vectors of force on each surface. The traction vectors are resolved into stresses with the correct notation. Each surface may have one normal stress and a shear stress which is broken into two directional components.

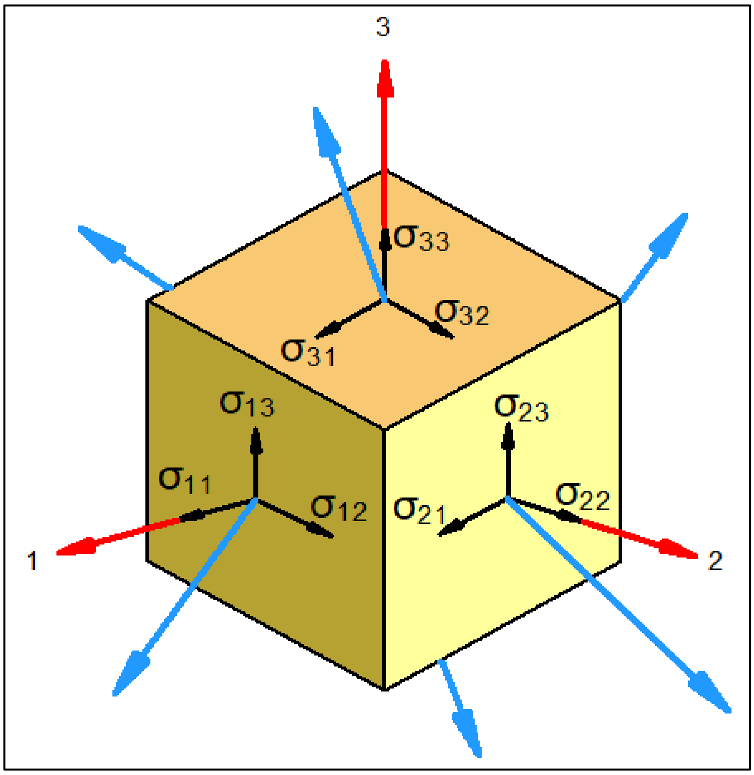

Figure 1. Showing a unit dimension cube with traction vectors and stresses.

The cube shown in Figure 1 may be rotated until the shear terms disappear. In this case the cube is aligned with the principal directions of stress.

In elastic theory strain may be directly related to stress as shown in Equation 1.

$$\{\varepsilon_{ij}\} = [C_{ijkl}]\{\sigma_{kl}\} \tag{1}$$

Where $\{\varepsilon_{ij}\}$ is a vector of strains which represents the elements of a symmetric matrix.

$\{\sigma_{kl}\}$ is a vector of stresses which represents the elements of a symmetric matrix.
$[C_{ijkl}]$ is the compliance matrix that relates stress and strain.

The compliance matrix is made up of functions of Young's moduli and Poisson's ratios and has 36 components that reduces to 21 through symmetry. Practically this is really too many components to measure and it may be reduced to 9 orthotropic values if some assumptions are made. One of the these is that shear does not lead to dilation. The number of parameters may be reduced further to a single value of Young's modulus and a single value of Poisson's ratio for an isotropic material.

Effective Stress

In rocks and other porous materials another term is used - effective stress. Effective stress describes the state of stress within rock or mass in the presence of a fluid under pressure. There are two ways of thinking about effective stress. One is in terms of the fraction of a surface within the mass which has the action of fluid pressure acting normally to it. This may be described in Equation 2.

$$\sigma'_{ii} = \sigma_{ii} - \alpha P \tag{2}$$

Where σ'_{ii} is the effective stress normal to a surface
σ_{ii} is the total stress normal to a surface
α is the fraction of surface area on which fluid may act
P is the fluid pressure at the surface

It should be noted that in a static fluid, its pressure may only act normal to a surface and therefore it can only affect the effective stress normal to the surface being considered; it does not affect the shear stresses, hence the subscript, *ii,* indicating this. The term α is treated as unity in soils and to be independent of direction. In rocks it is clearly dependent on the orientation of the surface under consideration with respect to rock discontinuities. If the surface being considered is shared with an open joint, then fluid pressure will act fully and α will be unity. If the surface crosses a discontinuity in an otherwise tight rock mass, then the value of α will be approximately zero. Thus the mapping of discontinuities is of extreme importance.

The other way in which effective stress may be considered is in terms of deformation. This is the poroelastic approach. In this case the effective stress is one that will, in the presence of a fluid under pressure, cause the same deformation as the stresses in a case where fluid pressure does not exist. This relationship is written in Equation 3 in terms of principal stresses for an orthotropic material.

$$\varepsilon_{ii} = \frac{1}{E_i}\sigma_{ii} - \frac{v_{ji}}{E_j}\sigma_{jj} - \frac{v_{ki}}{E_k}\sigma_{kk} - P\left(\frac{1}{E_i}\alpha_i - \frac{v_{ji}}{E_j}\alpha_j - \frac{v_{ki}}{E_k}\alpha_k\right) \tag{3}$$

Where ε_{ii} is the strain in the *i* direction.
σ_{ii} is the normal stress in the *i* direction
P is the fluid pressure
E_i is the Young's modulus in the *i* direction
v_{ki} is the Poisson's ratio associated with the –ve ratio of deformation in the *i* direction/deformation in the *k* direction brought about by loading in the *k* direction.
α_i is the poroelastic coefficient acting in the *i* direction.

Biot and Wills (1957) described α_i in scalar terms and this is called Biot's coefficient.

Rock is not necessarily orthotropic, but this assumption does deal with anisotropy, whereas the normal assumption of rock being isotropic is frequently hopelessly inadequate.

ROCK PROPERTY TESTING

Rock property testing may be thought of as being divided into those tests that measure the mechanical properties of the rock prior to failure, those tests which measure the failure load and some that measure the post failure behaviour. All these measurements are important with some taking precedence over others, depending on what is being constructed or excavated. What should be measured is really dependent on the expected failure mode. In this context failure can be defined by a state of deformation. Failure in a high-speed rail tunnel failure might be by excessive deformation of the track. If we are considering a longwall, then excessive deformation of the gate road pillars may be the concern. However, if the goaf does not form by suitable large deformation it will be of more concern. In a hard rock context, the elastic deformation that leads to a sudden loss of strength and energy release in the form of a rockburst may be the prime worry.

In shallow works with stiff strong rock, any failure will be associated with weaker structural features. In deeper, usually higher stress conditions the rock may fail without any weakening structures, while in some cases the energy release on failure may be significant, and post failure behaviour of the rock matters. Before any rock mass gets to failure it must pass through the pre-failure stress state and therefore its behaviour in this stress range must be determined for any meaningful modelling of rock behaviour to take place.

