Stress in the Ground

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Abstract

This paper examines stress in the ground, the processes used to measure these, and the bases for working out their distribution. The paper is primarily focused on stress in rock, with more attention given to sedimentary rocks.

What is Stress?

Stress is by definition the force acting on a unit area. While this has the same dimensions as pressure there is a difference – stresses may vary with direction. Unlike fluid pressure, stress may act in a direction which is not perpendicular to a surface. In this case it may be divided into components that are normal and parallel to the surface in question. The latter parts are the shear components. This directional aspect of stress is embodied in the tensor notation σ_{ij} where i is the vector normal to the plane in question and j is the direction in which the stress acts. Where $j = i$ the stress is normal to the plane i, and where $j \neq i$ the stress is a shear stress acting on the plane defined by the vector i.

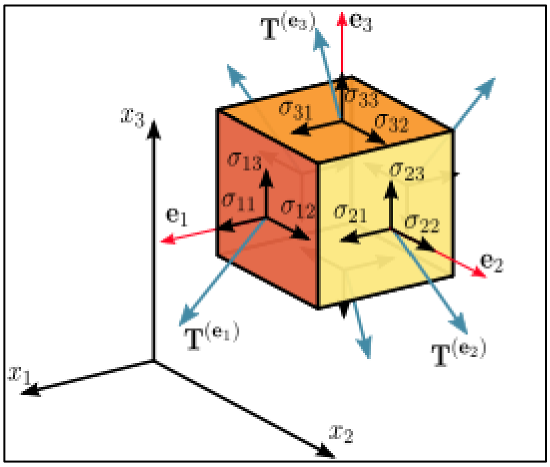

Figure 1. Showing surfaces on a cube of unit dimension described by vectors e_i perpendicular to these, surface tractions (forces) $T^{(e_i)}$ and stress tensors σ_{ij}. From Wikipedia.

The product of stress and the area of a surface is force. The net force on the faces of any body and field forces (normally gravity) must be zero or that body would accelerate. As the ground is not generally accelerating within the frame of reference of our planet it is generally in equilibrium, albeit with some complex stress distributions stopping it from doing so.

A full stress tensor has nine components which can be reduced to six as the shear stresses $\sigma_{ij} = \sigma_{ji}$. The tensor can be re-described into principal stresses which are major, intermediate and minor values of stress, oriented in such a way that the shear components are zero.

What is Effective Stress?

In materials such as rock and soil there are frequently fluids at pressure that fill pores and fractures. These fluid pressures are very important as they affect the effective stress within the ground. Equation 1 describes effective stress.

The simplest definition of effective stress is given in Equation 1, (Gray 2017).

$$\sigma'_{ij} = \sigma_{ij} - \delta_{ij}\alpha_i P \tag{1}$$

where:

σ'_{ij}	is the effective stress on a plane perpendicular to the vector *i* in the direction *j*.
σ_{ij}	is the total stress on a plane perpendicular to the vector *i* in the direction *j*.
δ_{ij}	is the Kronecker delta. If $i \neq j$ then $\delta_{ij} = 0$, while if $i = j$ then $\delta_{ij} = 1$.
α_i	is a coefficient affecting the plane perpendicular to the vector *i*. It lies between 0 and 1.
P	is the fluid pressure in pores and fractures within the rock.

The Kronecker delta term is used because a static fluid cannot transmit shear.
The directional subscript indicating direction in α_i is not usual practice where, for measurement reasons, only a scalar value is obtained.

Equation 1 indicates that positive fluid pressure acts to reduce the effective stress. In a soil it is generally simplified to Equation 2.

$$\sigma' = \sigma - P \tag{2}$$

Here no direction is ascribed to the stress (generally because it is not known) and the fluid pressure P acts in all directions in the soil over the full extent of any surface chosen within it. This may be simply understood for a granular soil with point contact. Experimental evidence shows that equation (2) applies to clays as well as granular soils and other particle types in between.

For the case of rock we need to revert to Equation 1. Here the value of α_i is generally not unity. There are two entirely different approaches to consider what α_i actually is. In one case we may consider it to be a fraction of a surface area on which fluid acts. This might be used to describe the open part of a planar joint in an impervious rock mass which has been partially filled with a sealing crystalline mineral such as calcite. The other way to look at α_i is in terms of poroelastic behaviour. This describes the behaviour of a soil or rock in terms of how fluid pressure affects its deformation. In this form $\alpha_i$is Biot's Coefficient (Biot and Wills, 1957).

To help describe the concept of poroelastic behaviour in rock we might think of the rock as being a giant sponge with some open and some closed pores. If we consider the sponge as having some elastic behaviour, when it is stressed it changes dimension and will recover its shape when it is de-stressed. If we then stress the sponge and then inject fluid pressure at the same pressure as the external stress and measure the amount of recovery of dimension obtained, then the fraction of dimension recovery is described by Biot's Coefficient. Because of the elastic behaviour of the sponge, Biot's coefficient is describing the effect that pressure has on deformation.

Mathematically, the relationship between the change in strain of a rock with orthotropic properties exhibiting poroelastic behaviour with a change in stress is described by equation (3).

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

where:

$\Delta\varepsilon_{ii}$ is the change in strain in the *i*th direction under the influence of the three Principal effective stresses in the *i*, *j* and *k* directions.

E_i is the Young's modulus in the *i*th direction.

v_{ij} is Poisson's ratio describing deformation in the *j*th direction due to stress in the *i*th direction.

$\Delta\sigma_{ii}$ is the change in total principal stress in the ith direction.

ΔP is the change in fluid pressure.

α_i is Biot's coefficient in the *i*th direction.

Fluid Pressure

As fluid plays such an important part in effective stress it is important to consider the fluid and what pressure it might be at. Ultimately it is essential to measure fluid pressure.

At the ground surface the fluid pressure is generally that of the air or of the hydrostatic pressure of some body of water above the ground surface. In the vadose zone there is both air and water and the difference in pressure between these is determined by the capillary pressure which is dependent on the soil or rock type and the degree of saturation. At low saturations capillary pressures may be much greater than atmospheric pressure. Therefore the water pressures in unsaturated ground are theoretically well below absolute zero pressure, thus indicating that the water is acting in tension without vapourising. This is surprising but important, especially in clays which may develop very high capillary pressures.

At depths below the vadose zone, groundwater pressures tend to rise hydrostatically. Artesian pressures or perched water tables may exist but these are the exception. With increasing depth fluid pressures may exist that are well above hydrostatic. The upper limit on these pressures is the minimum principal stress in the rock. Above this pressure the rock would open up and the fluid leak off. Also at greater depths, the fluids in the ground may include petroleum liquids and gasses.

Does stress matter?

The stresses in the ground affect its potential deformation. This deformation may be around an underground opening, an excavation or in an earthquake. Failure may be defined as excessive deformation. Excessive deformation in a high speed railway may be 5 mm misalignment in 10 m length of track. Excessive deformation may also be a major open pit wall collapse or a fault scarp caused by an earthquake. In the latter two cases the rock stress will have exceeded its strength, leading to a decrease in strength and energy release. Stress is frequently important to the design of wells for the purpose of petroleum production.

To be able to design in the materials of the ground it is important to know what the stresses and material properties are. The material properties prior to loss of strength are the Young's moduli, Poisson's ratios and Biot's coefficients. The values of these tend to vary with stress. In sedimentary rock the Young's modulus tends to increase with the average stress, sometimes several fold, while Poisson's ratio tends to increase with shear stress and decrease with mean stress. Biot's coefficient tends to decrease with increasing average stress. When some stress state, usually defined by the Mohr-Coulomb or Hoek and Brown failure criteria, is reached, the rock's shear strength will diminish, sometimes dramatically, leading to rapid deformation.

A useful rule of thumb for determining the importance of stress in contributing to the failure on intact rock in mine roadways is to see if the in-situ stress is more than a quarter of the uniaxial compressive strength. If it does so, then failure is likely and support methods will need to be considered more carefully.

What Leads to Stress in the Ground

Most sedimentary deposition is in a marine or lacustrine environment. The particles settle through water and build up in thickness. The types of material in the sequence change with time so that different layers are built up. The deposited material is a soil with a very high void ratio (pore space). As the soil is compressed by its own mass, water is squeezed out, the pore space is reduced and it becomes more dense. This is the process of consolidation. In fine grained material this may be a very slow process. In coarser material the water is squeezed out much more quickly.

