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Stress estimation in rock: a brief history and review

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Abstract

For this Special Issue of the Journal on Rock Stress Estimation, an introduction to the subject of in situ rock stress is presented here, concentrating on the history of the subject and the principles of both stress and its measurement. Following an historical overview of the need for rock stress information and the nature of rock engineering problems, stress and stress estimation are discussed. This is followed by a review of the stress measurement techniques. The importance of stress estimation and associated numerical modelling is highlighted. The conclusions emphasize the importance of a well-conditioned stress estimation campaign, by using more than one technique and supporting the campaign with a numerical model. The appendix focuses on the work at the Canadian Underground Research Laboratory.

1. Introduction

The distribution of forces in rock masses is a central concern of rock mechanics, both with respect to understanding basic geological processes such as plate tectonics and earthquakes, and the design of engineered structures in and on rock masses. These forces are derived from two main sources—gravity, and tectonic forces resulting from the motion of crustal plates.

Where the rock formations are relatively uniform, as in extensive flat lying sedimentary layers, the vertical force at a depth h will develop a uniform compression σ_z. proportional to the weight of overlying rock, i.e.

$$\sigma_z = \rho g h, \tag{1}$$

where ρ is the average mass density of the overlying rock and g is the acceleration due to gravity. Typically, $\rho g = 0.025-0.027\,\text{MPa/m}$ depth for most rocks. Where the rock structure is more complex, as shown in Fig. 1, the weight of the rock will not be uniformly distributed in the mass.

Overburden loads will be concentrated in the stiffer formations and shielded from the more deformable ones. Faults and other heterogeneities will also affect load distribution. In shallow workings, topographical variations too, will influence load distribution. In the case shown in Fig. 1, rock bursts were encountered at several sections of the tunnel being excavated through the rock (see legend in Fig. 1). Measurements taken in the tunnel in those sections indicated loads up to eight times the average overburden load, i.e. as given by Eq. (1) above. Since the total weight of the overburden above a given depth is constant, above-average loads at one or more locations indicates, of course, that the vertical loads in other parts of the rock will be below the average value.

Distribution of tectonic forces is also complicated by geological factors, with the added uncertainty in that there is no constraint on the total force, as is the case with gravity loads. Plate motions, interactions at plate boundaries and within plates are all driven by tectonic forces. The magnitude and orientation of the forces have changed over geological time; folds and faults created in response to forces from past epochs (as in Fig. 1), volcanic intrusions, etc. may all have been involved in creating the current heterogeneous system that is now subject to the current tectonic regime. It is to be expected, therefore, that the magnitude and orientation of these forces may vary considerably within geological systems such as illustrated in Fig. 1. There are, of course, many situations where the geological structure is

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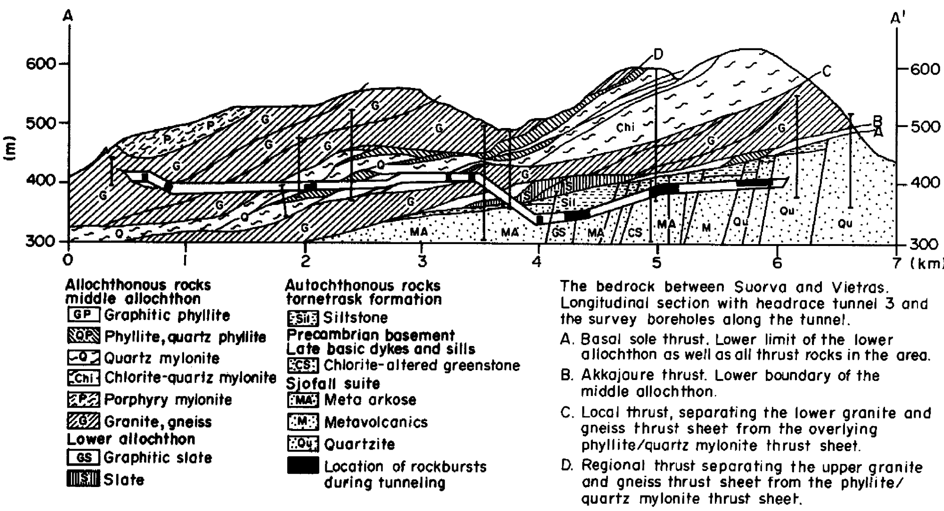


Fig. 1. Illustration of a highly non-uniform stress distribution encountered during the driving of a water supply tunnel in Northern Sweden (after Martna).

simpler, as in extensive sedimentary basins. Even here, the forces may vary in magnitude and orientation, especially in the vicinity of faults and fault systems.

Thus, unlike other materials used in engineering design, rock is pre-loaded, by forces that are, in general, of unknown magnitude and orientation. The problem of design in rock is complicated further by the fact that structural features such as joints, fractures, and bedding planes can have an important influence on the ability of the rock mass to resist these forces, i.e. on the strength of the rock mass, as measured over the region, often large, that is affected by the structure. This could have an adverse influence on the stability of the engineering structure.

Civil structures and mines have been built on and in rock masses for millennia. Rock blocks, quarried and selected to be free of obvious joints or other flaws, were the key element in construction of all kinds from the Pyramids, Roman bridges and aqueducts, to the remarkable Gothic cathedrals of the Middle Ages. These impressive structures evolved empirically and were designed conservatively, with care taken to ensure that all loads were in compression, taking advantage of the high compressive strength of rocks. Deformability and strength of the rock were not significant issues for these structures.

Serious interest in the deformability and strength of rock in situ, the loads imposed on the rock and the changes in load distribution produced by engineering activities, was stimulated primarily by several parallel developments in rock engineering, viz:

- the progressive extension of mines to greater depths, especially in stratified sedimentary formations such as coal, iron ore and bedded salt in Europe, and gold in India and South Africa, where violent rock bursts and loss of life were becoming more frequent;
- deeper exploration and production operations in petroleum engineering; and the search for improved oil and gas recovery;
- large public works projects, such as tunnels through difficult terrain (see Fig. 1), underground power chambers, rock slopes and high dams.

Although some difficulties started to appear earlier,[1] it was the world-wide economic boom associated with the post-World War II era, and some dramatic collapses such as the Malpasset dam foundation in France in 1959, the Coalbrook mine collapse in South Africa in 1960; the dam slope failure at Vaiont in Italy in 1963, and others, that provided a major stimulus to development of a better fundamental understanding of rock deformation and failure. The older term *strata control* was retained for the special problems of mining in stratified deposits, but *rock mechanics* emerged as the overarching discipline.

In trying to develop a theoretical framework for rock mechanics, it was logical to consider whether developments in other branches of engineering could assist in addressing the issues. Given the impressive success of *continuum mechanics* in these fields, could the same analytical techniques solve problems in rock mechanics?

Continuum mechanics, *the study of matter as a continuous medium without regard to its composition as an aggregate of discrete particles*,[2] and with it the Theory of Elasticity, originated during the latter part of the17th

[1] Severe surface damage and flooding due to mine subsidence, for example, had led to the establishment of a Strata Control Commission in the late 1930s in Northern England.

[2] Encyclopedia Britannica. Micropaedia, Vol. III, 1975, p. 114.

century as an outgrowth of the development of infinitesimal calculus. There was considerable debate, initially, on the validity of the continuum hypothesis for materials with discrete 'particulate' structure. Newton's molecular theory of structure was widely recognized, but it was not possible to establish the large-scale response of a material by integration of the force-deformation relation between individual pairs of 'particles'.[3] The molecular theory lost favour as the continuum approach was demonstrated to be capable of providing answers to important technical problems, stimulated by developments of the Industrial Revolution.

By the time that rock mechanics problems were being encountered and studied, continuum mechanics was well established. Several solutions in the theory of elasticity (e.g. Lamé in 1852 [1], Kirsch in 1895 [2] were of particular interest). Indeed, continuum mechanics provided the only well developed formal body of technology upon which one could draw, as distinct from empirical rules established from practical observation. Some applications appeared to have greater validity than others. Examination of force redistribution produced in a stratified rock mass by excavation of an ore-bearing planar seam, for example, appeared to be a more tractable problem of continuum mechanics than attempting to assess the force distribution in the rock mass of Fig. 1. In the latter case, however, the rock mass may be sufficiently homogeneous over various sections along its length to permit a useful analysis of the stability of the tunnel and to establish support requirements. For essentially horizontal excavations in relatively simple geological conditions, such as in longwall coal mines and the Witwatersrand gold reef in South Africa, it could be assumed as a first approximation that the deformation behaviour in the vicinity of the excavations was controlled by the overburden load, and that this was constant and equal to $\sigma_z = \rho g h$, as defined in Eq. (1) above.

