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"ROCK FAILURE DUE TO APPLIED HYDRAULIC FORCES"

By: John S. Sheffield, Jr.
Humble Oil & Refining Company

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Introduction

The purpose of this paper is to investigate rock failure by applied hydraulic forces. This investigation will be approached through a review of basic rock mechanics concepts.

Rock mechanics is the one most important physical consideration from the time a well is spudded until abandonment. Rock mechanics is defined as the area of knowledge which deals with how rocks that comprise the outer part of the earth's crust behave and respond to applied forces. We are interested in three basic modes of rock behavior--failure, failure prevention, and yielding.

Discussion

Rock behavior influences several phases of well life including drilling, completing, and operating. Specifically it influences (1) formation drillability, (2) lost returns, (3) hole sloughing, (4) hole closure, (5) abnormal pressure, (6) perforating, (7) fracturing, (8) additional recovery injection pressures and pattern efficiency, and (9) squeeze cementing.

This paper will be limited to rock behavior and response due to applied hydraulic forces. Other important man-induced forces that affect rock behavior are mechanical, such as the drill bit, packer, and fracture proppant, and explosive forces, which include the conventional jet perforating charge, proposed jet drilling charge, and the proposed atomic fracturing charge.

Before we can determine rock behavior due to applied forces, we need to briefly consider in-situ forces and rock properties.

In-Situ Rock Stresses

Materials that make up the earth's crust can behave as an elastic, plastic, or elastic-plastic material depending upon material nature, applied force, and temperature. Materials that have a tendency to behave plastically at low stress levels (depths less than 20,000 feet) are shale and salt and among the materials that normally behave elastically are sandstone and limestone.

Normally, when we study a material, we like to consider it homogeneous and isotropic. The rock that we are concerned with is porous and laminated; therefore, rigorously speaking, the rock in question is neither homogeneous or isotropic since homogeneous and porous are contradictory adjectives and likewise isotropic and laminated are opposites. However, initially in our stress analysis, we will consider the rock to be an elastic, homogeneous, isotropic, porous material.

Pore Pressure

Since rock is porous and contains fluids under pressure, we should examine how interstitial fluid pressure influences rock stresses. Because interstitial fluid is normally inert relative to rock, we need to concern ourselves only with how interstitial fluid pressure or pore pressure effects rock stresses.

Illustrated in Figure 1 is a porous sandstone formation containing fluid under pressure. This sandstone has a sealing plate or a reservoir fluid boundary plate, which has an area A, that is transmitting a gross stress, $\bar{\sigma}_z$, to the rock skeleton. The fluid pressure affects two elements of the model, the rock skeleton and the gross overburden stress. It has been proven that a hydrostatic compressive stress will produce negligible stress effects on the solid skeleton for all pressures of interest in hydrocarbon production. The gross force, $\bar{\sigma}_z A$, is supported by the rock through two mechanisms. Part of the force will be supported by the rock skeleton and part by the fluid pressure in the pores. If the rock surface porosity is equal to θ_s, then the force balance is equal to

$$\bar{\sigma}_z A = F + \theta_s Ap \qquad (1)$$

where F is the force supported by the rock skeleton. This equation is rewritten to describe the net effective rock stress

$$\sigma_z = \frac{F}{A} = \bar{\sigma}_z - \theta_s p. \qquad (2)$$

In evaluating the magnitude of surface porosity, we see that $\theta_s = 1$ for a rock composed of perfect spheres. The literature indicates that surface porosity of most sandstones varies from 0.85 to 0.97. Therefore, it is common practice to assume $\theta_s = 1$ which reduces Equation 2 to

$$\sigma_z = \bar{\sigma}_z - p, \qquad (3)$$

or net effective rock stress equals gross stress minus pore pressure. This equation is a very important concept in rock mechanics as it has been proven beyond any doubt that rock behavior is dependent upon level of effective stress and not gross stress.

Three Principal Stresses

The stress condition of any body can be described in terms of the magnitude and direction of only three stresses--the principal stresses. Therefore, we should consider these stresses in our analysis. The three principal stresses imposed on a porous rock element are illustrated in Figure 2. They are always mutually perpendicular and in-situ principal stresses are always compressive. By convention, the vertical stress is called σ_z, the least compressive horizontal stress, σ_x, and the other horizontal stress, σ_y.

Vertical Stress

The vertical principal stress, σ_z, is generally taken perpendicular to the earth's surface. Normally, the gross vertical stress on a rock is equal to the earth's bulk density times depth. The bulk density of saturated rock is usually considered to be 1 psi per foot; therefore, the effective vertical rock stress equals rock depth times 1 psi per foot minus pore pressure. One exception to this value is when overlying formations act as beams or domes. There are several authoritative references in the literature suggesting the possibility of this situation. In such cases, net vertical stresses would be reduced accordingly.