Earthquakes may cause coarser material to consolidate as they shake the particles down into denser packing. In very fine material the effect is less as the water cannot escape. Liquefaction sometimes occurs in finer soils subject to earthquakes. This is a process where the packing of the soil is disturbed and the soil would pack down into a finer form. It cannot however do this immediately, because the stress that was carried between the grains cannot escape immediately and the water carries the stress in the form of raised pressures. Until the grains repack this soil is very weak and unstable.

In packing down in various forms the soil may generate a range of lateral stress. If the soil lacks a lateral boundary or there is extension of its base the limit of lateral stress is defined by its active state. If the lateral boundary compresses, the stress is defined by the passive state. For a soil without cohesion, but with an internal angle of friction $\emptyset$, the range of state of horizontal stress may be approximated by Equation 4.

$$\sigma'_v\left(\frac{1-sin\emptyset}{1+sin\emptyset}\right) < \sigma'_h < \sigma'_v\left(\frac{1+sin\emptyset}{1-sin\emptyset}\right) \quad (4)$$

Where σ'_v is the vertical effective stress due to self weight and fluid pressure effects.
σ'_h is the horizontal effective stress
$\emptyset$ is the internal angle of friction of the soil

If $\emptyset$ = 30° then the limits of Equation 4 are given in Equation 5.

$$\frac{1}{3}\sigma'_v < \sigma'_h < 3\sigma'_v \quad (5)$$

This is a very large range which may depart significantly from the plastic state where $\sigma'_v = \sigma'_h$.

The value of vertical effective stress may vary due to deposition, erosion or changes in ground fluid pressures. The concept of overconsolidation is one where a soil has been buried to a significant depth and has developed stresses associated with that burial, and then erosion has removed material from surface, lowering the vertical stress. This raises the ratio of horizontal to vertical stress until failure occurs at something approximating to the passive stress state. This occurs near the erosional surface first.

Figure 2. Tangent moduli derived from testing two similar sandstone cores, one triaxially and one uniaxially. Tri-E1 is the Young's modulus transverse to the bedding direction, Tri-E2 is the Young's modulus in the direction of bedding. Uni-E1 is the modulus transverse to the bedding direction derived from a uniaxial test.

Lithification of sedimentary deposits will take place with time. In this crystals are built up which change and interconnect the grains. How much of the soil stress is carried over into the rock that is created is undetermined. Lithification leads to the development of rock that has more of the properties of an elastic solid, albeit not necessarily one with linear elastic behaviour. Indeed the moduli of sedimentary rock may be very nonlinear as is shown in Figure 2.

Diagenesis may take place altering the minerals and bonding within the rock mass, and changing stresses further.

Strain in the ground has a major effect on changing the state of stress. This strain may come from major tectonic plate movement, folding or faulting. Tectonic plate movement tends to set a regional principal stress direction trend. This is overlain by folding at different scales, imposing tensional and compressional components to the stress within the rock mass.

Faults are invariably stress relief features, and the force that is relieved ($stress \times area$) has to be carried somewhere within the rock mass. This shift of load leads to some other zone being stressed. Take for example the simple example of a reverse fault that is localised within one layer of strata and has limited lateral extent. Following the fault displacement, the force taken across the fault has diminished and is moved to the surrounding layers and to the end of the fault, thus raising the level of stress in these.

If continuing lateral strain is being applied to a rock mass, it is possible for it to reach a stress at which failure on a fault occurs. After this the stress regime is changed, and it is possible for the stress principal directions to be rotated, sometimes at 90^o. With continuing strain the stress builds up in the fault again, and failure ensues in a stick-slip cyclic process. The Japanese earthquake of 2011, which affected the Fukushima area, was a classic example of this behaviour. Similar events can however be seen in the coal mining areas to the west of Sydney and Wollongong (Gray, Wood and Shelukina, 2013).

Igneous rocks are either intrusive or extrusive. Extrusive material can only carry stress at the time of its placement caused by gravitational loading. Dykes and sills are good paleo-stress markers as they extrude in a direction perpendicular to that of the minimum stress. Large intrusions will carry the stresses required for their placement. These will be reflected in the stress in the surrounding rocks and in the visco-plastic behaviour of the rock mass in the molten state, which then becomes more elastic as the material cools. As all hot rocks cool they will shrink. This cooling strain generally de-stresses the rock mass. Where the boundary cooling is quicker than that of the main body of rock the rock outer boundary solidifies first and shrinkage of the mass continues. This induces a compressive stress in the boundary and is considered to be the cause of exfoliation fracturing of exposed batholiths.

Other forms of shrinkage may also exist that affect stress. One of these is the shrinkage of organic matter in the process of coalification. This shrinkage is thought to lead to the presence of cleating, a clear indicator that zero or at least very low lateral stress existed in the coal at some time.

The Concept of Tectonic Strain

The types of rock in the ground are variable, with very different stiffnesses. Under the influence of strain the stresses developed in each different stiffness of rock vary. To understand the stress distribution better the concept of Tectonic Strain (Gray, 2000) is useful. Tectonic Strain is the theoretical strain required to cause the rock mass to be at its current state of stress.

In the context of fairly horizontal stratigraphic units, the tectonic strains may be thought of as the horizontal strains that are required to change the horizontal stresses in the ground from those which would exist due to gravity alone in a zero lateral strain environment. They need not be due to gross tectonic movement. Rather they may be due to local faulting or folding. Indeed a component of the tectonic stresses may come from soil like behaviour of sediments with normal or over-consolidation of these prior to lithification. Some dimensional change can be expected in such lithification and any further diagenesis of the rock, and appear as a component of the tectonic strains. The effects of temperature on inducing strains may also be bundled into the tectonic strains.

Despite these limitations fairly even tectonic strains are found to exist in several measurements in a rock mass, and it is possible to use these to interpolate stresses between measurements.

The mathematics of determining Tectonic Strain are as below.

The average total vertical stress is, over a wide area, a summation of the product of density of all the superincumbent stratigraphic units with each stratigraphic unit's thickness multiplied by gravity, as shown in equation 6.

$$\sigma_v = g \sum_z^0 \rho_i \Delta x_i \tag{6}$$

Where σ_v is the total vertical stress
g is the gravitational acceleration
ρ_i is the density of the *i*th stratigraphic unit
Δx_i is the thickness of the *i*th stratigraphic unit in the vertical direction
z is the depth from surface

The effective vertical stress is given in equation 7.

$$\sigma'_v = \sigma_v - \alpha_v p \tag{7}$$

Where σ'_v is the vertical effective stress
α_v is Biot's coefficient influencing the vertical stress
p is the fluid pressure

The total horizontal stress due to self weight in a laterally confined situation with zero lateral strain is given in equation 8 and the effective horizontal stress due to self weight in a similar case is given in equation 9.

$$\sigma_{hsw} = \sigma'_v \left(\frac{v}{1-v}\right) + \alpha_h p \tag{8}$$

$$\sigma'_{hsw} = \sigma'_v \left(\frac{v}{1-v}\right) \tag{9}$$

Where σ_{hsw} is the total horizontal stress due to self weight
σ'_{hsw} is the effective horizontal stress due to self weight
v is Poisson's Ratio for strain in the horizontal plane brought about by stress in the vertical direction
α_h is Biot's coefficient influencing the horizontal stress
p is the fluid pressure

If we now use a simplified elastic model which does not account for creep behaviour, then we can subtract the effective horizontal stress due to self weight from the horizontal principal effective stresses to arrive at what we will term here to be tectonics stresses. These are shown in equations 10 and 11.

$$\sigma'_{t1} = \sigma'_1 - \sigma'_{hsw} \tag{10}$$
$$\sigma'_{t2} = \sigma'_2 - \sigma'_{hsw} \tag{11}$$

Where σ'_{t1} is the major tectonic horizontal stress
σ'_{t2} is the minor tectonic horizontal stress

Assuming a ground surface that is free to move vertically the tectonic strain may be calculated using equations 12 and 13.

$$\varepsilon_{t1} = \frac{\sigma'_{t1} - v\sigma'_{t2}}{E} \tag{12}$$
$$\varepsilon_{t2} = \frac{\sigma'_{t2} - v\sigma'_{t1}}{E} \tag{13}$$

Where ε_{t1} is the major tectonic strain
ε_{t2} is the minor tectonic strain

To examine the average tectonic strain for a group of stress measurements, the procedure is to rotate the principal strains into direct N-S & E-W strain and shear strain components and to find the mean of these. The principal tectonic strains and their directions may be calculated from these three mean strains. If tectonic strains are relatively uniform between adjacent stress measurements they may be used to calculate stresses in rock of varying Young's Moduli and Poisson's Ratios. The process is the reverse of that used to derive the tectonic strain.