Uniaxial Testing

The testing of rock mechanical properties is frequently reduced to taking core samples which are then sent to a laboratory for uniaxial compressive testing with loading along the axis of the core. It is normal to test the rock to failure and record this. For cases where the use of deformational properties of the rock are needed, it is usual to measure the axial and circumferential deformation with stress variation and thus derive the Young's modulus in the axial direction and its associated Poisson's ratio. These measurements only apply to loading in the axial direction and take no account of the deformational characteristics with loading in other directions. The Poisson's ratio from such tests is frequently very high due to surface deformation. The results frequently show non linear behaviour which is often ignored. If cyclic loading is used in uniaxial testing it will reveal the loading and unloading Young's moduli. It will also show the permanent offset associated with each loading cycle. An example of a cyclic loading testing of a siltstone exhibiting non-linear behaviour and permanent deformation is shown in Figure 2.

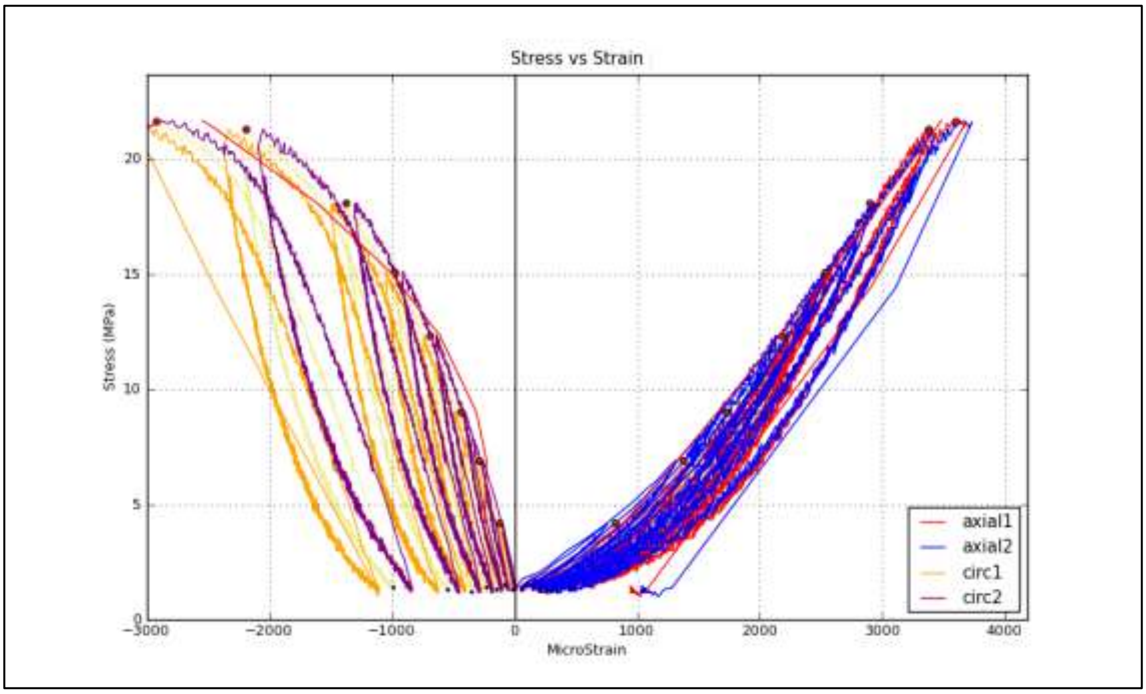

Figure 2. Results of cyclic uniaxial loading of a sandstone specimen. The x-axis shows the axial (+ve) and circumferential strain (-ve) while the y-axis shows the uniaxial stress.

Triaxial Testing for Failure Parameters

Triaxial testing of cores is normally conducted to determine the failure criteria of the intact rock. Such testing is normally conducted on core in a Hoek Cell. This device enables a confining load to be applied via an elastomeric sleeve to the sample while applying an axial end loading to failure.

The results of this are normally presented in terms of Mohr-Coulomb or Hoek and Brown (1988) failure parameters. The orientation of the core and the loading method where failure is brought about by higher stress than the confining stress means that failure can only take place in a restricted mode – sub vertical shear through the core. The Mohr-Coulomb failure criterion is described in Equation 4.

$$\tau_f = c' + \sigma_n tan\varphi' \qquad (4)$$

Where τ_f is the shear stress on a plane at strength based failure
σ_n is the normal stress to the failure plane
c' is the material cohesion in effective stress terms
φ' is the material friction angle in effective stress terms

Determination of Elastic Parameters Under Triaxial Conditions

Although rock deformational characteristics are seldom measured in triaxial testing~~,~~ Gray, Zhao and Liu (2018) have developed a testing process to measure triaxial parameters based upon the rock behaving in a nonlinear orthotropic manner. This is a major step forward from assuming that the rock is isotropic and linear. This testing also allows the poroelastic behaviour of the rock to be determined by gas injection. Figure 3 shows the real transverse Young's modulus of a sandstone core that has been tested in this manner. It is not linear and comparison with the other principal moduli show that the core is quite anisotropic. This is a normal characteristic for a sandstone or a siltstone where the ratio of lateral to vertical stiffness typically lies in the range of 1.2 to 2.5. It is also quite normal in these materials for the Young's moduli to increase with stress.

Another test which is of great use in determining the elastic properties of rock is a hydrostatic test, Gray, Zhao and Liu (2018). Here a rock fragment is fitted with strain gauges and is then encapsulated in silicone resin. This is then loaded hydrostatically. While the test cannot provide any absolute values of Young's moduli or Poisson's ratio, if any one of these is estimated (usually the mean value of Poisson's ratio) then the other values may be calculated. The test is particularly useful in determining non-linearity and anisotropy. It is a much simpler and lower cost test than a full triaxial test for rock elastic properties.

Uniaxial and triaxial tests for stiffness can give quite variable results from lamination to lamination of a sedimentary rock. It is therefore necessary to get an idea of this variability. This can only be achieved by using multiple strain gauges on a single sample or by the use of multiple tests.