A perusal of the rock mechanics literature of the 1950s, when International Strata Control Conferences and Rock Mechanics Symposia were being established, show that continuum mechanics was providing the analytical foundation for much of rock mechanics. In most cases, however, elastic analysis was given primary attention. From the work of Cook [3], however, more attention has been given to the question of stability of structures in rock.

For applications in other branches of engineering, the designer could use the theory of elasticity to establish the critically loaded parts of a structure and, from this analysis, limit the loads on the structure such that the strength of the material was never reached. The problems posed in rock mechanics are usually of a somewhat different nature. Thus, the strength of a rock mass and the forces acting in it are not well defined. Assessment of the stability of a situation is usually the relevant concern. Fig. 2 shows a 'complete' load–deformation curve for rock loaded in compression to disintegration, noting that similar curves are found for loading in tension and loading in shear (e.g. on joints).

Segments 1and 2 of the curve, to the left of the peak load represent the elastic region followed by moderate levels of inelastic deformation (distributed microcrack growth). To the right of the peak, Segment 3, the rock starts to disintegrate, deformation becomes increasingly localized along discrete paths, and is discontinuous. Application of continuum mechanics and the notion of 'stress' to the rock disintegrating in this region becomes dubious and, eventually, untenable.

For in situ situations, the rock undergoing disintegration as in Fig. 2 will be only part of the rock mass under load. The rock outside the disintegrating region will remain elastic and behave as a continuum. The interaction between the disintegrating region and the elastically unloading region determines whether or not the collapse will occur violently ('seismically') or gently ('aseismically'). In the violent situation, the elastic energy released by the 'internal region' at any given deformation (Segment 5) exceeds the energy that can be absorbed quasi-statically by the collapsing region. The excess energy is used to accelerate collapse, which becomes 'violent'. If the reverse case applies (Segment 4), then a stable failure process takes place. Additional loading, e.g. by progressively enlarging the size of an excavation, will probably produce a gradual deformation following the path of the declining rock resistance.

It is seen that continuum mechanics and notions such as stress and strain are valuable in rock mechanics, being applicable to the entire rock body during the rising portion of the loading curve, and to the region outside the disintegrating volume of rock on the falling portion.

In some civil engineering situations, loads on the rock can be maintained well below the compressive strength, i.e. at the 'maximum design load', or lower. This is the situation for the rock block masonry structures referred to earlier. For many mining activities, the right hand region in Fig. 2 is an essential component of extraction methods. Some methods rely heavily on promoting controlled disintegration and collapse. Tunnel excavations and boreholes may deform gradually or violently (as in the case discussed in Fig. 1). Thus, continuum concepts and measures such as stress can play an important role in understanding rock mass deformation, albeit in a somewhat different manner than in other fields.

[3] The continuing development of increasingly powerful computers and numerical modelling techniques is starting to overcome the difficulties in integrating the particle–particle force interactions.

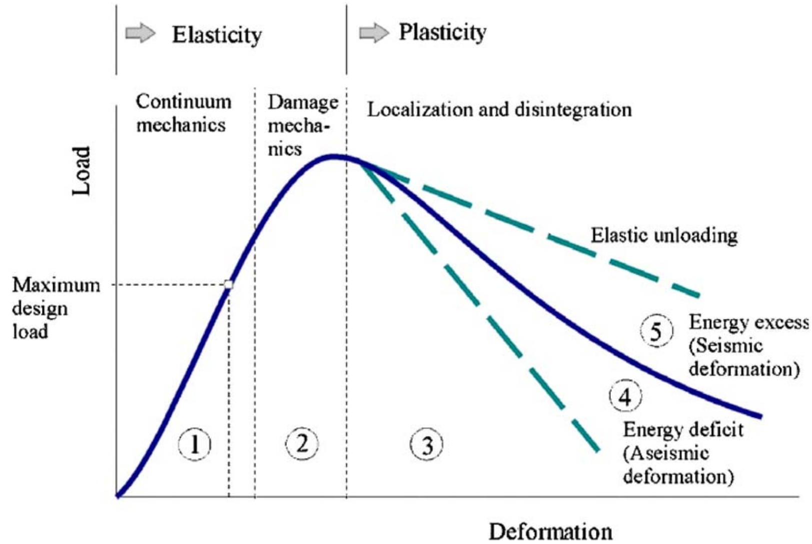


Fig. 2. Typical 'complete' load–deformation curve for rock loaded to destruction.

2. Stress

Stress is a central concept in continuum mechanics. Although now well accepted in rock mechanics, it is perhaps worth recalling the fundamental definition. We take an elementary area ΔA in the interior of the material, across which an elementary internal force ΔP is acting, at some arbitrary orientation to the area. The ratio $\Delta P/\Delta A$ is the 'average stress' across the area ΔA. The total stress σ at the point M within ΔA, as the area shrinks around ΔM is then defined as

$$\sigma = \mathrm{Lim}_{\Delta A \to 0}\, \Delta P/\Delta A.$$

The total stress σ is a vector applied at the point M and can be represented in terms of nine stress components, acting at M which define the components of the stress tensor at the point M, see e.g. [4, Chapter 2].

While rock can be sufficiently 'structured' to call into question the validity of a continuum approximation, the size of the overall region to be analysed can be large enough to accept such an analysis as a rough indication of how loads may be distributed in the structure. Accepting that stress analysis may be applicable, we now face the special problem of determining the forces (or stresses) already acting within the structure, usually referred to as in situ stresses, as well as any stress changes induced by engineering activities.

A variety of theoretical hypotheses have been proposed to suggest what the in situ state of stress should be, for different very simplified models of the rock mass. Thus, if we assume that the rock at depth h in a rock mass of infinite lateral extent, behaves as an isotropic elastic material from the moment it is created, then the in situ state of stress can be defined to be

$$\sigma_x = \sigma_y = \frac{v}{1-v}\sigma_z, \tag{2}$$

where σ_x and σ_y are principal stresses along the horizontal axis x and y. σ_z is the vertical principal stress, assumed to be $\sigma_z = \rho g h$ where ρ is the rock density, g is the gravitational acceleration, h is the depth of overlying rock and v is Poisson's ratio for the rock.

Since Poisson's ratio for most rocks, as measured in the laboratory, is of the order 0.25–0.33, then

$$\sigma_x = \sigma_y = (0.3 - 0.5)\sigma_z. \tag{3}$$

If we now assume that the long-term (geological time) shear strength of the rock tends to zero, then we will arrive at the uniform stress condition

$$\sigma_x = \sigma_y = \sigma_{\mathrm{v}}. \tag{4}$$

A similar state of stress is arrived at if it is assumed that the rock is in an unconsolidated fluid state when first deposited (e.g. at depth underwater).

When we recognize that tectonic forces can act essentially independently of the gravitational forces, then the horizontal to vertical stress ratio is indeterminate, but often

$$\sigma_x \text{ or } \sigma_y > \sigma_z \text{ (compression assumed positive).} \tag{5}$$

This brief exercise illustrates that an understanding of the state of stress within a rock mass will require direct observations/measurements within the rock. This is a significant difficulty not faced in most other applications involving structures composed of fabricated materials, for which the elastic (and plastic) properties, and the loads acting on the structure are usually well defined.

3. Stress estimation

Unlike other measurements, such as convergence between the roof and floor of an excavation, or pressure in a hydraulic support, stress measurement faces a basic obstacle, in that stress is a concept defined within the framework of continuum mechanics. It is not a physical phenomenon that can be measured directly. It is possible to estimate or determine the mean stress over a finite region of rock, provided a relation can be established between the mean stress, σ (Eq. (6)) and a measurable effect, ε, that this stress would produce over that region.

$$\sigma = \Omega\varepsilon, \tag{6}$$

where Ω is an operator that 'translates' the measured quantity into the desired stress.

The simplest example of an operator is the elastic modulus E of an isotropic, elastic specimen loaded in uniaxial compression, generating the measured uniaxial strain ε. Applying the operator as indicated allows the uniaxial stress σ to be determined.

All stress determination techniques require a relation of the form of Eq. (6) whereby some physical property can be measured and, using a known operator, converted to a value of stress. The validity of the stress determination depends entirely on the degree to which the assumed operator represents the relation between the (inferred) stress and the measured effect. We will discuss this further in connection with various stress estimation techniques.