The effective stresses due to tectonic strain may be calculated using equations 14 and 15.

$$\sigma'_{t1} = \frac{E}{1-v^2}(\varepsilon_{t1} + v\varepsilon_{t2}) \tag{14}$$

$$\sigma'_{t2} = \frac{E}{1-v^2}(\varepsilon_{t2} + v\varepsilon_{t1}) \tag{15}$$

The total effective horizontal stress may be calculated by adding the horizontal stress component due to gravity acting in a zero lateral strain environment as given in Equation 9 to the values arrived at in equations 14 and 15.

This may seem like an unnecessarily complex process but it gives a consistent basis by which to assess the state of stress in strata where stress measurements have not been undertaken.

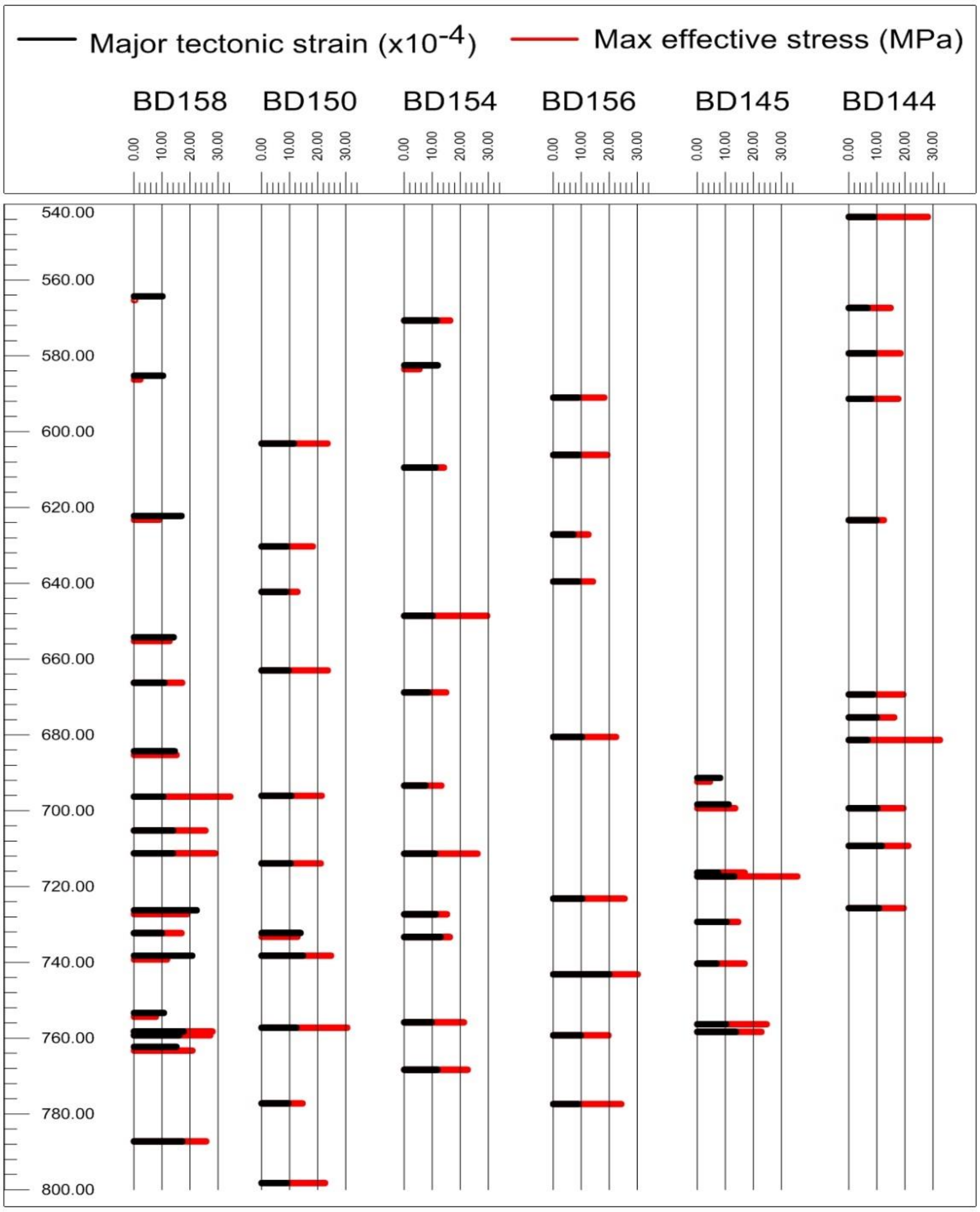

Figure 3. Showing the major effective stress and the major tectonic strain.

Figure 3 shows an example from a site in a sedimentary basin in eastern Australia. In it the major effective stresses from 68 stress measurements are shown in red. They are quite variable. The calculated tectonic strains are, however, quite even in each borehole and, with the exception of the first hole, they are quite even across the site.

Not all sites are going to provide as even a set of Tectonic Strains as are presented in Figure 3. Where there is faulting the calculated values of Tectonic Strain tend to vary, as stresses are redistributed around faults. Similarly quite different tectonic strains may be found above and below erosional surfaces. Tectonic Strain theory does however give a good basis for getting away from plotting horizontal stresses versus depth and quoting meaningless ratios between these and the vertical stress.

How Do You Measure Stress?

It is impossible to measure stress in the ground without disturbing it. The disturbance required is considerable and will affect the value being measured. Thus stress measurement is anything but exact. The next problem with measuring stress is that it tends to be so variable that a number of measurements are needed so that some sensible interpretation may be placed upon them to permit a sensible interpolation.

Soil Stress

The measurement of stress in-situ in soils is difficult. Anything put in the ground affects the stress and causes inelastic failure of some sort around the hole or plate penetration. Correlations exist with a variety of pressuremeter (devices that expand in a hole or slot) tests but essentially all are dubious. This is unlike the case for soil fill, where stress measurement sensors can be buried in the ground and can be expected to measure quite accurately.

Stress in Rock

The measurement of stress in rock can be accomplished with a variety of ways, with differing degrees of certainty. The principal methods used for rock stress measurement are:

- Overcoring
- Hydrofracture
- Borehole Breakout

Overcoring

Overcoring is the process where the stress change is determined by measuring a dimensional change within rock, when it is stress relieved, by coring over the top of it. Several variants of overcoring exist. To determine a stress field solution by overcoring requires that the rock properties remain elastic, and preferably linearly elastic.

The simplest overcoring method, which is often forgotten these days, is surface overcoring. This method provides a measurement of stress at the rock surface, often where it is most important – in a tunnel or at the bottom of an excavation.

In the past this used to involve the measurement of the change in diameter of points around a borehole after a central stress relieving hole was drilled (Tsur-Lavie and Van Ham, 1974). The diameter change was measured using a mechanical dial gauge with pivoted legs. These days it is best undertaken by smoothing the rock surface with a diamond grinding wheel on a disc grinder and gluing a strain gauge rosette to the rock along with a temperature sensor inserted into the rock. After the temperature has stabilised, the strain gauge and temperature sensor wires are folded over and attached to the rock, and a concrete sampling drill is used to core over these. At the end of coring the drill is removed, usually leaving the core in situ, and the strain gauges and temperature sensor are reconnected. A record of temperature and strain is then obtained, so that the strain may be related to temperature change and hence corrected back to the precoring temperature. The strain difference before and after overcoring is thus measured, and the rock can be removed to have its modulus measured. The strain, Young's modulus and Poisson's ratio are used to calculate stress. The author has used this technique at the bottom of deep foundations in Sydney and in a TBM tunnel in the Snowy Mountains. In the latter case four surface overcores were undertaken and a total stress tensor derived from their values.