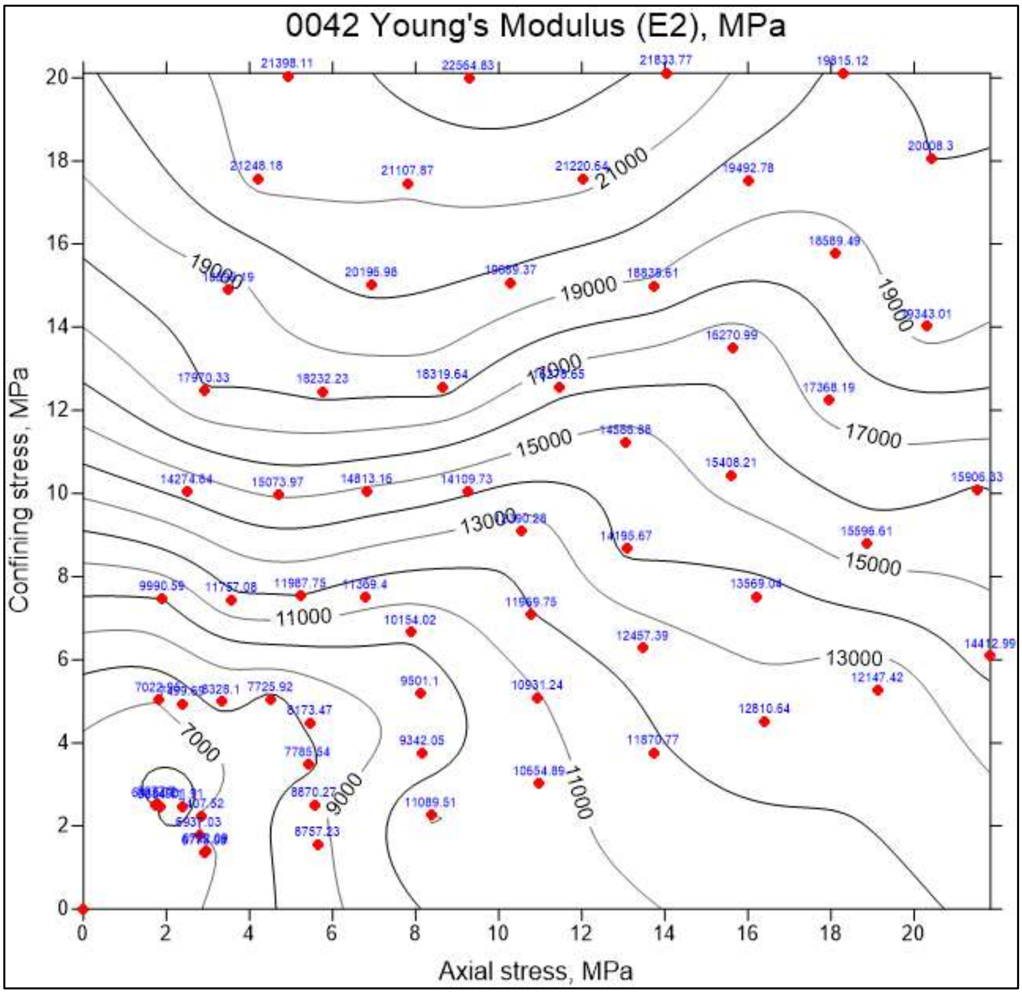

Figure 3. Transverse principal Young's modulus for a sandstone.

Shear Testing

Conventional shear testing (ISRM, 2007) involves mounting a joint within rock in a loading frame so that the joint can be normally loaded and sheared along its plane. It is complex to set up and therefore comparatively expensive. Its use is usually restricted to joints that are suspected of being a weak link in a rock mass. The normal shear test enables the joint to be normally loaded and sheared backwards and forwards to determine both the peak and the residual strength of the joint after any cohesion has been destroyed.

Because of the complexity of this test a quick shear test has been developed. Here a rock core is placed between two supports and centrally loaded. The setup to perform such a test is shown in Figure 4. The test must be conducted without a gap between the loading saddle and the supports to avoid the development of a bending moment in the core which leads to tensile stress being developed. Because the end supports are sitting on a spherical bearing the load on each surface being sheared is balanced. The rock will however, nearly always break at a single end. When the core has broken once it can be moved along and re-tested again. This enables multiple tests for the cohesion value (Equation 4) to be obtained from a single piece of core. The test method is used for laminated core with the laminations being essentially perpendicular to the core. The value provided by the test may be directly related to the cohesion term in the Mohr-Coulomb failure criterion.

Figure 4. Direct shear test on a core specimen.

Tensile Testing

In many cases the prime means of failure in a rock mass is tensile. Examples of this are caving zones including the goaf edge of a longwall. The most commonly used method to measure tensile strength is the Brazilian test (ISRM, 2007). This is in fact a compressive test across the diameter of the core and failure is partly by tension, induced by the geometry of loading, and partly by shear. Its analysis for tensile strength is also dependent on the core behaving in a linearly elastic manner. The test tends to predict a tensile strength value that is high.

It is possible to undertake true tensile testing of rock. The complication of how to hold on to a piece of rock in tension is fairly easily resolved by the use of high strength epoxy glues. Tensile loading may be applied easily along the axis of the core and equally so transversely to the core. In the latter case, the core is cut transversely to form a biscuit of rock which is then glued between two steel plates on each side of the biscuit leaving a gap between these as shown in Figure 5. This gap is monitored on each side by strain gauged C rings. The core biscuit is then loaded via these plates in a load balancing manner between each side plate and the load versus deformation value determined. Almost invariably one side of the core biscuit fails before the other due either to inhomogeneity in the rock or due to uneven loading leading to some degree of bending in the sample. The latter may be corrected by analysis of the uneven deformation of each side of the core to arrive at a more accurate tensile strength.

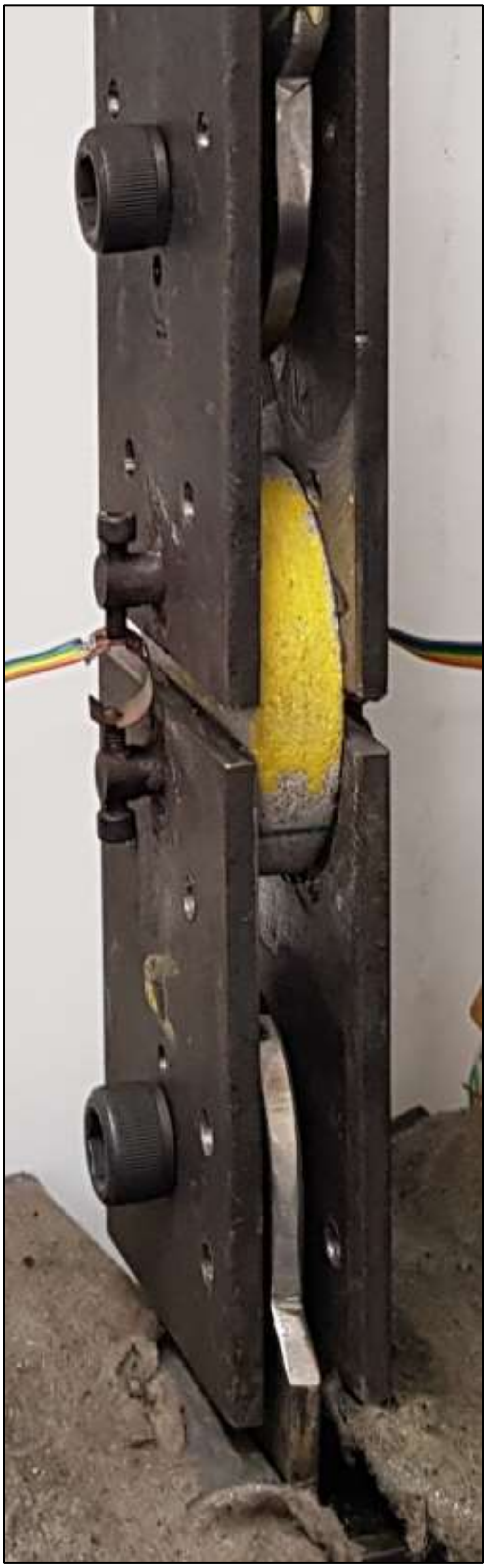

Figure 5. Transverse tensile rock test setup.