Stress estimation is often one part of a more comprehensive series of measurements intended to understand the overall behaviour of a structure. In some cases, it may not be necessary to determine stress explicitly, even though it may influence the behaviour of interest. In the case of a tunnel excavation, for example, the in situ stress affects the amount of support needed to stabilize the opening. Here, however, the stability of the excavation can be assessed directly by observing how the rate of diametral convergence of the tunnel changes with time. The tunnel wall will deform inwards, driven by the in situ stress σ_0. As the wall deforms, the support resistance σ_i increases, reducing the net differential stress $(\sigma_0 - \sigma_i)$, which is proportional to the net force producing radial convergence of the tunnel wall. Assuming that the support remains effective, the rate of diametral deformation will eventually decline to zero, as the resisting radial force exerted by the support and the force required to stabilize the rock reach equilibrium. It is advisable to assess carefully, in any measurement campaign, which parameters need to be measured/observed to achieve the overall goal. This is particularly so in the case of stress, which is often more difficult to determine than other, more direct measurements.

4. Techniques of stress measurement

Leeman [5] has published a comprehensive review, in four parts, of the state of development of rock stress determination, in 1964. The review confirms the comment earlier in this paper, of the almost simultaneous effort, world wide, to develop a better understanding of rock mechanics, both basic and applied. Of 44 papers and reports cited in the review, the two earliest were published in 1954 [Potts, United Kingdom; Ignatieff et al. Canada] and the most recent (several) in 1960. Although some instruments have changed in detail, the principles used in current techniques are all represented in Leeman's review. The principal exception is hydraulic fracturing, which was first suggested as a possible rock stress measurement technique 1 year after the review was published, [6].

One of the earliest reported methods [7,8] was based on the use of a flat-jack (also referred to as a Freyssinet jack). The technique is illustrated in Fig. 3. An extensometer gauge is installed between the points A and B in the rock surface (Fig. 3). This can be of various forms, but a (piano) wire tensioned between the two points was often used. The frequency of vibration of the wire is determined. A slot is then cut into the rock as shown. The slot should be wide enough to completely relieve the stresses acting across the points A and B. This is accomplished by making the slot width equal to three times the distance from the slot to point B. The flat-jack is then inserted into the slot and cemented into place to ensure good contact with the faces of the slot. The jack is then pressurized until the distance AB is restored to the value measured before cutting the slot, as indicated by the frequency of the wire transducer. It is then assumed that the pressure in the jack is equal to the average normal stress that was acting across the slot before the slot was cut.

Note that the measurement assumes that the normal stress/pin deformation relation on unloading as the slot is cut is the same as the flat jack pressure/pin deformation relationship on pressurization, i.e. that the rock is elastic over the range of pin deformation. However, an important limitation of this technique is that it needs to be conducted on the surface of an excavation, in the region of maximum (and varying across the depth of the slot) stress concentration around the excavation. This is the region where the rock is most likely to be overstressed and develop some inelastic deformation. It seems likely, therefore, that there will be some hysteresis between loading and unloading paths, so that the pressure required to return the pins to their original spacing will differ from the stress released by cutting of the slot. Thus, the operator Ω in Eq. (6) is not known, and the flat jack pressure may not represent the in situ stress. This inherent shortcoming of stress determinations made on the surface of an excavation is a main reason why most techniques involved measurements at depth within a borehole. One advantage of this

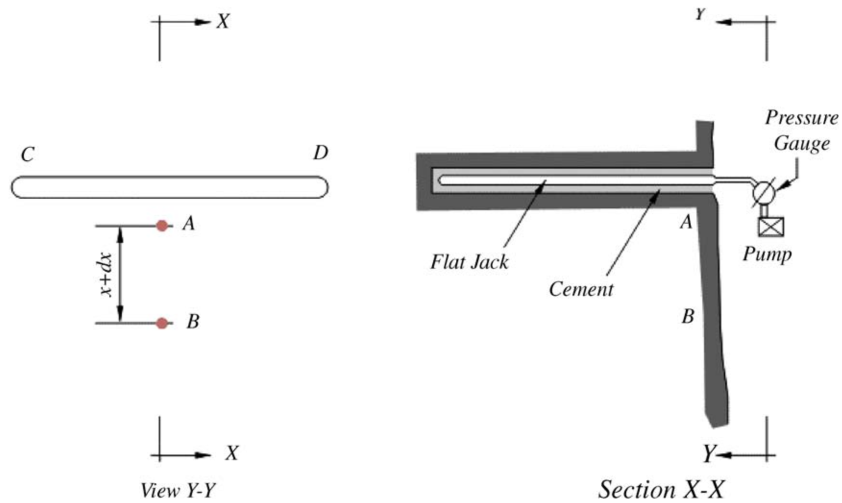


Fig. 3. The flat-jack technique of stress measurement in rock.

flat-jack method was that it allowed the use of a simple extensometer (i.e. placed between points A and B in Fig. 3), and did not require the development of special tools and transducers that could be placed within a small borehole. This difficulty is discussed by Hast in 1943 [9],[4] one of the first major contributors to measurement of in situ stresses in rock.[5]

Jaeger and Cook [12] attempted to develop a modified version of the flat jack technique in which the test was carried out in a borehole drilled to a depth (maximum of about 7 m) beyond the zone of stress concentration. The method is illustrated in Fig. 3. The following description is taken from Leeman [5].

> A borehole is drilled with a coring tool to the depth at which it is desired to determine the rock stress and in a direction parallel to one of the principal stresses, say σ_3. Sufficient length of core is left on the end of the borehole for two curved flat jacks A and B to be fitted into the annular groove between the core and the walls of the borehole. The pressure in the jacks A and B is raised to a value such that the tensile stress in the rock in the quadrants C and D on the outside of the annulus exceeds the tensile strength of the rock and causes tensile cracks to develop. The direction of these cracks will indicate the direction of the major principal stress σ_1, acting in a plane normal to the axes of the borehole. They may be observed by overcoring the flat jack and will be visible on the cylinder of rock removed from the borehole. The length of the borehole is next extended by means of a coring tool and the curved jacks AB again inserted in the annular groove so that the direction of the resultant pressure acts normal to the direction of σ_1. The pressure of the jacks is raised to a value P say, and overcored. A relief of pressure is noted due to the overcoring. Two pairs of jacks, EF and GH are then inserted in the annular groove resulting from the overcoring operation as shown in Fig. 4. The pressure in these jacks is raised to say, P in EF and P in GH to restore the pressure P in jacks AB. This procedure is repeated with the jacks AB oriented at 90° to the direction used above and the pressure P_1' and P_2' in EF and GH determined to restore pressure P in AB. By substituting these pressures in formulae derived by the theory of elasticity,[6] σ_1 and σ_2 can be obtained. The curved jack technique makes it possible to determine the stresses for some distance inside a rock face and in this respect eliminates one of the main disadvantages of the flat jack.

There is no indication that this technique was used beyond some initial trials. It is likely that it was abandoned due to practical operational difficulties.

Several of the most popular procedures developed for stress determination are based on analytical solutions in the theory of elasticity.[7] Some of these are described in the following sections.

[4] The first application of bonded electrical resistance strain gauges appears to have been described in 1938, but did not become commercially available until some time later.

[5] The 1943 publication, Hast's Ph.D. thesis, focuses on the development of a high modulus inclusion cell and a magnetostrictive transducer for determination of stresses in masonry brick and concrete structures. Application of the high modulus tool to stress determination in rock is described in later paper [10,11]. Dr. Hast was perhaps the first to indicate, based on his extensive measurements, that not infrequently, lateral stresses were greater than the gravitational stresses. This is the situation represented by inequality (5) in this paper.

[6] The formulae are given in the original paper by Jaeger and Cook [12] and also in Sections 10.4–10.7 of their book *Fundamentals of Rock Mechanics* [4].

[7] These developments in practical techniques for stress determination were taking place when high-speed computers and numerical methods of analysis were not well developed. Heavy reliance was placed therefore on available 'classical' solutions in the theory of elasticity.