Horizontal Stresses

The magnitude of the two principal horizontal stresses, σ_x and σ_y, is a function of the vertical stress, Poisson's ratio (the ratio of unit transverse strain to unit axial strain), rock coefficient of internal friction, and formation folding and faulting. The origin of horizontal stresses is not as straightforward as the origin of vertical stresses; however, we are conscious of their presence as evidenced by earthquakes, earth quivers, faulting, and folding. These earth failures are partly a result of enormous horizontal stresses on the earth's crust. There are several theories as to where these stresses originate or what causes them. The most attractive theory is that the upper earth's mantle below the earth's crust is plastic and is moving very slowly causing uneven drag forces against the earth's crust. These drag forces cause uneven pulling and torturing of the earth's crust which in turn effect horizontal stresses and rupture in some instances.

If we assume the surface of the earth is a semi-infinite, perfectly elastic solid, the effective horizontal stresses due to the effective weight of the overburden are equal in all directions and have the following value

$$\sigma_x = \sigma_y = \frac{\nu}{(1 - \nu)} \sigma_z. \qquad (4)$$

If (Poisson's ratio) $\nu = 0.25$, a representative value for sandstones and limestones, then $\sigma_x = \sigma_y = \frac{1}{3} \sigma_z$ and if $\nu = 0.5$, which is the case when the rock is in a pure plastic state, then $\sigma_x = \sigma_y = \sigma_z$, or the stress system is hydrostatic.

An important clue pertaining to the magnitude of horizontal forces can be obtained from a study of faulting or other geological movements in the vicinity of the formation in question. If we assume that the Mohr theory of shear failure describes normal faulting, we can conclude that a horizontal stress is the smallest of the three principal stresses and is one-third the vertical stress. Taking this assumption one step further, we can conclude that the minimum value of $\frac{\sigma_x}{\sigma_z} = \frac{1}{3}$ because if this ratio were smaller than one-third, normal faulting would occur.

Another indirect indicator of the magnitude of the least horizontal stress is fracture propagation pressure. We will see later on in the discussion that a vertical fracture is initiated and propagated along a plane perpendicular to the least compressive horizontal stress. This being the case, fracture propagation pressure, usually referred to as instantaneous shutdown pressure, has to overcome the least effective compressive horizontal stress, σ_x, pore pressure, and rock cohesive strength to extend the fracture. Assuming that at the fracture tip there is a point stress, a negligible amount of pressure is required to overcome rock cohesive strength. Therefore, fracture propagation pressure is equal to the gross least compressive stress,

$$p_{wf} = \sigma_x + p_o = \overline{\sigma}_x. \qquad (5)$$

For a fracture propagation gradient of 0.75 psi per foot, formation pore pressure of 0.45 psi per foot, and a gross overburden pressure of 1 psi per foot, the ratio of least compressive effective horizontal stress to effective overburden stress $= \frac{\sigma_x}{\sigma_z}\left(\frac{0.75 - 0.45}{1.00 - 0.45}\right) = \frac{1}{1.83}$. It can be concluded that principal effective rock stresses are not necessarily equal to each other, effective vertical stress is equal to 1 psi per foot minus pore pressure, least effective horizontal stress is approximately one-half the effective overburden stress and no less than one-third, and the magnitude of all three principal stresses is strongly dependent upon pore pressure, formation elastic and plastic characteristics, and the geologic movements in the vicinity of the formation in question.

Rock Properties

In order to determine the rupture or breakdown pressures required to initiate a fracture or to determine thé rupture pressure that will cause lost returns, it is necessary to consider rock mechanical properties. The most important property as far as rupture is concerned is tensile strength.

Tensile Strength

Rock tensile strength is a notoriously undependable quantity and is hard to measure in either the laboratory or field. For flawless sandstone test cores, tensile strength ranges from zero to over 1,000 psi. However, small rock volumes will withstand greater stresses than larger volumes. This is true of all brittle materials and is even observed in many ductile materials. For example, a steel wire's tensile strength will increase as wire diameter is decreased. The reason for this is that a large volume of a material is more likely to contain a flaw or weak spot than a small volume. Another way to explain this phenomenon is to point out a simple anology. If a single brick is pulled out of a brick wall and tension failed, we will probably find that some finite amount is required to fail the brick. On the other hand, if we attempt to put a tensile stress on the entire brick wall, we will probably find the wall will fail at zero tension, or some small amount of tension, because the mortor has cracked, the wall has settled and cracked, and because of irregularities in wall construction. Therefore, the assumption is generally made that in-situ rock tensile strength is zero. We can support this assumption with failure equations that we will consider later; however, at the present state of the art, the statement that rocks will fail at zero tension is conjecture, hopefully intelligent conjecture.