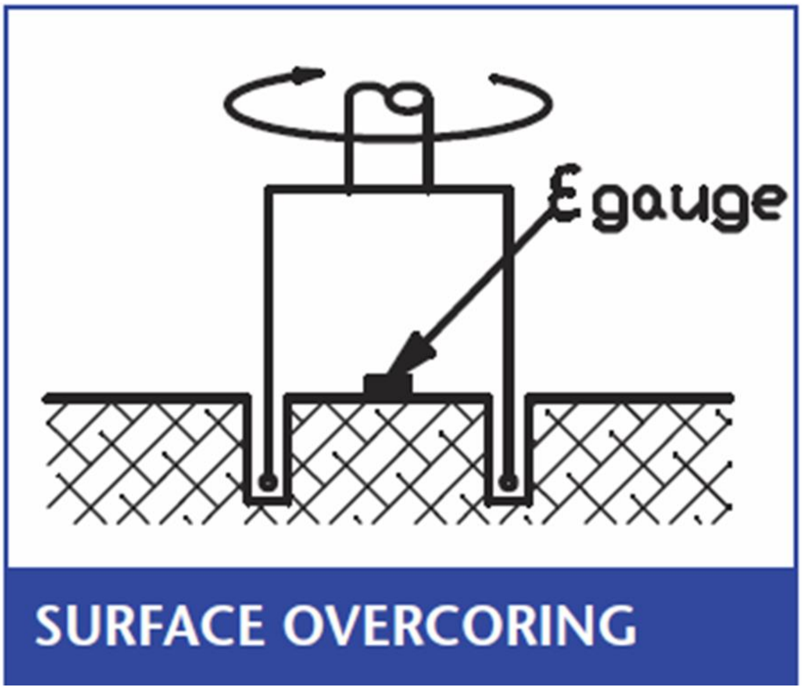

Figure 4. The concept of surface overcoring.

Overcoring is, however, most commonly undertaken at the end of a borehole. Most of the devices used for this are glue-in strain gauge devices, the adhesion of which is either impossible or compromised in wet holes. For this reason, they are most commonly used in dry upward holes that readily drain away drilling fluid. The glue in devices include:

- Doorstoppers
- Leeman Triaxial Cell
- CSIRO HI Cell
- ANZSI cell
- Borre Probe
- Cone cell

While the two main mechanical devices are:

- USBM deformation gauge
- Sigra IST tool

Doorstoppers

The doorstopper type cells (Leeman, 1971) used a strain gauge rosette glued to the end of a flat ended borehole and then overcored. These only provide measurement of the stress perpendicular to the hole. They were followed by the work of Saito in Japan (Gray, 1980), who drilled a flat-ended hole with a radius at the edge and whose doorstopper was fitted with additional strain gauges. Saito presented a solution for the full stress tensor for this device. The recent development in this area has been that of cone overcore devices that are glued into the end of a conical hole and permit the full stress tensor to be determined (Obara and Ishiguro, 2004). All of the doorstopper variants have the advantage that they can be overcored to achieve stress relief within a small drill advance distance. This means that they have a greater probability of being used successfully in jointed rock.

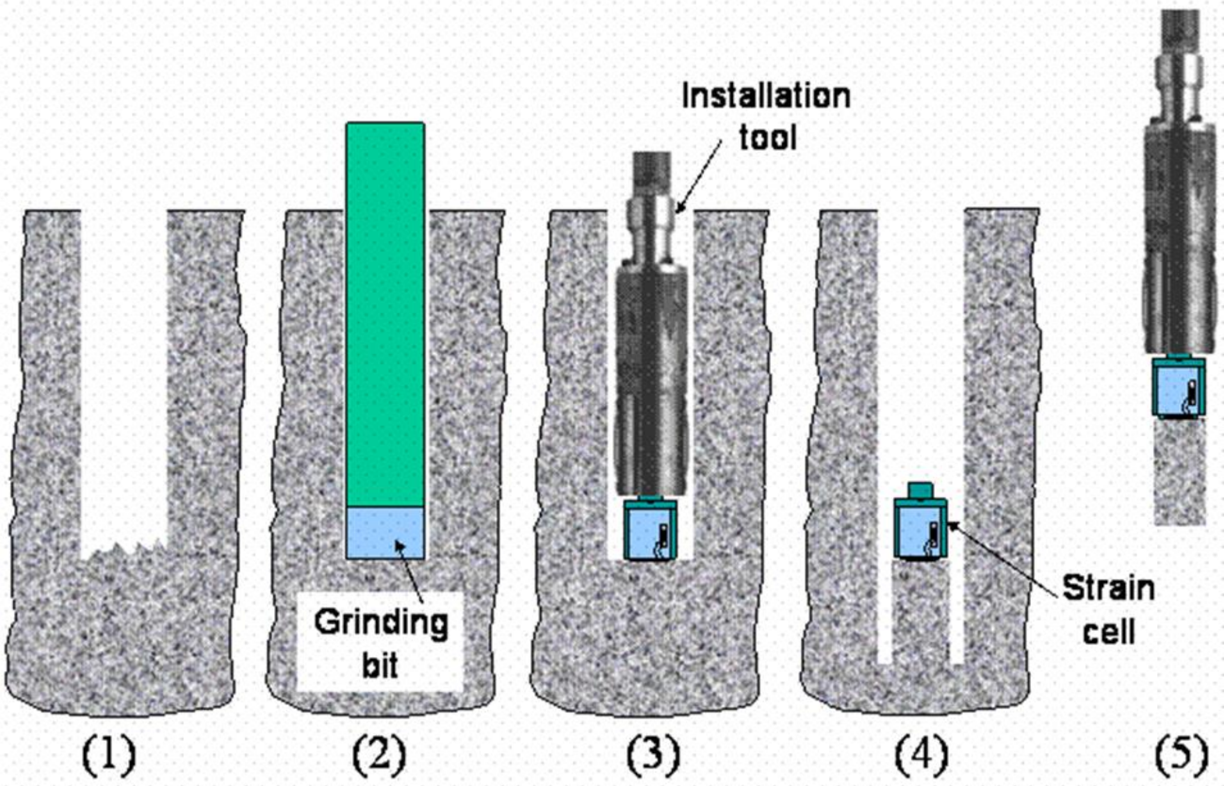

Figure 4. Overcore stress measurement process using a doorstopper.

Glue in Pilot Hole Overcore Devices

The other type of glued-in cells were fitted into a pilot hole and drilled ahead of the main borehole. The Council of Scientific and Industrial Research (CSIR) cell (Leeman, 1968) was the best known early version of these. It had a great advantage in that the strain gauge rosettes used contained four gauges at45° to each other. This could be used as a simple check that the gauge was actually adhered to the borehole wall, as the sum of strain changes of each of the two orthogonal gauges should add up to the same value. The disadvantage of this device is that it was not designed to provide progressive strain measurements during overcoring. Rather, a measurement could be made before and after overcoring alone.

The CSIRO Hollow Inclusion (HI) cell followed. This did not place the strain gauge on the borehole wall but, rather, epoxy resin is exuded between the cell, which has integral strain gauges, and the pilot hole. When this has set, the cell can be overcored. The system is used with a cable that carries strain readings from the 12 strain gauges to the borehole collar, so that the overcore may be progressively monitored. The need for a cable is a serious disadvantage. So too is the problem of delamination of the epoxy from the pilot hole where its expansion becomes too great due to de-stressing.

The CSIRO device was followed by the Auckland New Zealand Soft Inclusion (ANZSI) cell (Mills and Pender, 1986) which used the inflation of a low pressure rubber tube (packer) to push strain gauges against the pilot hole wall. Since then a number of other variants of these devices have been built by various groups.

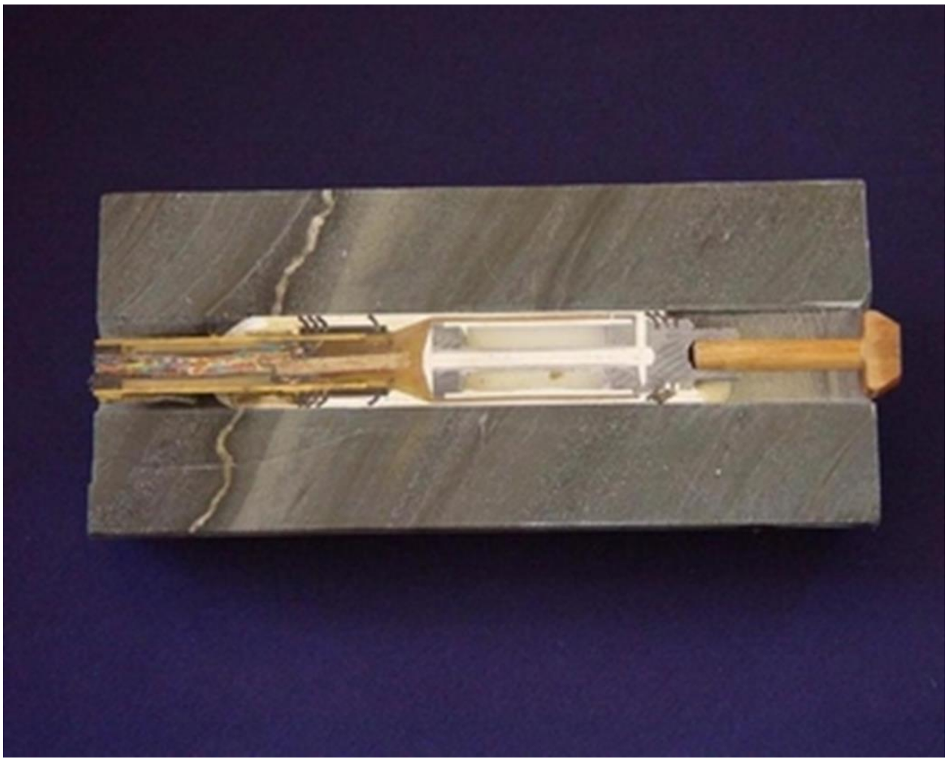

Figure 5. A cross section of an overcore containing a CSIRO HI cell.