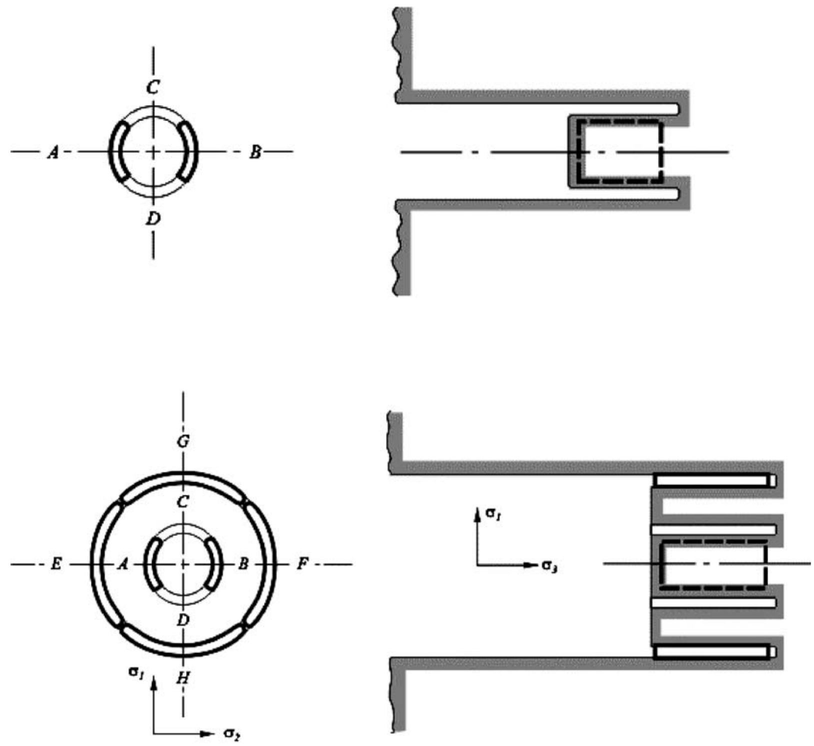


Fig. 4. Borehole jack method of stress determination (after [4]).

5. Stress meters

Theoretical solutions for the stress concentration in a circular elastic inclusion within an elastic medium of lower modulus had been published by several investigators, see [5].

Fig. 5 shows a circular elastic inclusion of modulus E' within an elastic material of modulus E. It is assumed that the circular interface between the two media is 'welded' so that no differential movement can occur across the interface. It is seen that the ratio between the vertical stress σ_1' in the inclusion to the vertical stress in the host material changes very little when the ratio of the modulus E' of the inclusion is five times or more greater than the host material. This solution spawned a considerable number of so-called 'stress metres'. The principal advantage of these instruments was that it was not necessary, provided the modulus of the inclusion was considerably higher than that of the rock, to know precisely the modulus of the rock. It was difficult practically to satisfy the 'welded' requirement, and instruments were usually wedged tightly (pre-stressed) into the hole. They were intended primarily to determine stress changes subsequent to installation, as distinct from total in situ stresses. The ability of a stress metre to detect stress reductions at a site subsequent to installation depended on the level of pre-stress that could be generated when installing the tool in the borehole.

Although a wide variety of 'stress metre' designs were popular in the 1960s, many appear to have lost favour. Pressure cells, in which a thin 'flat jack' is incorporated diametrally within an otherwise solid steel cylinder, now fulfil this role. The jacks can be inserted at a desired depth and orientation in a borehole and then pressurized to ensure good contact with the borehole wall. Subsequent changes are monitored on pressure gauges usually located close to the mouth of the borehole. Introduction of the relatively compliant fluid 'layer' into the cell lowers the overall modulus so that the cell needs to be calibrated for the rock type (i.e. modulus) if the pressures are to be used as a measure of the stress in the rock acting normal to the pressure cell. Frequently, pressure cells are used to monitor changes in the stress distribution in the rock rather than to establish absolute values.

6. Borehole deformation strain cell

The analytical expressions for the stresses and displacements produced around a circular hole in an elastic isotropic medium subjected to normal and shear stresses at infinity are shown in Fig. 6, reproduced from [13].[8] These equations for in plane (xy) values are based

[8] This report contains also the related solution for stresses and displacements around a circular hole in a transversely isotropic elastic medium, (derived by D.S. Berry). The report also contains a copy of the review by Leeman and a draft of a similar, though less extensive review, *Determination of the Stress in Rock—A State of the Art Report,* by Leonard Obert, Science Adviser, US Bureau of Mines. July 1966.

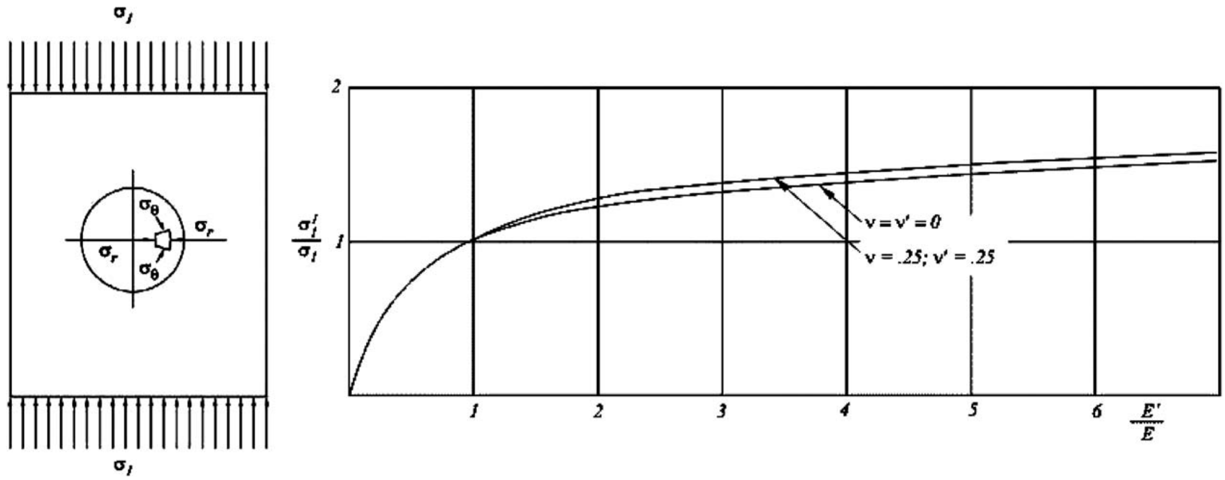
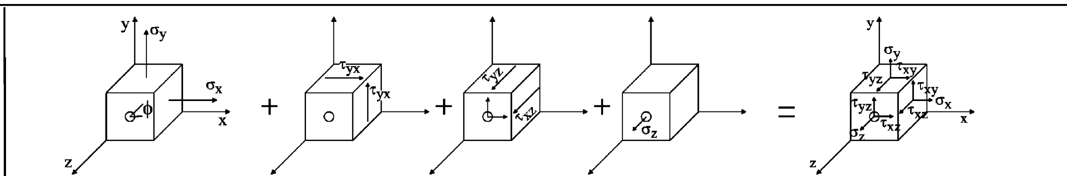
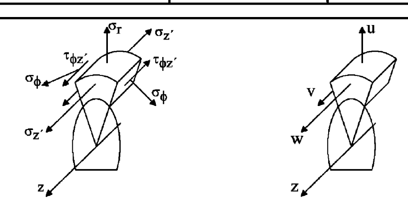


Fig. 5. Stress concentration within a high modulus 'welded' elastic inclusion.



Stresses applied at infinity / Stresses at wall of hole (r=a)					
σ_ϕ	$\sigma_x+\sigma_y-2(\sigma_x-\sigma_y)\cos 2\phi$	$-4\tau_{xy}\sin 2\phi$	0	0	$\sigma_\phi=s_1+s_2-2(s_1-s_2)\cos 2(\phi-\alpha)$*
$\sigma_{z'}$	$-2\nu(\sigma_x-\sigma_y)\cos 2\phi$	$-4\nu\,\tau_{xy}\sin 2\phi$	0	σ_z	$\sigma_{z'}=\sigma_z-2\nu(s_1-s_2)\cos 2(\phi-\alpha)$
$\tau_{\phi z'}$	0	0	$2(\tau_{yz}\cos\phi-\tau_{xz}\sin\phi)$	0	$\tau_{\phi z'}=2(\tau_{yz}\cos\phi-\tau_{xz}\sin\phi)$
Displacements at wall of hole u_r	$\frac{a}{E}\,[(\sigma_x+\sigma_y)+2(1-\nu^2)(\sigma_x-\sigma_y),\ \cos 2\phi]$	$\frac{4a}{E}\tau_{xy}\sin 2\phi$	0	$\frac{-a\nu\sigma_z}{E}$	$u_r=\frac{a}{E}\,[(s_1+s_2)+2(1-\nu^2)(s_1-s_2),\ \cos 2(\phi-\alpha)-\nu\sigma_z]$
Displacements at wall of hole v_ϕ	$\frac{-a(1-\nu^2)}{E}\,(\sigma_x-\sigma_y)\sin 2\phi$	$\frac{2a}{E}(1-\nu)\tau_{xy}\cos 2\phi$	0	0	$v_\phi=\frac{-a}{E}(1-\nu^2)(s_1-s_2)\sin 2(\phi-\alpha)$
Displacements at wall of hole w	$-\frac{\nu z}{E}\,(\sigma_x+\sigma_y)$	0	$\frac{4a}{E}(1+\nu)[\tau_{xz}\ \cos\phi+\tau_{yz}\sin\phi]$	$\frac{\sigma_{z'}z}{E}$	$w=\frac{1}{E}\{[\sigma_z-\nu(\sigma_x+\sigma_y)]z+4a(1+\nu),\ (t_{xz}\cos\phi+\tau_{yz}\sin\phi)\}$