Dimensional Change Testing

The change in dimension of a rock can be brought about by temperature variations or the loss of fluid.

If mining is going to significantly drop the temperature of the rock, then the effects on stress may be important. In this case, the rock temperature should be measured and so should the thermally induced strain within the rock. If thermal effects are going to be important then it is also necessary to measure the specific heat of the rock and its thermal conductivity.

The other important case to consider is that of dimension change brought about by fluid loss. One of the most important aspects of this is the shrinkage of coal due to the desorption of gas. This can lead to destressing of the coal with gas loss, either naturally or by drainage with associated effects on permeability (Gray, 1987, 1 & 2). It is also important in terms of pillar stability, as drainage may cause the loss of lateral confining stress within the coal (Gray, 2014).

The measurement of dimensional change is accomplished by strain gauging a sample and subjecting it to changes in the appropriate condition, for example temperature. In the case of gas, it involves subjecting the sample to gas under pressure and waiting for equilibrium to establish itself. This is repeated in a series of steps.

Point Load Testing

The point load test (ASTM D5731-16) is another useful test for strength and mode of failure. The point load test involves loading a piece of rock between two points with a radius of 5 mm. The value of the failure load is recorded and related to the size of the specimen to arrive at a value of strength called a point load index. In anisotropic rock such as bedded sedimentary lithologies it can be used to test across, as well as along, the plane of weakness to highlight anisotropy. The point load itself induces a complex loading situation comprising compression, shear and tension. The correlation between point load and uniaxial compressive strength that is frequently used is therefore somewhat tenuous. Careful examination of the failure mode during the point load test can however reveal a lot about the strength of the rock. For example, point loading across the bedding plane, leading to shear on the bedding plane, indicates a weakness on that plane. Such a weakness should also be revealed by testing with loading parallel to the bedding plane. Point load values are corrected to that of a standard 50mm diameter core resulting in a value of $I_{s(50)}$ that is comparable across different core diameters.

Protodyakanov Index Testing

The Protodyakanov Index is a simple useful test to determine toughness of a rock (Wood and Gray, 2015) but also showing a reasonable correlation to tensile strength. It involves placing 20 – 30 mm lumps of rock or coal in a tube and dropping a slide hammer on to them. The amount of fines that are generated and are less than 0.5 mm diameter is measured after each blow. The method is originally from Russia and is used, amongst other applications, in the determination of outburst propensity and the suitability of coal for hydraulic mining.

ROCK STRUCTURE

The use of the term structure here extends through the range of faults to the investigation of joint sets to cleating within coals. It also includes bedding and schistosity. The structure within a rock mass at all these levels will generally affect how it will behave. Finding large faults is generally part of the exploration programme for all purposes, and smaller discontinuities, or the lack thereof, is just as important. The nature of the fault or joint surface is well recognised, and a great deal has been written on this subject. This tends to focus on the frequency of joints, their persistence and frictional properties, including roughness. The rock mass rating systems described later all take this information into account.

It is just as important to know whether joints exist or not. A lack of jointing may be of immense importance as it means that the rock may take tension. Generally rock mechanics dismisses the tensile strength of rock because all rock is regarded as jointed. This is not always the case and massive rocks, lacking in joints, pose their own set of problems when caving is required as part of a mining operation.

The nature of joint infill, or lack thereof, is also important. Joints that are open will conduct fluid and have fluid acting on their faces thus tending to push the faces apart therefore changing the effective normal stress to the joint surface. If the joint is filled, then how fluid acts within the joint will depend on the nature of the infill. Infills that lack porosity will not conduct fluid and therefore prevent the effects of fluid pressure acting within the joint. This means that complete infill such as calcite or quartz will prevent fluid pressure operating within a joint. Interestingly, undisturbed talc or chlorite also fall into this category. If movement on a joint occurs, bringing about dilation and the creation of space for fluid to act within, the situation changes as fluid may act within the joint. Joints that are filled with particulate matter, including clays, may expect to be subject to fluid pressure in the same manner as a soil.

Rocks exist that contain small scale jointing. The cleats in coal that are brought about by shrinkage are an example of these. Gray, Zhao, Liu and Wood (2017) describe the effect of stress changes on these and the way in which fluid may act within them. At low stresses, the cleats are open and fluid acts poroelastically within the mass while at higher stresses the effect is smaller. This effect is very important in the onset of failure in outbursts and coal bursts. Coals are immensely variable, and some show no poroelastic effects while others do to a large extent. This behaviour tends to be anisotropic. As its name suggests poroelastic behaviour also applies to porous rocks. In many tight crystalline rocks the poroelastic coefficient, α, is zero, while in other rocks it may extend almost up to unity. As described in the section on effective stress, the fluid pressure affects both the deformation and the strength of the rock mass.

Structure also affects rock strength. All sedimentary rocks show bedding to some extent. This may extend from bedding planes that are a few millimetres apart to those that are tens of metres apart. These planes frequently form a failure surface. This is hardly surprising as an examination of the bedding surfaces will frequently show mica lying flat on these surfaces. Anyone handling laminated core will be well aware of the ease with which many rock samples break along the laminations. This strength difference should be described by the geologist logging the core, but is seldom properly addressed in subsequent core testing and design parameters.

Not only do bedding planes affect strength but they also affect the stiffness of the rock. The stiffness to deformation parallel to the bedding plane is normally greater than that measured perpendicular to the bedding plane. The more highly laminated the material the greater this difference. There are exceptions to this.

LOGGING CORE AND GEOPHYSICAL LOGS

Borehole Geophysics

Borehole geophysics provides a very great deal of information on aspects of the geotechnical behaviour of rock. This comes principally from the use of sonic logs and acoustic televiewers.

Full waveform sonic logs measure the arrival of a pulse with time over several receivers. They enable the P, S and Stonely wave arrival to be detected. They can be used to derive the velocity (or slowness) of P and S wave transit through the rock. From this information can be derived the Young's modulus and Poisson's ratio of the rock. This derivation is based upon the use of equations that depend on isotropic rock properties. Where isotropic behaviour is not the case, the derived values will not be accurate but provide an indication of these values and their relative nature. The Young's moduli derived from sonic logs tend to be higher than those determined by physical testing, depending on the physical test method. The use of these logs does however provide a good indication of like groupings of material properties.