Compressive Strength

Although hydraulic rock failure is not directly related to rock compressive strength, several salient facts have come out of compressive test work that are worth considering. There has been an extensive amount of laboratory compression test work performed and several widely accepted conclusions on how confining forces, pore pressure, temperature, and strain rate affect ultimate compressive strength.

When rocks are subjected to a compressive load and at the same time laterally confined with some amount of pressure, it is found that rock compressive strength increases with increased confining pressure. Also, most rocks have small increases in elasticity and yield stress as confining pressure is increased and magnitude of rock pore pressure does not influence ultimate strength so long as effective confining pressure remains the same. For example, if a rock is subjected to a gross confining pressure of 3,000 psi and a pore pressure of 1,000 psi, the net confining pressure will be 2,000 psi. The rock will fail and behave in compression, for practical purposes, identically to a similar rock that is confined with a pressure of 2,000 psi and zero pore pressure. We can conclude from this point that if gross confining load remains constant, which one would expect in a particular geographic area, abnormal pore pressure in a formation would tend to decrease the rock compressive strength because if pore pressure is increased and gross confining pressure remains constant, then effective confining pressure is decreased.

The literature states that in temperature ranges up to 550 F and confining pressures of up to 30,000 psi, sandstone is not appreciably affected by heating; however, other materials that have a tendency to behave plastically at low stress levels, such as shales and salt, are significantly influenced by temperature. Materials of this type at room temperature and under ordinary compression exhibit brittle failure; however, under triaxial compression, they lose their brittleness at low stress levels and assume a plastic or plastic-elastic state which is a function of temperature for any particular stress--the higher the temperature, the more pronounced plastic flow.

Rocks when subjected to a load display a time-dependent response. When strain rate or stress rate is increased, yield point and ultimate compressive strength and Young's modulus increase. However, within the usual range of testing, these mechanical properties are only slightly influenced by strain rate.

Summarizing rock properties, although we will assume, in the remainder of this paper, that rock will fail under zero tension loading, it should be recognized that some rocks will require a finite amount of tension to fail and rock tensile strength will probably be influenced by the factors affecting rock compressive strength.

Hydraulic Induced Rock Stresses

Although we have oriented our investigation so far toward hydraulic rock failure or prevention of failure, material presented on in-situ rock stresses and rock properties is applicable to rock mechanics in general. Now we shall turn our thinking to hydraulic induced rock stresses around a wellbore. An equation needs to be considered that describes the stress state around the wellbore for three conditions: (1) when wellbore fluid pressure equals pore pressure, (2) when wellbore fluid is a penetrating fluid and is greater than pore pressure, and (3) when wellbore fluid is a nonpenetrating fluid and is greater than pore pressure. A penetrating fluid is defined as a fluid that can be pumped into a rock at some finite pump rate without rupturing the rock. A non-penetrating fluid is defined as the converse of the aforementioned definition and is comparable to a low-water-loss drilling mud.

An equation has been developed* to describe the hydraulic fracture initiation process in terms of hoop stress. This equation is based on Lame's expression for stresses in a thick-wall cylinder and is similar in some respects to equations developed by others; however, the fact that a porous rock can be subjected to a transient pore pressure due to a penetrating fluid has been taken into consideration.

Wellbore stresses are expressed in terms of a hoop, radial, and vertical stress. These stresses are illustrated in Figure 3 on a drilled rock that is triaxially stressed by σ_x, σ_y, and σ_z. Hoop stress, σ_θ, is a tangential stress, and radial stress, σ_r, is perpendicular to the hoop stress. Radial failure results in hole sloughing or caving while hoop failure results in vertical fracturing. The equation* for hoop stress σ_θ is expressed in the following relationship.

$$\sigma_\theta = \frac{\sigma_x + \sigma_y}{2}\left[1 + \frac{a^2}{r^2}\right] - \left[\sigma_y - \sigma_x\right]\left[1 + 3\frac{a^2}{r^2}\right]\cos 2\theta$$

$$+ (p_w - p_o)\frac{a^2}{r^2} \qquad (6)$$

$$+ \left[(1 - 2\nu)\beta + 2\nu\right] p - p_o + \frac{(1 - 2\nu)(1 - \beta)}{r^2}\int_a^r (p - p_o)\, r\, dr$$

This is a building-block equation as it is put together in three parts. The first line of the equation represents a compressive stress that is a result of a drilled rock being horizontally stressed by σ_x and σ_y. The second portion of the equation on Line 2 represents a tensile stress exerted by a wellbore fluid pressure that is greater than pore pressure. The third part of the equation indicates the tensile stress on the hoop exerted by a penetrating fluid injected into the surrounding wellbore rock. Equation 6 is graphically

* See acknowledgement.