Stress and Displacement Components at Wall of Hole (Positive as shown)



*Definitions of secondary principal stresses (s_1, s_2) as used above

$$s_1 s_2\}=\left(\frac{\sigma_x+\sigma_y}{2}\right)\pm\sqrt{(\sigma_x-\sigma_y)^2+4\tau_{xy}^{\ 2}}$$

$$\alpha=(1/2)\tan^{-1}[2\tau_{xy}/\sigma_x-\sigma_y]$$

*$u_r=U/2$ U=diametral displacement

Fig. 6. Stresses and displacements at the wall of a circular hole in an isotropic elastic medium.

on solutions developed by Kirsch in 1898 [2]. Out of plane effects are derived in [13]. The expression for the radial displacement, u_r, in this table forms the basis for borehole deformation cells that have been widely used for in situ stress determination. Measurement of the radial deformation across four different diameters (i.e. four different values of the angle ϕ) will allow the equation for u_r to be solved for the magnitude and orientation of the secondary principal stresses $s_1,$ s_2, and their orientation α in the xy plane, and the magnitude of the axial stress σ_z along the borehole. The shear stresses acting parallel and perpendicular to the borehole do not affect the radial deformation. As noted by Panek in1966 [14], these stresses have no influence on the diametral deformation of the drill hole "because a linear element experiences no change of length due to a shearing stress that acts parallel to it or at right angles to it. The effect of shearing stress is only to change the angle between two linear elements, one of which is parallel to, and the other perpendicular to, the direction of the shear."

The deformation cell contains some form of transducer designed to measure the change in radial displacement of a borehole when the hole is overcored by a larger concentric hole. Overcoring removes the pre-existing stress field from the annulus of rock. Change in displacement can then be related to the change in stress. As seen from the expression for the displacement u_r in Fig. 6, a value for the modulus of elasticity E is needed to solve for the stresses. The modulus can be determined in the field by placing the overcored length of core in a pressure cell that allows pressurized fluid to be applied to the outer periphery of the section of core containing the strain cell. The displacement of the hole as the fluid pressure is increased gives a measure of the modulus. As in the case of the flat jack pressurization, it is possible that the unloading and the reloading moduli may differ, but this can be checked by cycling the pressure applied to the core and observing the loading and unloading response.

Various types of cells have been developed. The Maihak cell rested on the bottom of the borehole and was maintained in position by springs. A pin was screwed out from the cell to contact (firmly) the wall of the borehole. A vibrating wire transducer determined the initial diameter at the pin location.[9] Overcoming produced a change in diameter at the pin. This produced a change in frequency of the vibration (proportional to the change in diameter) that was measured on an oscilloscope.

The US Bureau of Mines cell uses a beryllium copper cantilever to which is bonded resistance strain gauges to form the transducer. The cantilever spring loads a contact ball against the borehole wall.

As originally developed, in the late 1950s, both the Maihak and the US Bureau of Mines cell were designed to measure changes across one diameter only at each setting. Thus, in order to solve for the principal stresses, it was necessary to move the cell deeper into the hole in stages, reset at a different orientation and the overcore then advanced to relieve the stresses on the cell each time. The CSIR (South Africa) cell was designed with two deformation sensors oriented at right angles to each other. This has the useful attribute that the sum of the two measured radial deformations (or, equivalently, the sum of the two instrument readings) should be constant independent of the orientation of the tool in the hole. This provides a useful practical check that the equipment is operating correctly.

The US Bureau of Mines cell has evolved considerably. It was modified [15] to incorporate three diametral displacement sensors oriented at 60° to each other. This greatly improves the stress determination operation, which can be a tedious process. According to Thompson [16] "The USBM borehole deformation gauge is commonly regarded as the most accurate and reliable instrument available for determining in situ stress using overcoring stress determination."

7. Low modulus inclusions

As noted in the discussion of borehole deformation cells, instruments that measure radial displacement changes only are not affected by the shear stress components acting in the direction of the borehole axis. Thus, it is not possible with these instruments to determine the complete state of stress in the vicinity of the borehole for one hole orientation only. Several investigators have examined the possibility[10] of incorporating strain gauge rosettes into the interior of a low modulus plastic, which could then be glued into the borehole. Overcoring of the inclusion would generate strains in the plastic. Analysis of the strains registered by the gauges would allow determination of the complete stress tensor from one hole and one overcoring operation. To the writer's knowledge, no successful practical stress determination instrument of this specific type has been developed. The CSIR (South Africa) and CSIRO-HI cell, however, are designed with the same purpose, using gauge rosettes that are glued to the surface of the inner borehole before overcoring. Martino et al. [17] describe applications of both instruments. There is no classical analytical solution for the stress developed on the flattened surface due to in situ stresses, but several authors have derived approximate solutions from photoelastic analysis and numerical analysis.

8. CSIRO (Australia) hollow inclusion (HI) triaxial strain cell

This cell allows, in principle, the complete stress tensor to be determined from a single overcoring operation in one borehole. Strain gauge rosettes

[9] Vibrating wire instruments are popular since transmission of the signal frequency from the transducer is not affected by the resistance of the lead wires from the instrument to the measurement.

[10] The late Dr. Manuel Rocha described a low modulus plastic inclusion for rock stress determination, in the review *Rock Mechanics in Portugal* distributed at the First I.S.R.M. Congress, Lisbon, 1966. Strain gauges of various orientations were embedded in the plastic cylinder, which was glued into the borehole before overcoring.

attached to the outer surface of a thin molded epoxy cylinder are bonded to the wall of the borehole at different orientations. Overcoring of the inner borehole induces strains in the gauges that are influenced by all of the in situ stress components. Resolution of the measured strains should yield the in situ stress tensor at the overcoring location. The method is used widely and is considered to be a valuable technique. Problems of improper bonding of the gauges to the rock are reported. Depending on the orientation of the hole, some of the components of the stress tensor may be small, so that measured values may be suspect. It is useful, once the stress tensor has been determined, to repeat the test—if possible, using a hole drilled at an orientation for which the stress components are all of substantial magnitude. Martino et al. [17] have developed the CSIRO–HI measurement technique such that it is possible to observe the response of all strain gauges in the cell continuously both during the glue curing process and during overcoring. This allows failures of the bonding to be identified, so that results from the affected gauges can be eliminated from the stress determination.

9. Doorstopper gauges

The doorstopper gauge is a method of gluing a strain gauge rosette directly to the flattened bottom of a borehole. The hole must be carefully ground flat and then cleaned to ensure that the glue will bond the gauges to the rock. The technique is so named because the circular rubber assembly with the rosette on the base and wires from the base to a connector on the remote end has an appearance not unlike that of a standard doorstopper. Leeman indicates that a doorstopper technique was used as early as 1932 to determine stresses in a rock tunnel below the Hoover Dam in the United States, and also in Russia in 1935. The technique is still used and has been adapted recently for application in deep boreholes Martino et al. [17].

10. Hydraulic fracturing

The techniques described so far are all intended for measurements made from boreholes to a depth generally not exceeding some metres from a surface or underground access. Hydraulic fracturing is a technique developed originally over 50 years ago as a method of stimulating production in petroleum reservoirs. A sealed-off section of a borehole in an oil or gas-producing horizon is pressurized until a fracture develops in the borehole wall. The pressure is increased and a granular material (e.g. coarse sand) is added to the pressurizing fluid so that the material flows into the extending fracture. Fractures can be extended hundreds of metres from the borehole. When the pressure is dropped the fluid returns to the borehole but the fracture remains propped open by the granular material. The fracture forms a high permeability path to the well. Hydraulic fracturing can greatly increase the productivity of a well.

Analysis of the mechanics of hydraulic fracturing by Kehle [18] led the author [6] to suggest that the method be developed and adapted for use as a technique of stress determination. An important advantage of hydraulic fracturing is that it is routinely used at depths of several thousands of metres from a surface access. Rummel [19] has successfully determined in situ stresses by hydraulic fracturing at a depth of approximately 6 km, using aluminium packers. High temperatures precluded the use of rubber packers. The technique has also been used in deep boreholes from underground tunnels. The method is illustrated in Fig. 7.