While not generally used in mineral exploration, the use of a sonic tool with a rotating dipole (Alford, 1986) gives an added benefit of providing information on the directional nature of the shear wave propagation velocity. This is a result of anisotropy within the rock mass which is considered to be related to the state of stress.

Acoustic televiewers provide an image of the borehole wall by sending and receiving acoustic signals through the drilling fluid. When the hole is dry they must be replaced by optical televiewers. A view of the borehole wall provides information on rock structure which may include faults, joints and bedding. Because it provides this in oriented form, it substantially removes the need for core orientation or the drilling of inclined holes drilled to obtain a gravitational component for orientation. It is possible to match the results of core logging with the image from the televiewer to arrive at the true core orientation.

Acoustic televiewer images also provide information on borehole breakout. This is the compressional failure of the borehole wall brought about by stresses of sufficient magnitude acting around the stress concentration of the borehole to cause wall fracturing. This breakout occurs perpendicular to the major stress direction. In some highly directional stress fields it is also possible to see a tensile fracture of the rock take place in the same direction as the major stress.

Other geophysical log types may be used to identify rock type or markers within the sequence. In a sedimentary environment resistivity, natural gamma and gamma-gamma density logs are of particular use. Other rock types may require other tools such as electromagnetic induction, induced polarisation, magnetic susceptibility or neutron logs.

Core Logging

While borehole geophysics is of immeasurable value and is the only information that is generally available to the petroleum industry, it is most beneficial to have some real core to relate to the geophysical behaviour. Geophysics can then be used to greater advantage and certainty. Getting core out of the ground enables far more detailed description of the lithology and detailed examination of the structure within it. It also provides rock samples for testing.

Core needs to be logged to a method so that all the relevant parameters are acquired. This poses real challenges and different groups have developed multiple different methods. The authors have taken the approach that the core should be logged within the split from the triple tube core barrel before disturbance. Logging should be undertaken in three passes. The first is for a full lithological description, including mineralogy, grain size, colour and any other identifying feature. The second is measurement of the core structure orientation within the split. This is achieved by taking three length measurements within the split: one at the far side, one at the near side and one at the top of the core as it lies within the split. This provides relative orientations of any structure within the core. This information can then be converted to absolute orientation by comparing core logging results with televiewer images. Finally the core is removed from the split to reveal each structural feature, its roughness and infill. At the same time pieces of intact core may be checked for strength by testing by hand or hammer blows required to break them. Sometimes doing this will reveal more about the lithology as fracture surfaces reveal the fresh rock in more detail. The use of point load testing is also useful as a comparative strength indicator.

After logging the core is sampled for the testing of material properties. The core is then broken (or preferably cut) into 1 m long segments and placed in core trays.

Photographs are taken of the core at 0.5 m intervals while the core is in the split and of the full core trays after placement there. This photography must be conducted with a camera at a fixed distance and orientation from the core. Photographs provide a reference for dealing with uncertainties in description that invariably arise between the geologists who log the core.

Geotechnical Indexes

Several core logging systems have been designed to assist in empirical geotechnical design. These started with RQD which was a measure of the level of core intactness based on each core barrel length. One of the key limitations of RQD is this relation to the sampled core barrel of material and not the units of rock within the core barrel. This matters in longer core barrel lengths. Other systems have followed for specific engineering applications. These are summarised in Table 1.

Table 1: Commonly used Rock Mass Classification Systems

	Originator	Country of Origin	Method	Application Areas
Rock Quality Designation (RQD)	Deere et al., 1967	USA	Using only intact core length	Core logging, tunnelling.
Rock Mass Quality (Q)	Barton et al., 1974 (last modification 2002)	Norway	Product of different factors	Tunnels, mines, and foundations.
Rock Mass Rating (RMR)	Bieniawski, 1973 (last modification 1989-USA)	South Africa	Sum of single parameters	Tunnels, mines, slopes, foundations.
Geological Strength Index (GSI)	Hoek and Brown, 1997	Canada	On the basis of joint frequency and joint properties	All excavations.
Coal Mine Roof Rating (CMRR)	Molinda and Mark, 1994	USA	Sum of single parameters	Coal mine roof support design.
Mining Rock Mass Rating (MRMR)	Laubscher, 1975	South Africa	Sum of single parameters	Evaluation of block caving.

With the exception of RQD all these systems involve the examination of the joint surface as a major control on stability. This, combined with some measure of joint frequency, leads to a stand up time or fall down time relationship or some rock strength modifier (GSI). There is a major problem with these systems in that they characterise water related strength reduction by the examination of the state of water ingress into underground openings that already exist. In exploration, the openings are not yet in existence, and some improved method should be used based on core from exploration holes. The suggestions made are given below. These have some consistency of approach as they are based upon the concept of effective stress in a fracture. However, as with any part of any empirical design, they should be treated with caution.

Table 2: Original method of calculating the Joint water reduction factor (Jw) for Q-system

Joint Water Reduction Factor	**Jw**
A. Dry excavations or minor inflow (humid or a few drips)	1
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66
C. Large inflow or high pressure in competent rock with unfilled joints	0.5
D. Large inflow or high pressure, considerable outwash of joint fillings	0.33
E. Exceptionally high inflow or water pressure at blasting, decaying with time	0.2 – 0.1
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1 – 0.05
Notes: i) factors C to F are crude estimates. Increase Jw if drainage measures are installed.	
ii) Special problems caused by ice formation are not considered	
iii) for general characterisation of rock masses distant from excavation influences, the use of Jw=1.0, 0.66, 0.5, 0.33 etc. as depth increases from say 0-5m, 5-25m, 25-250m to >250m is recommended, assuming that RQD/Jn is low enough (e.g. 0.5-2.5) for good hydraulic connectivity. This will help to adjust Q for some of the effective stress and water softening effect.	

The suggested modified method of calculating the Joint water reduction factor (*Jw*) for Q-system is given in Equation 5.

$$Jw = 1 - 0.95\left[\frac{hw}{z}\left((1-j) + (j\alpha)\right)\right] \quad (5)$$

Table 3: Original method of calculating the Groundwater (*A5*) for RMR system

General condition	**joint water pressure/major in situ stress**	**Ratings**
Flowing	>0.5	0
Dripping	0.2-0.5	4
Wet	0.1-0.2	7
Damp	0-0.1	10
Completely dry	0	15

The modified method of calculating the Groundwater (A5) for RMR system is given in Equation 6.

$$A5 = 15 \times \left[1 - \frac{hw}{z}\left(j\alpha + (1-j)\right)\right] \quad (6)$$

Table 4: Original method of calculating the Joint water reduction factor (Jw) for CMRR system

		Condition	**Rating**
ADJ3	Ground Water Adjustment	Dry	0
		Damp	-2
		Light drip	-4
		Heavy drip	-7
		Flowing	-10

The suggested modified method of calculating the Joint water reduction factor (*ADJ3*) for CMRR system is given in Equation 7.

$$ADJ3 = -10\left(\frac{hw}{z}\left(j\alpha + (1-j)\right)\right) \quad (7)$$

In Equations 5 to 7 the following applies

hw = water head

z = depth

j = joint infill area ratio (fraction of joint that is filled)

α = poroelastic behaviour of infill material (see table 5 for more details)
Crystalline infill and alteration material =0 (eg. calcite, quartz, talc)
Carbonaceous material = 0.3 to 0.6
Material placed by transport or disturbed = 1 (eg sand, silt, gouge, clay).