Mechanical Pilot Hole Deformation Overcore Devices

In addition to the glue-in devices, there are the overcore devices that monitor the pilot hole diameter mechanically. The first of these is the USBM borehole deformation gauge (Obert, Merrill and Moran, 1962; Merrill, 1967). It is a biaxial device that measures the change in borehole diameter in three places. It is only suitable for use in dry holes or holes with very limited water pressure. This piece of instrumentation is still made and is very much in use, particularly in North America. It was followed by an attempt to build a device at The University of Queensland that used three triangular pin-beam type strain gauges (Leahy, 1984) pressed into the wall of the pilot hole. This suffered from problems of the pins slipping on hard rock surfaces, but was nearly successful.

The Sigra in situ stress tool or IST (Gray, 2000) is the modern development of the USBM tool. It has been in use since 1996 and has accomplished measurements at depths from 1.5 to 1000 m. It is used integrally with HQ or PQ wire line coring. It has six pin pairs to measure diameter. This is three more than the number required for a solution and therefore gives a measure of redundancy.

It is lowered into a pilot hole on wireline where it mechanically locks into place. Throughout the test it electronically records the diameters across the pin sets, the temperature and the output of the three magnetometers and three accelerometers. These latter measurements permit the tool's orientation to be determined. The limitation of this tool and that of the USBM device is that it is biaxial. It is necessary to therefore assume the axial (vertical) stress. The deformations that are measured and the stresses that are calculated are perpendicular to the axis of the tool. In its most common use, the tool is used in vertical boreholes where the vertical stress is presumed to be lithostatic. This is not an unreasonable assumption in most cases, especially where the rock is flat-lying sediments or is at a shallow depth.
In summary Sigra's IST system is:

- A quick biaxial overcore system
  - 100 m hole overcore in 1 hour
  - 500 m hole overcore in 2.5 hours
  - 1000 m hole overcore in 4 hours
- Used primarily with HQ wireline coring system though it may also be used with PQ
- Mostly used in vertical holes

The stress measurement procedure is shown in steps in Figures 6 and 7.

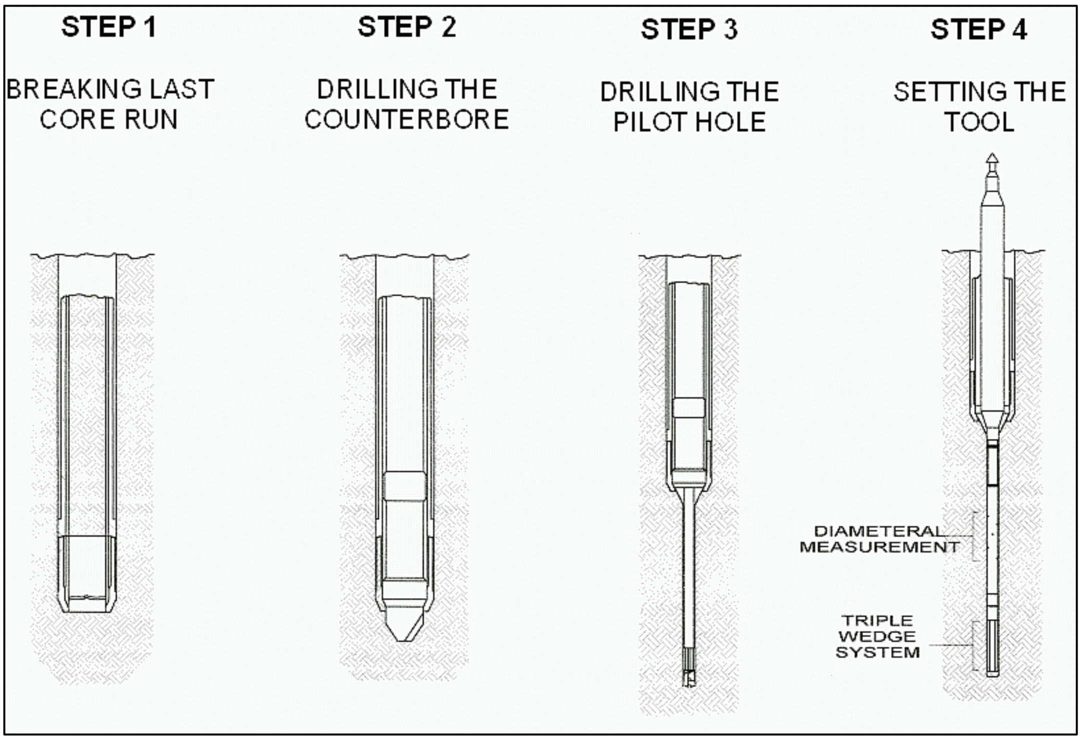

Figure 6. Initial steps in the Sigra IST stress measurement procedure

Figures 8 and 9 show photos from IST operations while Figure 10 shows the pin displacements during the overcore and Figure 11 the best fitted result to experimental data.

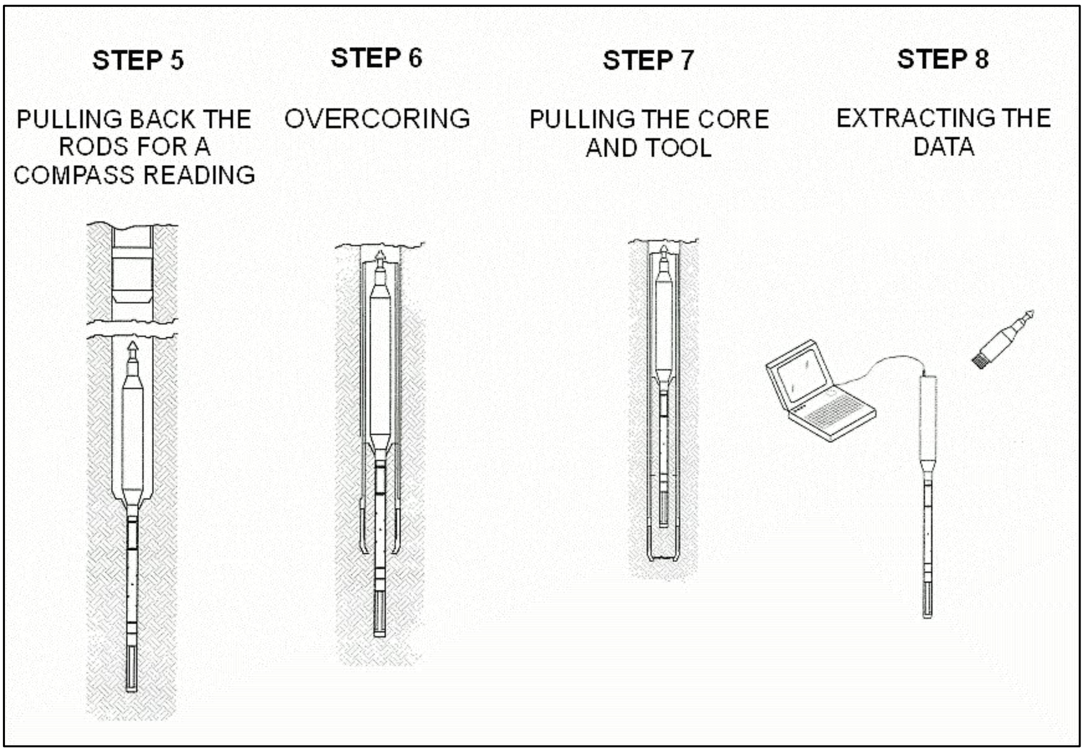

Figure 7. Further steps in the Sigra IST stress measurement procedure.