A section of the borehole is sealed by inflatable packers, placed below and above the horizon that is to be fractured. The sealed off interval is then pressurized by an appropriate fluid, e.g. water, to a pressure P. Application of pressure to the borehole walls generates a tangential tension σ_θ in the wall of the borehole. When the tangential tension is high enough to overcome the tangential compression induced around the hole by the in situ stress state and, further, to reach the tensile strength of the rock, a fracture develops along the length of the packed-off interval. Although not shown in Fig. 7, there is a possibility that, if the stress acting along the axis of the borehole is low compared to the tangential stress required to cause the vertical fracture, then a horizontal fracture could develop and propagate from the well in preference to the vertical fracture Bedding planes that intersect the hole would further facilitate opening and propagation of the horizontal fracture. Once a fracture is initiated and fluid enters, it is assumed that it will propagate at a pressure somewhat above the normal compression acting across the fracture. Shutting off the pump and closing the pressure system should allow the fluid to stop flowing in the fracture, so that pressure losses due to flow are eliminated (assuming that leaf-off of fluid into the formation can be neglected). This static pressure is known as the Instantaneous Shut-In Pressure (ISIP).

In the case of a vertical hole, and a vertical fracture, the stress conditions governing initiation and propagation of the fracture are as follows. If we assume, first, that the hole is drilled vertically and that the vertical stress (σ_v) is a principal stress and that it is larger (compression assumed positive) than the maximum horizontal principal stress σ_{H}. The fluid pressure in the packed-off interval is raised to the value P_{b}, at which a vertical fracture is initiated and propagates in its own plane. The minimum horizontal stress is designated σ_{h}.

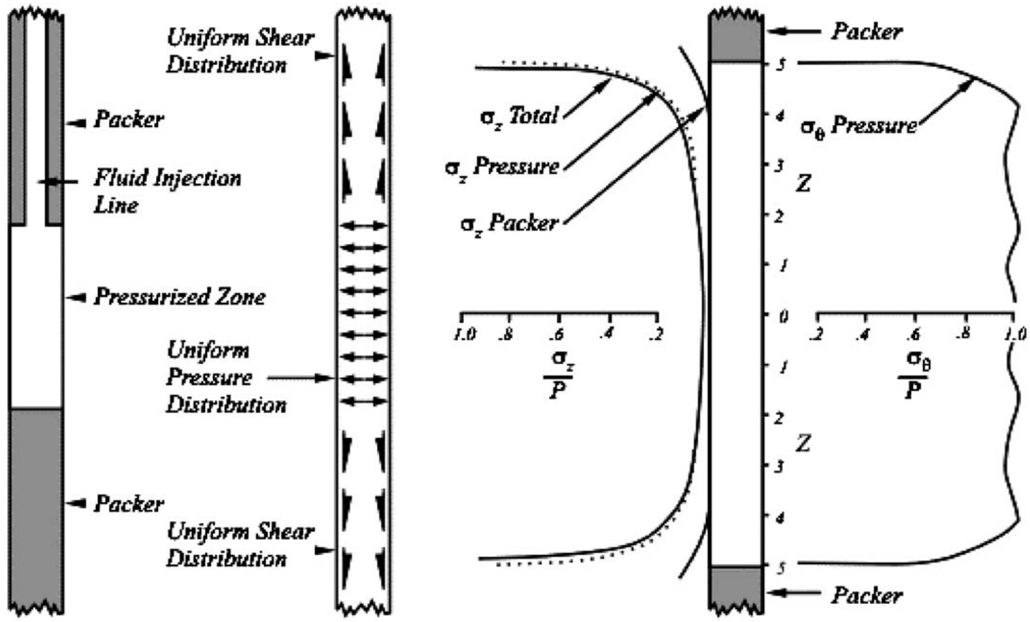


Fig. 7. Hydraulic fracturing technique (left) and the resulting stress distribution (right).

Under these conditions we determine, from the classical Kirsch 1898 equations [2] for stress concentrations around a circular elastic hole

$$\sigma_H + 3\sigma_h + T - P_b - P_0, \tag{7}$$

$$\sigma_h = \sigma_{isip}, \tag{8}$$

where T is the tensile strength of the rock; P_b is the fluid pressure at which fracture occurs; P_0 is the ambient pore pressure and P_{isip} is the ISIP.

It is assumed in these equations that the permeability of the rock is low enough that none of the pressurizing fluid enters the pores of the rock during the measurements. For more permeable rocks, it is necessary to consider the effect of flow into the rock. Haimson and Fairhurst [20] have developed modified expressions to take into account this effect for the assumed case of steady flow.

Thus, with P_b and P_{isip} measured, and P_0 known, Eqs. (7) and (8) can be solved to determine σ_H and h_e. The vertical principal stress, σ_v, is usually assumed to be equal to the overburden pressure, as given in Eq. (1), i.e.

$$\sigma_v = \sigma_z = \rho g h. \tag{9}$$

Observation of the orientation of the fracture generated at the hole, either by some form of borehole logging device that can identify fractures, or using an inflatable "impression packer" that is expanded under pressure against the open hole after the fracturing, and forms an impression of the fracture. The orientation of the fracture is also recorded. In this way, the "complete state of stress" at the fracturing depth can be determined.

Hydraulic fracturing is now widely used for in situ stress determination, but interpretation of the fracturing results as presented above can be misleading. Consider, for example, the situation where the borehole is vertical and a vertical fracture is observed. This does not necessarily ensure that the vertical stress is a principal stress. If we assume, for example, that a shear stress acts parallel to the axis of the hole (τ_{xz} in Fig. 6). In this case the principal stresses σ_H and σ_h are, in fact, the secondary principal stresses (s_1 and s_2 in Fig. 6). The location, in the horizontal (xy) plane, of the maximum and minimum stress concentrations around the hole is the same as when σ_H and σ_h are true principal stresses, and, assuming that the state of stress is essentially constant over the packed off interval, the minimum (least compressive) stress concentration will occur at the same (two) orientations (180° apart) at each level, forming a vertical trace along the wall of the hole. The effect of the shear stress acting parallel to this trace is to rotate the orientation of the maximum and minimum stresses (acting at each point along this trace) at some angle to the trace. The true minimum (least compressive) stress concentration will be at this angle to the trace, and tensile fracture should be initiated at this orientation rather than vertically.

To propagate in this direction, however, the fracture would need to extend around the periphery of the hole, where the stress concentration becomes rapidly more compressive. Extension around the hole[11] in this

[11] It could be imagined that a fracture could propagate in this minimum principal plane *in the interior of the roc*k, but how the transition from the borehole to the interior would take place is not obvious.

direction would tend to be suppressed. Also, the same micro-cracking initiation process could develop all along the vertical trace. It could be that a series of inclined microcracks are all formed "simultaneously" along the trace joining to form what would be interpreted macroscopically as a single vertical fracture. Such a fracture is often taken to indicate that the borehole is parallel to a principal stress direction.

The example above illustrates the potential for misinterpretation of the in situ state of stress from the interpretation of effects at the borehole wall. Laboratory teats have shown that hydraulic fractures can change orientation as they propagate into the rock from a borehole. The Hydraulic Test on Pre-existing Fractures (HTPF) method developed by Cornet and Valette [21], and discussed elsewhere in this Special Issue, overcomes this ambiguity of classical hydraulic fracturing stress determination by pressurizing and opening only (planar) fractures that are known to exist in the rock before pressurization. By observing the pressure (normal stress) required to reopen several such fractures that are known to exist at different orientations, it is possible to solve for the complete stress tensor in the vicinity of the test interval.

The hydraulic fracturing method benefits from the fact that the technique is a vitally important factor in the economics of petroleum production operations. The fluid and rock mechanics principles governing hydraulic fracturing are under constant vigorous research by petroleum companies, so that advances in its use for stress determination, and interpretation of field results seem assured.

11. Other techniques of stress determination

It is not possible in one review article to cover all approaches that have been, and are, used for the purpose of stress determination in rock. There are other techniques that may have advantages in particular situations, e.g. undercoring and the borehole slotter, (these are discussed in the URL report referred to below). The main aim of this review is to illustrate the general principles that have been used over the past half-century. It appears that most of the basic ideas have not changed significantly, but increased experience and technological improvements are making it possible to achieve significant gains in performance.

Stress influences properties other than the elastic response, which has been the underlying principle on which the techniques discussed above are all based. Several techniques associated with the acoustic properties of rock for example, have been found valuable, such as observations of changes in the velocity of wave propagation and resistivity with stress in rock pillars. Usually, however, the changes were most significant at low stress levels, becoming almost independent of stress at the higher levels, and thus of limited application as indicators of possible danger e.g. in the detection of highly stressed mine pillars. (These are discussed by Leeman [5, Sept. 1964]). Recent impressive advances in micro-geophysical techniques suggest that the limitations may be overcome and that there could be applications to stress determination or stress monitoring.

