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LARGE VOLUME TESTS IN ROCK MECHANICS

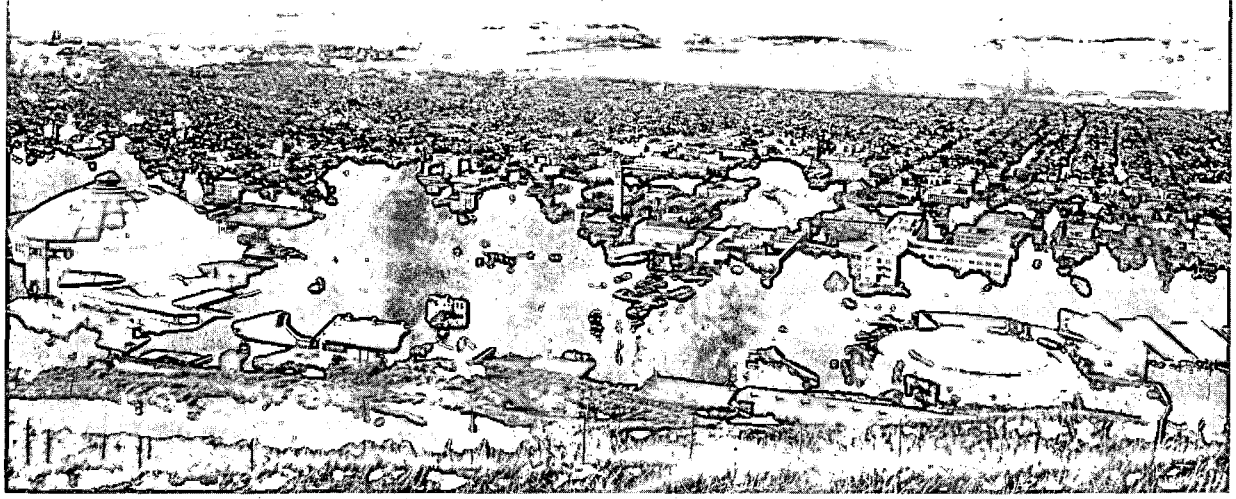
D.J. Watkins and R.K. Thorpe

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LARGE VOLUME TESTS IN ROCK MECHANICS

D. J. Watkins* and R. K. Thorpe**

Abstract

Few laboratory tests have been performed on rock specimens with principal dimensions in excess of 30cm. While available data have shown that sample size can have a major influence on measured properties, the magnitude of these effects remains uncertain. Interpretation of laboratory data is further complicated by the effects of sample disturbance, stress relief, and the inability to fully reproduce in situ boundary conditions. In situ methods of testing enable more representative volumes of a rock mass to be studied in a relatively undisturbed state, but are limited by uncertainties of boundary conditions and the inability to reproduce generalized stress conditions. For engineering design, rock mass properties must be estimated by synthesis of data from a variety of laboratory and in situ testing procedures, rock mass classification, and fracture characterization techniques. An experimental program to study the strength and permeability of a 1.0m diameter by 2.0m high specimen of granitic rock provides an example of large volume laboratory tests. This sample failed in unconfined axial compression at a peak stress of 7.4 MPa compared to uniaxial strengths on the order of 100 MPa measured from 5.2cm diameter samples. Initially, as the axial stress was increased from zero to 5.56 MPa, the permeability of the specimen decreased from 5.25cm/sec to 0.5cm/sec, but then increased to some 2.0cm/sec due to shear dilatancy and fracture opening associated with failure of the sample.

Introduction

The design and analysis of underground openings, natural and engineered slopes, dams and foundations require knowledge of the constitutive properties of the rock at the site. In many mining and civil engineering projects where the flow of water into or out of the rock mass must be considered, it is also necessary to study the hydraulic properties of the rock mass. Unlike some commonly used construction materials, such as steel and concrete, rock is a highly complex and heterogeneous medium. Fractures and other discontinuities are present in all rock masses. They can range in scale from microscopic to major features such as those associated with faulting and stratigraphic bedding. This complexity is reflected in its mechanical and hydraulic

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properties and its behavior in response to loading. In practical design problems, discontinuities frequently dominate the behavior of the rock and thus tests to measure rock properties must be conducted on a scale sufficient to account for their contribution to macroscopic behavior of the rock mass.

Role of Large Volume Tests in Rock Engineering

It has long been recognized that, with respect to strength and deformability, a rock mass behaves differently from the small specimens that have traditionally been tested in the laboratory (Heuzé, 1980). More recently, the potential for a similar scale effect has been observed in experimental determination of the hydraulic properties of fractures (Witherspoon et al, 1979). Because very few laboratory facilities are available that have the capability to test specimens with principal dimensions in excess of about 30cm, there remains considerable uncertainty as to the magnitude of scale effects on the measured properties of rocks. Large scale laboratory tests are difficult and expensive to perform, but further research on large specimens is needed so that rock mass properties can be more reliably estimated from routine tests on small specimens. In some engineering projects, accurate prediction of rock mass deformations is critical and justifies laboratory tests on very large specimens. For example, changes in rock mass permeability resulting from excavation and thermally induced displacements are a major concern in the design of underground facilities for disposal of hazardous wastes.

Another complication that affects interpretation of laboratory data is the sensitivity of earth materials to sampling disturbance. In addition to direct mechanical damage that inevitably occurs, particularly to joints and other discontinuities, the sampling process causes the specimen to be stress relieved. Because rock properties are stress history dependent, these effects are irreversible. Even when the in situ stress state and boundary conditions are known they can rarely be accurately reproduced in the laboratory and these limitations add to the uncertainties in measurement of material properties.