Figure 8. An IST tool being lowered into an HQ drill pipe prior to overcoring.

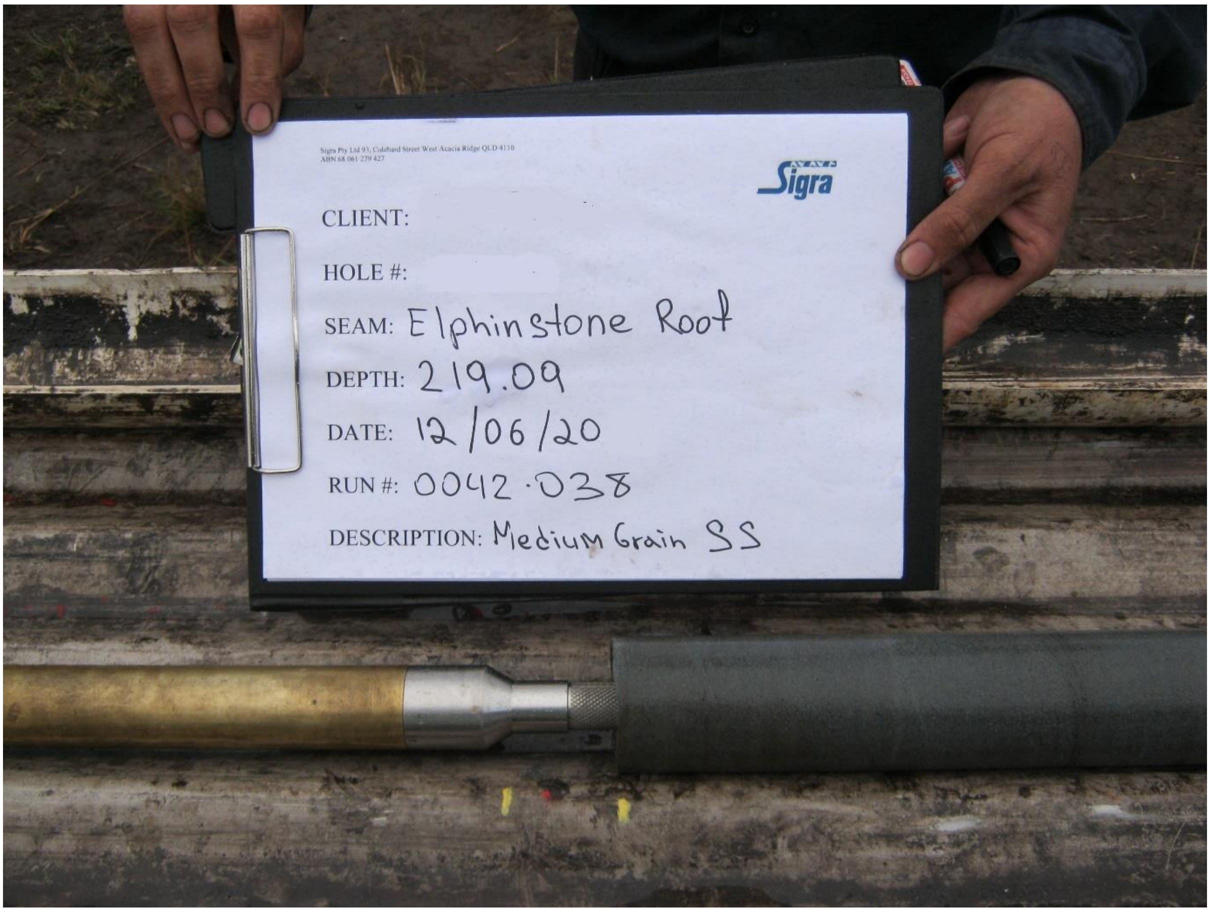

Figure 9. An IST tool set in a core of medium grained sandstone after overcoring.

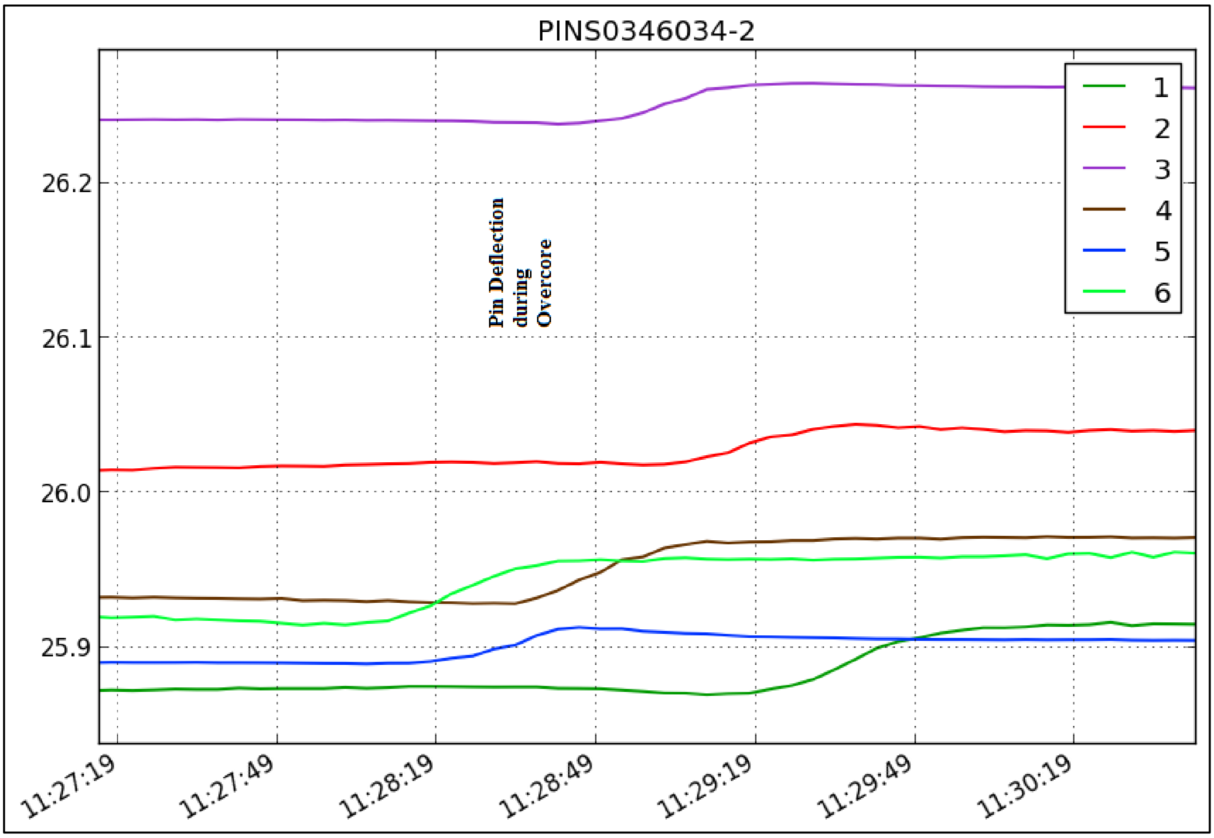

Figure 10. Pin displacements during an overcore

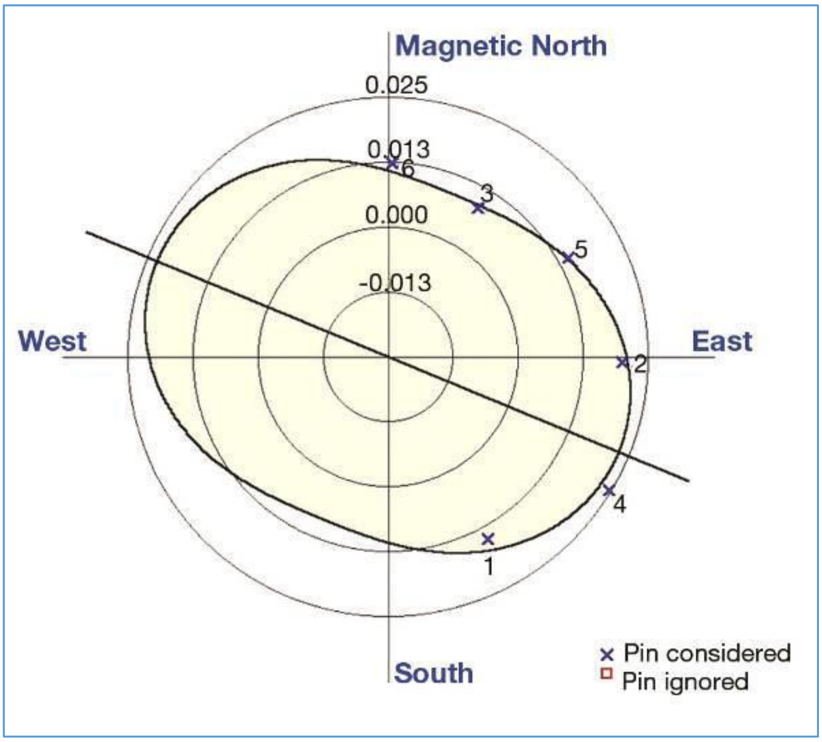

Figure 11. Plot of overcore displacements and the theoretical best fit to experimental data from an overcore using the Sigra IST overcore tool.