12. Core discing

The pre-loaded nature of rock masses has consequences in rock stress observation. The process of boring of holes to obtain cores results in stress concentrations directly at the coring bit/rock interface. As the core is formed, the annular groove causes the in situ stresses to be redistributed, creating high-induced stresses across the core. This can result in damage (irrecoverable strains and microcracks) to the core. If the in situ stresses are high, and the rock brittle, this can result in 'core discing'—the core is produced in the form of thin 'poker chips'. The thickness of the chips decreases as the stress intensity increases; in extreme cases, the discs can become so thin that they have the appearance of *milles feuilles*, or *flaky pastry*. Observation of discing in cores is often taken as evidence of high stress zones in the rock.

13. Borehole breakout

The stress concentrations that develop around a borehole in stressed rock can result in inelastic deformation, damage, and fall out of broken rock in the zones of maximum stress concentration. The hole develops an oval or elliptical shape. The major axis of the deformed (breakout) shape is taken to be coincident with the direction of the minimum secondary principal stress (s_2 in Fig. 6) with the maximum stress (s_1) orthogonal to it. Read et al. [22] have observed that the axes of the breakout may sometimes not pass through the centre of the borehole. As noted earlier, shear stresses acting along the borehole do not influence the position of maximum and minimum stress concentration around the borehole. Read et al. suggest that the asymmetry is a consequence of the stress distribution (influenced by the shear stresses) and a possible onset of damage in the rock ahead of the coring bit. This suggests that observation of asymmetric breakout may be an indication that the borehole does not coincide with a principal stress direction.

14. Core damage

Core discing and breakout are clear indications that rock can be damaged during coring. It is important to recognize that all core that is recovered from depth has been subject to stress concentration and can be damaged as it is formed. While not as obvious as in the cases of discing and breakout, this damage can have important practical consequences. When direct access to the underground is not available, tests on borehole cores provide the primary basis for prediction, not only of the type of rock to be encountered but the mechanical properties of this rock. In the case of radioactive waste repositories, for example, prediction of the long-term ('creep') will be based on laboratory tests on core specimens. There are cases where, for example, the laboratory tests indicate that there is no threshold of deviator (shear) stress below which creep will not occur. Hydraulic fracturing measurements at depth in the same rock formations indicate that the maximum horizontal (principal) stress is larger than the vertical stress. The rock is of the order of 200 million years old. If we assume that the rate of change of tectonic stresses is insignificant compared to the stress relaxation rate due to rock creep, it would seem that a rock with no threshold for creep should develop a uniform (hydrostatic) state of stress at depth. Why then is the non-uniform state maintained? Could the explanation be that the laboratory specimens are damaged and hence exhibit mechanical behaviour that is not representative of the rock in situ i.e. that for the rock in situ there is a threshold deviator stress below which the rock does not creep? This has important implications with respect to the validity of mechanical properties derived from cores.

15. Importance of stress estimation

As can be inferred from the discussion in this paper, estimation of the state of stress in rock can be a difficult and in some case quite costly undertaking. Instruments may be damaged or lost or, in some cases, may need to be placed permanently in place. It is important therefore that the benefits expected from the measurements be carefully evaluated and the programme well planned—which is the motivation for Parts 1 and 4 of the ISRM Stress Estimation Suggested Methods published in this Special Issue. A serious difficulty of many rock engineering projects is to estimate the strength and deformability of the rock on the scale of the excavations or structures involved. The problem to be addressed is more one of understanding the interaction between the in situ stress (and the changes induced by engineering) and the resistance of the rock to the stresses imposed on it. Will the structure be stable or not? It may well be that some test which could assess this interaction directly would be of greater practical value. Can the measurement system provide warning of potential instability or similar problem and allow remedial measures to be taken to correct the problem? Observing the change in rate of convergence of a tunnel over time to assess the adequacy of the support system, mentioned earlier, is a good example of such a measurement system. The trou-test used by coal miners in France is another.

Some coal seams are known to be susceptible to outbursts on the longwall face. The outburst is an explosive ejection of coal from the face that can result in severe damage and sometimes fatalities. The outburst appears to involve a combination of the high stress concentration (front abutment pressure) that is associated with the longwall operation, the stress reduction in the direction of advance, especially close to the face, possible high gas pressure in the coal, resulting in low 'effective strength' of the coal. The miners monitor this problem by drilling holes into the face at intervals along it. A simple plastic bucket, calibrated on the side to indicate the volume, is placed to catch the cuttings as the hole is deepened. The volume of cuttings is recorded as a function of the depth of the hole. If the region of high stress and potential outburst (the coal may not always contain the free gas under pressure) is encountered, the coal around the drill hole starts to collapse rapidly (mini-outburst) and the volume of cuttings per unit of advance increases dramatically. Rules have been established to define the proximity of the potential outburst region to the face to guide actions to be taken (e.g. destressing of the coal by water infusion or, possibly, blasting). While stresses play a key role in the outbursts, they alone are not the critical factor, and direct determination of the stresses is not the most meaningful indicator of the severity of a problem.

16. Numerical modelling and stress estimation

Numerical modelling can play a vital role in planning a programme of stress estimation, especially in deciding on the location of the stress tests. The model analyses are particularly valuable in situations where access to the underground is either limited or non-existent, as is often the case, for example, in petroleum exploration and production, and in the search for sites for nuclear waste isolation. The numerical model should try to include as much as possible of the relevant geological structure that may significantly affect the stress distribution. Faults are particularly important, since (depending on the cohesive and frictional properties assumed for them)[12] they can change significantly the

[12] The model could examine the effect of different assumptions for the fault properties on the in situ stress distribution. This exercise could help decide on measurement location.

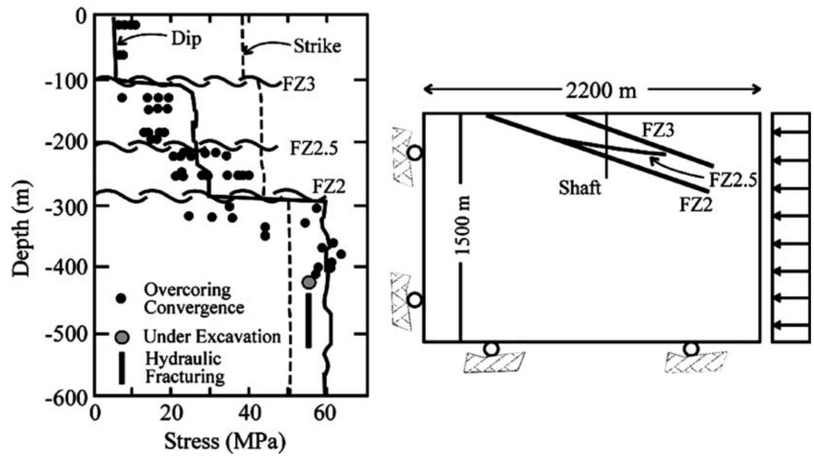


Fig. 8. Observed in situ stresses at various depths at the URL, Pinawa, and numerical modelling interpretation.

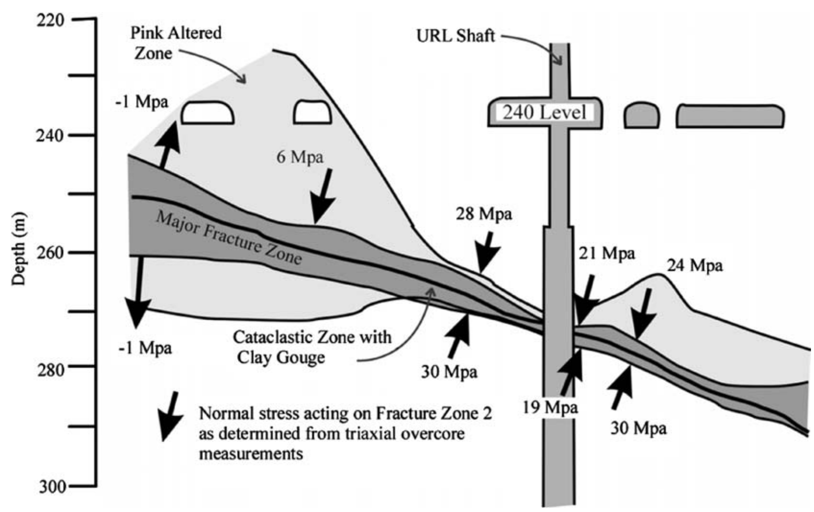


Fig. 9. Measured variation of normal stress along the thrust fault zone FZ2 at the URL, Pinawa, Canada. The horizontal and vertical scales are the same.

stress distribution in the interior of the rock. An aspect of numerical modelling that requires considerable judgment is to strike a balance between model complexity and over simplification. A well-prepared model can suggest optimal locations for the in situ tests and can indicate to planners how many tests will be necessary.[13]

A numerical model can also assist in the interpretation of stress observations, as illustrated by Fig. 8. Stress determinations at the URL, Pinawa, have been made at various depths, from near surface to somewhat more than 500 m, using several techniques. Results indicated a strong influence of the thrust fault zones (FZ3, FZ2.5, and FZ2), with a large increase from a variety of values 0–30 MPa above FZ2 to 60 MPa below FZ2. A numerical model incorporating frictional slip on the thrust faults and boundary conditions as shown in Fig. 8 was able to demonstrate that slip along the faults would significantly reduce the stresses above F22, to values consistent with those observed. It is seen also that the orientation of the maximum principal stress changes by 90° above and below FZ2 (i.e. the stress in the strike direction, dashed line, exceeds the stress in the dip direction, solid line, above FZ2, whereas the reverse is true below FZ2).