STRESS IN ROCK

The concepts of stress have previously been introduced. The deformation of rock is dependent on the state of stress and the elastic properties of the rock mass. When the rock ceases to behave elastically it usually progresses into a strength based shearing or tensile failure. Bucking failure may also occur, though this is principally a structural stability phenomenon associated with laminated or jointed rock.

The state of stress of rock is frequently complex. In sedimentary sequences material is lain down and consolidates. During this period it can be expected to behave as a soil with internal stresses that lie between the active and passive states of lateral stress and with a vertical stress which is essentially gravitational. The fluid pressure will act within the soil mass with a poroelastic coefficient of unity. If erosion takes place the properties of the soil change to a one which is over consolidated, meaning it has been loaded beyond its current overburden stress. This term is particularly applied to clays. The way in which any soil is laid down affects its packing and mechanical properties. Notable examples are sands that in a loose state will tend to consolidate with shearing, as in an earthquake, and tend to liquefy if saturated. Sands which are not loose will tend to dilate on shearing and draw in fluid thus lowering pore pressure and tending to make the mass stronger. Such phenomena also occur in clays. The quick clays of Norway, which readily collapse with slight disturbance, behave completely differently from over consolidated London clays which will stand for years relying on negative pore pressure to hold the mass together, and then suddenly collapse when this increases.

With lithification these soils, of vasty differing initial properties, form chemical bonds and become rock-like. The mass becomes more elastic and loses some pore space. Its behaviour is then quite different. Diagenesis may take place and the mineralogy of the rock changes further. If sufficient heat and pressure exist metamorphism may occur, taking these changes to another stage. With each chemical change there can be expected to be some volume change and with it a change in stress state. Changes in temperature of the rock mass will also cause large stress changes depending on the confining boundary of the rock in question.

In addition to these processes there may be strains imposed upon the rock mass by geological deformation. This deformation includes gross tectonic movement but on a local scale folding and in particular faulting. Faults relieve stress during seismic events but may cause stress concentrations near their tips. Frequently the ongoing tectonic process builds up stress on a fault, leading to a seismic event associated with more movement with possible tip extension. A cycle may be set up where stress builds up and is relieved by fault movement so that the stress in the area in question is quite different depending on which part of the stress and failure cycle the rock mass is in. This mechanism may lead to quite varying stresses over a mine sized area, as was observed by Gray, Wood and Shelukhina (2013).

One of the concepts used to help describe the varying stress distribution through strata of differing stiffness is that of tectonic strain (Gray, 2000). If we assume that the vertical stress in rock is overburden, then in a zero lateral strain environment and elastic rock this will lead to a horizontal stress. The tectonic strain is the calculated horizontal strain required to take the rock mass from the zero lateral strain state to that currently existing. Frequently the stresses in bedded strata of differing stiffness vary greatly but the tectonic strain is relatively even through the sequence, or at least part of it. The tectonic strain model is disturbed by disconformities or faulting.

Coals, and presumably to a lesser extent, carbonaceous shales, show another effect. This is shrinkage of the coal with loss of gas and water. This shrinkage relieves lateral stress while the self weight stress remains acting vertically. This is an extremely important feature of coals as it affects their rock mechanics characteristics and permeability quite drastically.

Igneous and metamorphic rock masses may show even more complexity in their stress distributions than sedimentary rocks because they have undergone a wide variety of processes to reach their current state. Dykes and sills are indicative of the state of stress at the time of their intrusion. They behave like giant hydrofractures with a highly viscous fluid. This means that they will propagate in the plane of minimum stress. The exception to this is where pre-existing structures such as faults require less pressure to open compared with that required to break intact rock. These, like any other hot intrusive rock, will shrink with cooling and reduce in stress.

As with sedimentary rocks, the stiffer the igneous or metamorphic rock, the more it will change stress with tectonic strain.

The Measurement of Stress in Rock

The measurement of stress is one of the key elements in planning underground activity. It is important because stress in combination with the rock’s elastic properties will lead to deformation on excavation. This deformation will be associated with a stress redistribution that may lead to strength based failure and greatly increased deformation. Some deformation will not be acceptable, and in the case of caving operations large deformation is essential.

There are three techniques to determine stress that have a sound physical basis and a number of marginal ones that may help in the absence of these. The three are: hydrofracture, overcoring and the examination of borehole breakout. These are discussed in more detail in Gray (2018) but are summarised here. The standard analysis of all these methods is based on the assumption that the stress distribution around the borehole is that which would exist if the rock were linearly elastic and isotropic. This may be quite inaccurate if the rock is non linear or anisotropic. It is possible to analyse the results if the real rock properties are known but this requires substantially more effort, warranted if stresses are likely to be critical for a project.

Hydrofracture

Hydrofracture involves sealing a section of borehole, usually with packers, and raising the pressure of the fluid within the sealed section until the rock fractures. Fluid is then pumped into the fracture zone. A valve is then closed to seal the test section of the hole, the pump is stopped and the decay in pressure is observed. The minimum stress is considered to be the pressure at which the fracture closes. This is determined by analysis drawn from the petroleum industry (Barrie, Barrie and Craig, 2009). Pumping is then recommenced to re-open the fracture. The assumption is then frequently made that the re-opening pressure can be used with the closure pressure to arrive at the maximum principal stress. This assumption seldom provides a good major stress estimate. The analysis is based upon the fracture being coincident with the borehole axis, something that does not always occur. Practically most hydrofractures are initiated by the packers being pumped up to too high a pressure. This can be avoided with the correct techniques and equipment.

A derivative of hydrofracture is hydrojacking. This involves pressurising a natural joint to re-open it and determine its closure pressure, and hence the stress normal to the joint.

Overcoring

Overcoring is a process where a hole is drilled to test depth. In most common form a of overcoring a pilot hole of smaller diameter is then drilled ahead of the main hole, and an instrument to measure changes in the dimension of the pilot hole or strains on the pilot hole wall is then inserted. The pilot hole is then drilled over by the process of overcoring during which the instrument records the changing diameter of the pilot hole. The instrument is recovered with the core. The measured deformations can be related to the stress via the rock's elastic properties. The general analysis technique for such a test method is based on isotropic, linear rock behaviour. Where this does not exist more care must be taken to measure the rock behaviour including its orientation. Where the rock does not behave elastically the method cannot be used. This particularly applies to situations where the pilot hole wall fails due to the stress situation.