Overcore Accuracy

Converting an overcore deformation or strain measurement to a stress requires the mathematics relating the two, and detailed knowledge of the stress-strain behaviour of the rock. Overcore analysis is essentially based upon the isotropic linear elastic behaviour of the material surrounding a pilot hole in rock. The problem with this is that a lot of rock is not linearly elastic or isotropic. This is illustrated by the relation of total stress to Young's moduli shown for a sandstone in Figure 2. The best that can be obtained in laboratory core testing are the orthotropic properties of the rock, though the two Poisson's ratios related to stress and strain perpendicular to the core axes cannot be precisely determined. In the event that the rock is porous Biot's coefficients are also important and need to be measured. Prior to the paper by Gray (2017) Biot's coefficient had not been incorporated into overcore theory, though its role is of some importance. This is shown in Equation 16

$$\Delta D_i = \frac{D}{E}\left[2\sigma_m + 4\sigma_D(1-v^2)\cos 2\theta_i - v\sigma_z - \left((1-2v)+(1+v)\alpha_r\right)p)\right] \qquad (16)$$

where:

D is the pilot hole diameter.

ΔD_i is the change in pilot hole diameter at angle θ_i from the principal stress direction.

E is Young's modulus perpendicular to the hole.

p is the fluid pressure at the overcore location.

σ_m is the mean total stress acting perpendicularly to the axis of the hole.

σ_D is the deviatoric stress (major stress – minor stress)/2 acting perpendicularly to the hole.

σ_z is the total stress acting on a plane perpendicular to the axis of the hole.

v is Poisson's ratio.

α_r is Biot's coefficient in the direction radial and perpendicular to the hole.

Equation (16) may be re-arranged to provide a solution for stresses based on the pilot hole deformation in overcoring.

Equation (16) may be broken into two parts. In addition to the effect of Young's modulus, the mean diameter change is affected by the mean horizontal stress perpendicular to the hole axis (σ_m), the stress along the hole axis (σ_z), normally taken as overburden stress, Poisson's ratio (v), Biot's coefficient (α_r), and fluid pressure (p). Thus the potential for errors in the mean stress grows with depth associated with the axial stress and fluid pressure. The deviatoric component of stress (σ_D) may however be more accurately determined from the out of roundness changes to the hole diameter measured during overcoring and influenced only by the Young's modulus and one minus the square of Poisson's ratio, $(1 - v^2)$. The latter remains reasonably close to unity even if the rock is totally plastic ($v = 0.5$).

The accuracy of overcore devices is limited by the ability to measure borehole diameter change or strain, and the amount of deformation that occurs during the stress relief brought about by the overcore process. The Sigra IST tool can measure to a sensitivity of 0.1 micron in the laboratory, or practically to one micron in the field during a good drilling process across a 26 mm diameter pilot hole. In a 10 GPa stiffness rock this corresponds to a stress measurement sensitivity of 0.2 MPa. If the rock is however, 50 GPa, this sensitivity reduces to 1.0 MPa.

Overcore tools that use glued-on strain gauges may have a nominal accuracy of 0.2 microstrain in the laboratory; however, in the field two microstrain is more realistic. This corresponds to a stress sensitivity of 0.04 MPa in 10 GPa rock. Once again temperature and drilling fluid pressures limit this uncertainty to a significantly greater value – maybe 0.4 MPa.

Hydrofracture

Hydrofracture is the prime means used by the petroleum industry to measure the minor principal stress at great depth. It is also used by others at shallower depths. It involves sealing section of borehole and raising the fluid pressure until failure occurs. The pumping is then discontinued. Fracturing fluid then leaks off and the fractures close. A schematic of the concept of hydrofracturing is shown in Figure 12. Figure 13 shows an idealised single fracturing sequence with subsequent fracture closure.

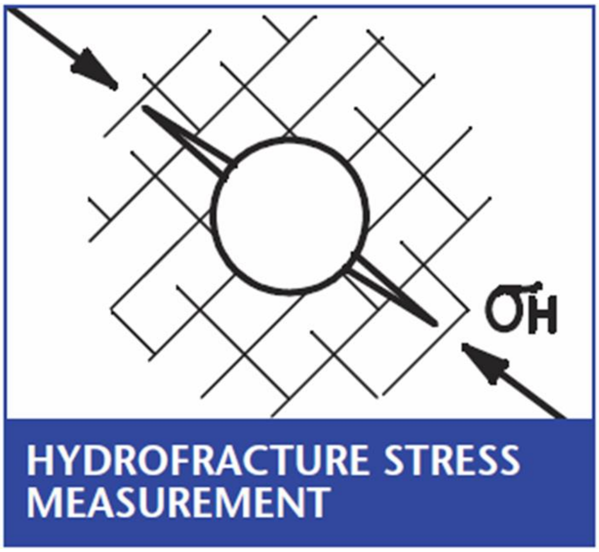

Figure 12. Schematic of hydrofracture development around a borehole in a field with major principal stress σ_H.

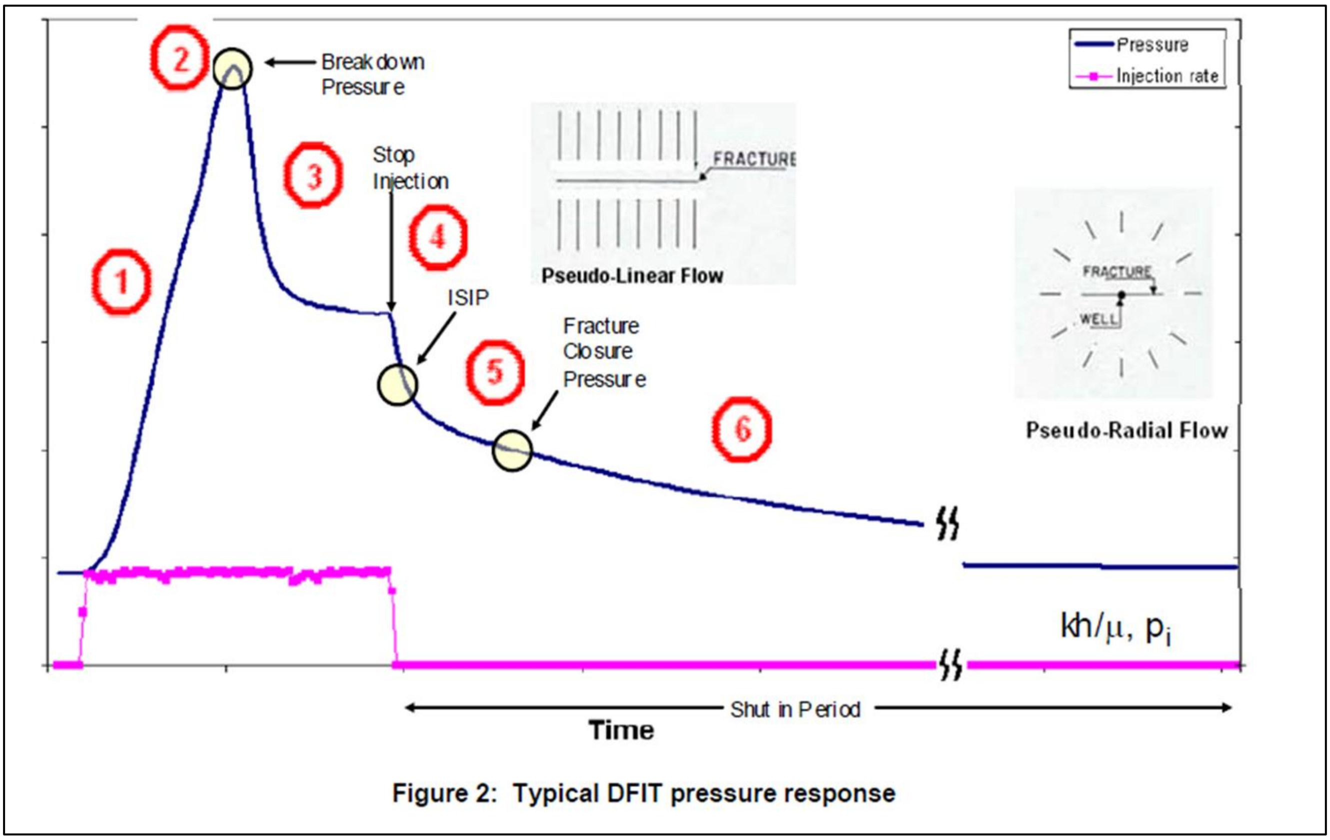

Figure 13. Flow and pressure trace versus time for a single hydrofracture with fracture closure.