Changes in orientation of stresses with depth can also occur due to the superposition of the effects of ancient tectonic regions (in older rock structures and formations) with current tectonics acting on both older and younger rock formations.

Fig. 9 shows the results of a series of (normal) stress determinations made at several locations along the thrust fault fracture zone FZ2. These clearly indicate the considerable variability in stress over relatively short distances, correlated to the thickness of the variation in thickness of the gouge zone along the fault. A raise bore shaft driven through the region of high local stress

[13] Boreholes used for stress estimation are often multi-purpose, being used for geological and geophysical information as well as rock mechanics.

found virtually no water flow when passing through the (thin) fault zone. A hole drilled some 25 m or so from the raise into the low stress region encountered water under high pressure and large flows into the hole.

17. Conclusions

Although a stress estimation campaign can often provide information of great value to a project, it is important to be well aware of how the information is to be used. Reference has been made earlier to situations in which, although the state of stress was a primary influence in a problem, some other measure can be of greater benefit in understanding or controlling the problem. Stress determinations made in conjunction with other measurements of deformation, microseismic and associated geophysical techniques are likely to be more informative than when used alone.

In almost all cases, the aim of stress estimation is to gain an understanding of the state of stress, and its variability, *over a region*. One or two measurements at specific locations may not be sufficient to allow extrapolation to the larger region. In some cases, as in mountainous areas, topographic effect may produce considerable local variability in stresses.

The value of numerical modelling, a tool not available to rock mechanics in the 1950s, in helping to understand how stresses may vary locally and inform planning for stress measurement has already been emphasized. Eventually, after reflection and planning such as mentioned above, a strong case can sometimes be made that measurements to establish the state of stress in a region can provide valuable information not possible otherwise. Usually, measurements in several locations are necessary to extrapolate the results with confidence to other situations.

In some cases, especially where no direct access at depth is possible, one is limited to making measurements in one or a small number of boreholes. In such cases, efforts should be made to make measurements at different horizons in the boreholes, so that variability in one dimension at least can be estimated. Hydraulic fracturing tends to be the method of choice for deep boreholes (several hundreds of thousands of metres). Where fractures are intersected sufficient to allow its use, the HTPF technique [21] can help remove some of the uncertainty associated with classical hydraulic fracturing.

In regions where little or no information is available, estimation of simply whether or not the tectonically induced ('horizontal') stresses exceed the gravitational ('vertical') stresses can be valuable in the planning of engineered structures in or on the rock. One hole can sometimes suffice to indicate whether or not this is the case. Where underground access is available, the method used often depends on the familiarity of the investigator with one or more particular techniques. Considerable advances have been made in the ability to install instruments and transmit (and telemeter) data, but the basic techniques used have changed little. Much still depends on the skill of the individual installing the instruments and interpreting the results.

Extending stress determination techniques to depths of interest in tectonophysics to study of plate motions and earthquake mechanics is a formidable, but important and exciting challenge. Observations by Scotti and Cornet [23], suggesting that the use of focal plane solutions to indicate the orientation of stresses in the vicinity of faults at depth may not be reliable, illustrates the uncertainties to be addressed. It seems likely that, at such depths, some non-invasive techniques will be required, either to supplement or replace drilling.

As a primary mechanism in controlling the deformation and stability of rock masses, on all scales, the state of stress in rock will remain a topic of major interest in rock mechanics, and methods to establish what the actual stresses are will continue.

Appendix A

In situ rock stress research at the Canadian Underground Research Laboratory (URL), Pinawa[14]

Rock mechanics staff of the URL, Pinawa has carried out extensive sustained efforts, over many years, to use and improve many of the types of instruments discussed in this paper. The report by Martino et al. [17] is the most comprehensive and useful survey known to the author for anyone considering practical application of these techniques for stress determination.

The granite found at the URL, especially at the deeper (420 m) level, is very homogeneous and free of major fractures. This situation has allowed the team to examine a number of important issues in rock mechanics and stress determination, and to investigate possibilities for improving the performance of various tools. For example: A Deep Borehole Deformation Gauge (DBDG), a modification of the US Bureau of Mines Borehole Deformation Gauge (BDG) has been designed to operate at depths up to 1000 m, Fig. 10. It has been tested to 120 m only, because below this depth, core

[14] The Underground Research Laboratory (URL) at Pinawa, Manitoba is a facility of Atomic Energy of Canada Limited (AECL) established to examine the feasibility of geological isolation of high-level radioactive waste in Canadian Shield Granite. Regrettably, AECL has decided that it can no longer afford to support the URL. The facility, which has made major contributions to rock mechanics knowledge, is scheduled to be closed later this year. The report cited above is one example of the excellent work of the URL team of researchers.

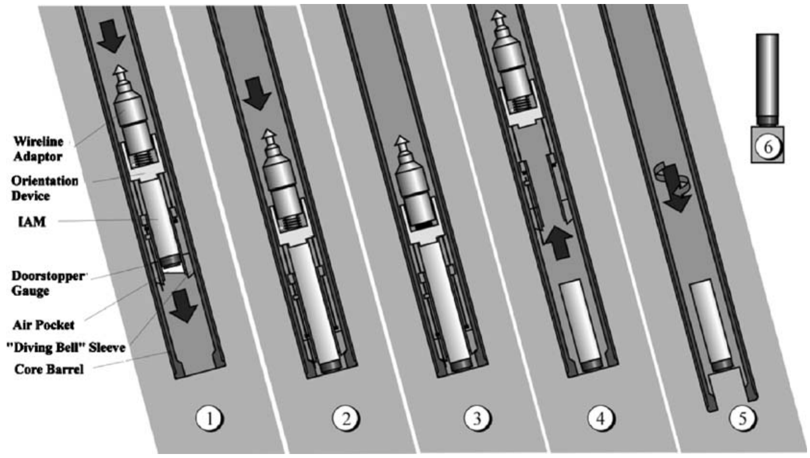


Fig. 10. The installation of the Deep Doorstopper Gauge System. Left to right, (1) the DDGS is lowered down hole with the wireline cable; (2) the DDGS reaches the bottom; (3) the mass of the upper assembly creates an impression in the orientation device; (4) after the glue has set the installation assembly is retrieved with the wireline; (5) the IAM* starts monitoring and the diamond drill extends the borehole; (6) the IAM and core are retrieved with the core barrel [*The Intelligent Acquisition Module (IAM) is a batter-operated data logger that stays attached to the doorstopper gauge during stress measurement testing].

discing caused by the high stress field precludes successful measurements. A Deep Doorstopper Gauge System (DDGS) has also been designed to make measurements to a depth of 1000 m and tested at several depths below the 420 m level. Fig. 10 illustrates the DDGS installation procedure. Although results to date have been disappointing (12%) success, this is due mainly to down hole battery life and inconsistent adhesive performance. The staff is confident that the goals of the DDGS are achievable.[15]

CSIR and CSIRO cells have been modified to allow continuous reading during overcoring. This allows the performance of each gauge to be followed; debonding can be identified, allowing more effective interpretation of the results using only gauges that are performing correctly. It is noted (p.17) that, at the URL, CSIRO-HI cells have operated continuously for over two years in water-filled boreholes. The author recommends that anyone contemplating stress determination work in rock obtain a copy of the report by Martino et al. [17]).[16]

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[16] Copies available from Ontario Power Generation, Inc. Nuclear Waste Management Division (H16) 700, University Avenue, Toronto, Ontario, Canada M5G 1X6. Fax: (416)-592-7336.

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