Borehole Breakout

Borehole breakout is a case where the rock has gone beyond the elastic state and breaks by compression along two sides of the borehole wall because of the stress concentrations there. It is possible to determine the breakout orientation and width within a borehole by the use of an acoustic scanner. If borehole breakout does occur and the minimum stress has been determined by hydrofracture and if the compressive strength of the rock perpendicular to the hole is known, then it is possible to estimate the major stress based on linear elastic theory of stress distribution around a borehole. The sticking points in analysis are the lack of linear elastic behaviour and inadequate knowledge of the compressive strength of the rock in the horizontal direction. The oilfield approach of correlating strength with a sonic log is not really adequate for this purpose as this log primarily measures axial stiffness. Despite this, there are a range of papers which purport to be able to solve for the state of stress from breakout information alone. This is not possible as there is insufficient information to do this. Big data or artificial intelligence approaches do not change this.

Sometimes in extremely directional stress fields it is possible to see a consistent tensile fracture in the acoustic scan of the borehole wall. This is an important indicator of unusual stress conditions.

If breakout does occur it is a warning of things to come on excavation.

Other Methods for Stress Determination

There are a range of other less direct methods for the determination of stress in rock. These include anelastic recovery, the reloading of rock to listen for acoustic emission and the measurement of core ovality. The last of these has a good physical basis but only when used with an appropriate core bit which does not regrind the outside of the core after cutting. In this case the ovality of the core can be related by elastic theory to the stress difference between the major and minor stress directions perpendicular to the hole.

Stress Measurement Programmes

Measuring stress in rock is difficult and tends to be expensive. Real measurements must be incorporated into the drilling programme. For these reasons there is a tendency to minimise the number of tests and to try to extract all the information from an inadequate sample. The fundamental problem with this is that stresses vary and may change rapidly from rock type to rock type or with proximity to faults, folds or disconformities. It is therefore far more valuable to make as many measurements as possible using as many methods as are appropriate so that an overall assessment of stress and its variability with location and stratigraphy can be made.

FLUIDS IN ROCK

Fluid Storage in Rock

Rock contains fluids that may range from heavy oils, light hydrocarbons, gases, connate water or readily exchanged water in an active aquifer. The storage of these fluids may be in pore space or in fractures. It may also be by adsorption of lighter hydrocarbons into carbonaceous material as is the case with coals or shales. Because of the pressure and temperature variations with depth the pressure, volume and temperature (PVT) behaviour of fluids in liquids is important, as are the changes in viscosity with changing conditions. Gas may be released from solution in oils or water on recovery from high pressures. Sampling the fluid contained within the rock for compositional analysis is therefore most important.

Fluids are released from rock in four ways. The first is by lowering the pressure so that the rock pore volume reduces and the fluid volume increases. The second is by the drainage of the volume of pore or fracture volume of the fluid. The third is by the release of one fluid from another, typically gas from solution in water or oil and the fourth is by desorption of gasses from the solid material.

The petroleum industry refers to storage in terms of porosity and compressibility of the rock mass and fluid with respect to pressure. Hydrogeologists refer to storage in terms of storativity and specific yield. The first describes the release of fluid per unit area of a confined aquifer with respect to a change in head while the latter describes the drainage of fluid from an unconfined aquifer.

The petroleum industry tends to rely as much as possible on geophysical logs to determine porosity assisted by the testing of core in a porosimeter where core is retrieved.

Fluid Movement In Rock

The movement of fluids in rock can be primarily described as Darcy flow with fluid being driven by a potential gradient. For an isotropic rock this is described in Equation 8.

$$\frac{dq}{dA} = v = -\frac{k}{\mu}(\nabla p + \rho g \nabla z) \tag{8}$$

Where $\frac{dq}{dA}$ is the volumetric flow rate per unit area in the direction of the flow vector
- v is the apparent velocity of flow
- k is the absolute permeability
- μ is the dynamic viscosity of the fluid
- ∇p is the derivative of pressure with respect to direction
- ρ is the density of the fluid
- g is gravitational acceleration
- ∇z is the derivative of the downwards component of direction with respect to direction

In this case the potential is made up of pressure and gravitational components. Hydrogeologists reduce this to measurements of hydraulic head (*h*)and hydraulic conductivity (*K*) which incorporates density and viscosity so that the equation is simplified to the form shown in Equation 9.

$$\frac{dq}{dA} = v = -K\nabla h \tag{9}$$

The relationship between hydraulic conductivity and absolute permeability is given in Equation 10.

$$K = \frac{k\rho g}{\mu} \quad (10)$$

Where different phases (water, oil, gas) exist in a rock mass each phase impedes the flow of the others by physically blocking the passages within the rock mass assisted by the effects of capillary pressure. The presence of different phases requires the consideration of this relative permeability behaviour.

Pressure Measurement

What is most important from a geotechnical viewpoint is the measurement of pressure of the fluid in rock and the way in which the fluid will act within the rock mass to change effective stress. Pressure measurements may be made by sealing a section of borehole and measuring the stabilised pressure within that section. The sealing may be accomplished by the use of conventional gravel pack piezometer installations, packers of varying types or by the use of cement grouts. While it has become fashionable to directly cement pressure transducers into a borehole with a permeable cement grout this frequently leads to a lack of connection with the formation or to intra-connection within borehole. A better method of cementing in pressure transducers into a borehole is described by Gray and Neels (2015). In this method transducers are cemented into the hole and water is injected through the filter tip on the transducer to displace partially set cement grout and connect the transducer to the hole wall. This method permits multiple, testable, pressure sensing locations to be made within the borehole while maintaining sealing between pressure sensing points.

Pressure measurement may also be accomplished during permeability testing as is described below.

Permeability Measurement

As can be seen from Darcy's law (Equations 8 and 9) the flow through rock is dependent on potential (pressure+density*gravity) gradient and its permeability. If flow needs to be determined or drainage behaviour modelled, it is necessary to measure both pressure and permeability. Indeed it is not possible to measure permeability without the measurement of some potential difference. Practically this can be achieved by measuring pressure change with time.

Most permeability measurement is made in boreholes. During exploration these are conveniently the holes which are being drilled for exploration, by coring or chip sampling, rather than production wells. Unfortunately, it is common practice to conduct tests in these while flow is coming out of or being injected into the hole. This causes fundamental problems because much of the pressure difference between that existing in the well and that in the rock mass adjacent to the well is associated with near well bore losses. These losses may be due to plugging of the well bore by muds or by the effects of stress concentrations around the well bore closing fractures. This effect is dependent on the pressure within the rock near the borehole. As the pressure in the well bore changes during flow periods so does the near well bore loss. This is further complicated in the case of injection tests because it is difficult to avoid blocking the near well bore area further by muds or particulate matter in the borehole fluid.