The fracture closure pressure is a measurement of the minimum principal stress. In recent years intense activity has been focused by the petroleum industry on the use of hydrofracture as a means of production from low permeability reservoirs. The determination of the closure pressure for minimum stress has been of the utmost importance and such methods as G Function analysis have been developed (Martin et Al, 2012).

While obtaining the minimum closure pressure and thus the minor stress may be possible from hydrofracture, obtaining the major stress is much more difficult. Firstly, the assumption must be made that the borehole is drilled in one of the axes of principal stress. Further, that pre-existing fractures do not exist. Then, making the assumption that the rock is linearly elastic and has a certain tensile strength, it is possible to theoretically calculate the fracture opening pressure in terms of the closure pressure and the major stress. This approach is unrealistic because the tensile stress of the rock is unknown. The approach is then generally taken that the fracture can be re-opened following initial closure by a second pressurisation cycle, and that the fractured rock will behave exactly as though it were a rock without any tensile strength.

This approach is unrealistic as the first fracture will generally not close perfectly and pressurised fluid from the borehole will progress rapidly along it on re-pressurisation. This will lower the subsequent re-opening pressure. In addition, the poroelastic effects of fluid in the rock need to be considered as well as non-linearities of the elastic behaviour. The process to obtain a value of the major principal stress perpendicular to the borehole becomes very complex and interpretation is uncertain.

Hydrofracture as it is usually practised in geomechanics has another problem, as packers are used to seal a section of borehole. The packer scaling pressure against the borehole wall must exceed the fluid pressure to avoid leakage. The consequence of this is that it is very easy for fracture initiation to be caused by the packers themselves. This might just be avoided if the packer scaling pressure is dynamically maintained just above the fracture fluid pressure. In most applications this is not the case.

Hydrofracture can however be usefully used to open joints and find the normal stress across these. Knowing this normal stress may be all that is required. If multiple joints of different orientations exist in a rock mass it is possible to hydrofracture each of these and get a closure stress, and then use these multiple values to calculate a stress tensor.

Borehole Breakout

Borehole breakout also provides a method of assessing biaxial stress distribution around a hole. The method of measurement here is the dimension of the failure of the borehole wall. Breakout is shown schematically in Figure 14. This breakout is usually measured by an acoustic scanner as shown by the trace in Figure 15. If the wall stresses are insufficient to induce compressive or tensile failure of the borehole wall, then no indication of the stress field may be made.

Generally the measurement of breakout or tensile fracture only permits the direction of major stress perpendicular to a borehole to be estimated. The petroleum industry uses borehole breakout width in combination with hydrofracture closure pressure and uniaxial compressive strength estimates to arrive at two dimensional stress measurements. However core taken from the hole and tested along its axis will generally not provide an accurate value of the transverse uniaxial compressive strength in anisotropic rock. Neither do relationships between the sonic log and uniaxial compressive strength provide an adequate value of UCS for the determination of stress from breakout.

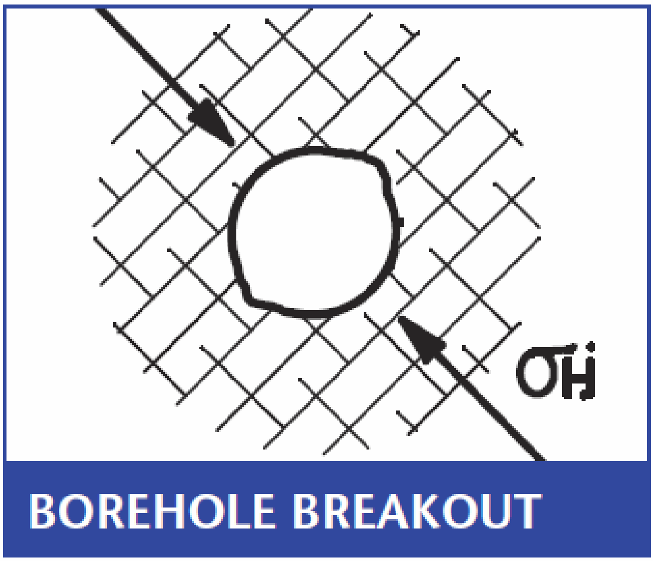

Figure 14. Stylised borehole breakout.

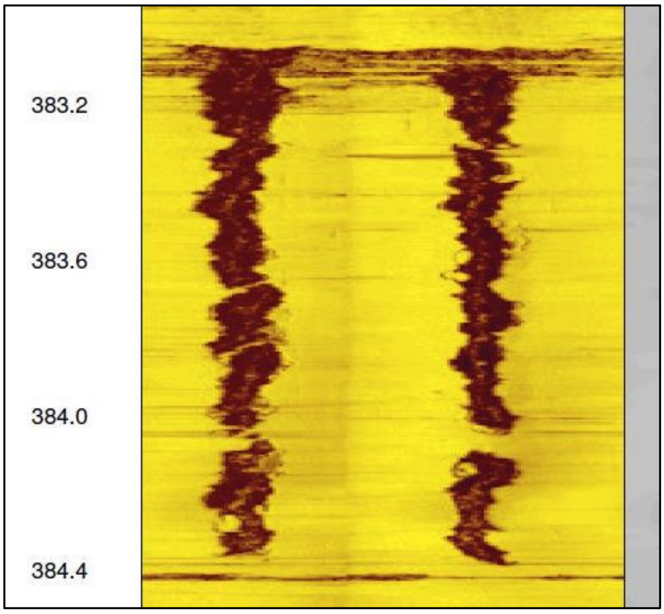

Figure 15. Borehole breakout shown in an acoustic scan. Depth on y axis. Wrap around view of hole wall from 0 to 360 degrees shown on x axis.

Other Rock Stress Measurement Methods

Attempts at measuring stress by the Kaiser effect whereby the core emits intergranular noise, when it is stressed beyond its original stress state, have been proven conclusively to not work by Hseih, Dight and Dyskin (2015). There has been a recent resurgence in the use of post elastic strain recovery of core as a method to estimate original rock stress (Wang et al, 2012). The method did not, however, generally work in the past and must at best be regarded as being of dubious accuracy today. This especially applies in lightly stressed near surface tunnelling applications. Generally, the more direct the measurement, the more accurate it should be, and recourse to alternatives should be avoided.

Conclusions

The determination of the stress regime in rock is a complex process. Not only is it necessary to use the right stress measurement technique for the rock type, but then there is a need to interpolate and possibly extrapolate the stresses away from the points of measurement. This requires a good knowledge of geology. Any model developed should take into account the lithology and structural geology. A real programme to measure stress may include multiple stress measurements by a number of techniques, then interpreted in terms of Tectonic Strain. This might be followed by an examination of borehole breakout to determine stress direction where other measurements have not been made. Faulting, as revealed by seismic reflection, may then be the key to understanding the regional stresses. What is certain is that the concept of some unique far field stress loved by numerical modellers is not a reality. Neither does stress necessarily increase monotonically with depth.

While hydrofracture may appear the most direct means to measure stress it is really only suitable for the determination of the minor stress if that is in a plane that runs through the borehole. The determination of the major stress is fraught with complexity. Practically, hydrofracture has significant problems associated with packer pressure initiating the fracture and with pressurised fractures that do not quite close. Overcoring has more uncertainty in the determination of the value of mean stress, but more certainty in the determination of the difference between major and minor stresses. The values of stress derived from overcoring are dependent on the material properties – Young's moduli and Poisson's ratios. These need to be measured properly, and nonlinearities and anisotropy taken into account properly in the analysis for stress. This is difficult and generally not attempted.

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