One way to get accurate permeability measurement is to measure the pressure changes associated with borehole flow in a piezometer in an adjacent hole. This has the added advantage that it enables measurement of the storage characteristic of the rock mass. If however a single hole test is being performed it is really essential to derive the permeability from pressure changes after the end of the flow period. By doing this the well bore loss effects on the permeability measurement may be avoided. A convenient way to achieve this is by the use of a drill stem test (DST) technique used by the oil and gas industry. In this the test zone is straddled between packers and pressure stabilisation allowed to take place. A valve is then opened to allow flow from the rock into the substantially empty drill string. After a flow period the valve is closed and the pressure build-up is monitored. By using the flow and analysing the transient pressure build-up period it is possible to reliably determine permeability. By extrapolating the pressure build-up it is also possible to arrive at the static fluid pressure.

There are variants of this test. It is possible to inject rather than produce from a test zone. Apart from complications with injecting well fluid rather than the reservoir fluid the analysis is the same. It is also possible to conduct a drill stem test in one hole and to observe its transmission in an adjacent one. If this pulse testing is conducted between multiple holes it provides a basis for finding both the anisotropy and inhomogeneity of the test area (Gray, 2015).

One test that is frequently used in the mining and civil engineering areas in an attempt to determine permeability is the packer test. In this a zone of a hole to be tested is sealed between packers, or between the packer and the end of the hole. Water at a controlled pressure is applied at surface. The flow rate is then permitted to apparently stabilise, usually only over 5 minutes, and is then measured. The value derived from this test is a Lugeon which is given in Equation 11.

$$Lugeon\,Value = \left(\frac{q}{L}\right)\left(\frac{P_o}{P}\right) \tag{11}$$

Where q is the flow rate (litres/minute)
L is the length of borehole being tested
P_o is the reference pressure (MPa)
P is the test pressure at surface (MPa)

This contradicts every rule of permeability measurement as it fails to take into account any pressure difference, as there is no measurement of initial head or pressure. In addition, the analysis period of the test is a flow period and so suffers from well bore effects. Also, the assumption of steady state behaviour is made in the analysis but never in fact exists. Nevertheless it is frequently incorrectly specified as a measurement of permeability. Tables even exist that purport to correlate the value in Lugeons with permeability. The test technique was in fact developed as a measure of whether rock could be grouted. If anything, the test is an indication of the near well bore loss behaviour and little to do with the permeability of the rock mass.

The packer test can readily be replaced by a DST with not much more complication and thus enables meaningful measurements to be made. One critical difference though is the need to wait for the tested rock mass (reservoir) to yield its information. The test pressure recovery cannot be hurried and may take from an hour to 24 hours to yield useful results. If more time than this is required the permeability is usually too low to be of interest. Gray (2017) describes a wireline tool for this purpose which can be used with HQ coring. This system has been in use for some years. A closed chamber variant of this tool also exists, which is suitable for low permeability testing and the collection of formation fluids.

The Measurement of Gas Content in Rock

The gas content of rock is important not only from the petroleum viewpoint. In coal mining it is essential to know gas content for ventilation and gas drainage design. Gas pressure is also very important in failures of coal, particularly outbursts. Gas pressures in carbonaceous rocks are related to the gas content and the sorption isotherm of the rock. The sorption isotherm is the relation between the gas pressure and stored volume by adsorption. It ignores free gas in pore space. Sometimes it is necessary to determine both free gas and adsorbed gas. Free gas under pressure is available to do work immediately, sometime with disastrous consequences. For example the presence of hydrogen in deep metal mines is well documented, and outbursts do occur from porous kimberlite.

The petroleum industry approach to gas content is based on determining the porosity and gas pressure. The coal seam methane or coal mining approach is to collect core samples and to place them in canisters for desorption. Following the bulk of desorption the core is crushed to enable it to rapidly release residual gas. Assumptions are made to arrive at the gas loss on core retrieval before it is placed in the canister. The gas content is usually reported as the volume of gas per mass of coal. The shale gas approach is to determine the total organic carbon in the shale and to relate that along with the reservoir pressure to a gas content via some typical isotherm.

Alternatives that lead to the collection of all gas are core barrels that capture gas. These may be pressurised or alternatively have adequate volume to hold any gas released on core recovery.

Another alternative involves drilling an open hole in overbalanced mode so that the drilling fluid pressure exceeds the fluid pressure in the formation. In this case the mud and cuttings from the hole are circulated out of the hole. In transit up the hole annulus free gas will expand and gas will desorb. At the hole collar is a rotary seal between the drill rod and the casing so that all the gas, mud and cuttings are forced through a pipe into a cyclonic separator. The gas volume can then be measured from the top outlet of the separator while the mud and cuttings pass out the bottom. Cuttings can be separated from the mud on the shaker. The cuttings can then be further desorbed so that the entire gas content may be determined. This approach is further described by Gray, Singh and O'Brien (2013). This approach is particularly useful in determining the gas content of thick sequences.

DIRECTIONAL DRILLING

Directional drilling has a particular use in exploration to determine the geology, particularly in assessing a tunnel alignment or a complex orebody. The methods of directional drilling have changed considerably over the years and have mostly come from the oil industry. Directional drilling in this industry started with jetting bits that preferentially eroded one side of a hole, it then progressed to the use of down hole mud motors and has now progressed further to rotary steering systems that will drill far further without stick-slip drilling problems. In favourable conditions the oil industry has drilled holes with 12 km laterals which were completed in 12 days using rotary steering systems. In the mining or civil exploration directional coring is possible but much slower than open hole directional drilling. Uncored directional holes may yield a great deal of geotechnical information through being subsequently logged geophysically. Here, as in vertical holes, the most important logs are sonic and acoustic televiewers.

CONCLUSIONS

In this paper an endeavour has been made to cover a very wide field of information that might be needed for geotechnical purposes in an exploration operation. Some emphasis has been given to understanding the fundamentals of stress, including effective stress, and material properties. It emphasises the fact that rocks do not behave in a linearly elastic, homogeneous, isotropic manner. This fact is almost universally ignored in numerical models of rock excavation. The paper introduces a number of new tests for rock to measure non linear, anisotropic and poroelastic behaviour as well as simple methods to measure tensile strength and shear strength. The paper discusses logging methods and methods to measure stress, fluid pressure and permeability. It introduces the concept of Rock Index methods with some modifications to their approach to groundwater to enable these to be used in exploration rather than in a mine opening. However rock index approach is considered no more than a guide before more detailed analysis and design. Finally it looks at gases in rock which occur in many instances.

The keys to good geotechnical information recovery during an exploration operation are the understanding of operational constraints to mining, having some idea of the rock mass and potential local variation before starting and using the most appropriate exploration techniques.

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