Uncertainties in estimates of rock mass properties developed from laboratory tests have led to the development of in situ testing procedures in which relatively large volumes of the rock mass, or sections of discrete discontinuities, are tested in a relatively undisturbed state. However, the

boundary conditions in in situ tests are usually poorly defined and it is not possible to subject the rock to the generalized stress conditions that can be reproduced in the laboratory. It is also important to recognize that the volume of the rock mass affected by the construction of a major facility, such as a hydroplant turbine gallery or a mine, is orders of magnitude larger than the volume of rock influenced by even the largest practical in situ tests. Thus, design and analysis cannot rely solely on a deterministic approach calling for direct measurement of the properties of all elements within the rock mass. It is necessary to estimate these properties by synthesis of data from a variety of techniques that include not only laboratory and in situ rock mechanics tests but also important contributions from the disciplines of geology, geophysics and hydrology. Extensive work is required, particularly in crystalline rocks, to map and characterize fractures and discontinuities (Thorpe, 1979 and 1981). Several empirical and semi-deterministic rock mass classification systems have been developed that provide methodologies for data synthesis. See for example Barton et al., 1980; Bieniawski, 1979; and Hardy and Hocking, 1980. However none of these methods is universally applicable. Selection of the number, type and scale of laboratory and in situ tests, methods of rock mass and fracture characterization and analytic procedures is one of the most important elements in design. Each geologic site is unique and the rock mechanics investigations to be made and the problems to be resolved must therefore be considered on a site and design specific basis. Ultimately, if uncertainties regarding rock mass properties are to be minimized, the estimates and inferences developed from site investigation and testing programs must be verified by performance data gathered from the completed structure of full-scale prototype test facilities constructed on site.

Types of Large Volume Specimens

Table I summarizes the principal types of large volume laboratory specimens and some of the tests that can be performed on them. For the purpose of this discussion, the term "large" will be assumed to apply to specimens of rock with principal dimensions on the order of one meter. Within this class, typical cylindrical specimens have diameters within the

range of 0.5m to 2.0m and lengths between 1.5 and 3.0 times the diameter (Watkins, 1981). Joint and discontinuity surfaces contained within prismoidal samples have areas on the order of $0.25m^2$ to $1.5m^2$. Other specimen shapes may be required for some testing purposes but the foregoing are the most commonly encountered.

Laboratory tests are normally performed on rock samples with maximum dimensions limited to several centimeters. Cylindrical samples for routine strength and deformability testing are obtained from cores with diameters of 5 to 8cm. Specimens with diameters from 10 to 20 cm are less common and triaxial tests have rarely been performed on specimens larger than 30cm. However, a unique series of experiments was conducted by Singh and Huck (1972) on cylindrical samples up to 91.5cm diameter. Laboratory direct shear tests on cubic and rectangular block samples have generally been restricted to specimens with maximum dimensions of some 23cm (Goodman, 1969) but Krsmanovisc and Langof (1964) have reported tests on blocks of rock 40cm by 40cm plan dimension. Permeability and fracture conductivity tests on a 0.91m diameter by 1.82m high cylindrical sample of intact rock containing an artificially induced fracture have been reported by Gale (1975) and Witherspoon et al. (1977). Large specimens have also been employed for several other more specialized purposes such as development of down hole geophysical tools, research on fault slip mechanics (Dieterich, 1978), study of hydraulic fracturing (Haimson, 1981) and investigation of acoustic emissions (Estey et al., 1980).

Strength and Permeability Tests on an Ultra-Large Specimen of Granitic Rock

An example of large volume laboratory testing is provided by a program conducted to investigate the strength and permeability of a specimen of quartz-monzonite recovered from an iron ore mine at Stripa in Sweden (Thorpe et al., 1980). The Stripa mine is the site of a series of large-scale in situ hydrologic and geotechnical experiments that are part of the Swedish-American Cooperative Program on Radioactive Waste Storage in Mined Caverns in Crystalline Rock (Witherspoon and Degerman, 1978).

The 1m diameter by 2m high cylindrical specimen (see Figure 1) was recovered from the rib of an entry by drilling a circular pattern of horizontal

holes around its periphery. This was done while the rock was held in compression by a rock bolt through the central axis (see Andersson and Halén, 1978). After shipping to the laboratory, the specimen was prepared for testing by capping with reinforced concrete. As shown in Figure 2, the specimen was pervasively fractured with two sets of principal discontinuities, one oriented normal to the long axis (A, B and C) and one steeply inclined (D, E and F). The fracture filling minerals were predominantly chlorite and sericite with lesser amounts of epidote, calcite and other minerals. Except for the principal discontinuities, most fractures appeared to be well healed and simple falling-head tests showed that the secondary fractures made only a small contribution to the sample's permeability.

The specimen was tested in the large triaxial testing machine located at the University of California's Richmond Field Station (Becker et al., 1972). This equipment is shown in Figure 3 with the triaxial vessel being closed over the rock specimen. The test program took the form of a modified unconfined compressive strength test in which the axial loading, at a rate of 0.5 MPa/min, was interrupted and held constant at several stages. At each constant load stage permeability tests were performed by injecting water into (divergent flow) and withdrawing water from (convergent flow) a borehole drilled through the axis of the core. The test configuration is shown schematically in Figure 4. To ensure saturation of the sample and to keep air in solution, the triaxial vessel was filled with water at a pressure of 1,400kPa throughout the test. Linear variable differential transformers (LVDTs) and strain gauges were mounted on the specimen to measure overall axial deformation, circumferential strain at mid-height, deformation of the principal fractures, and strains in the rock matrix. This instrumentation is shown in Figure 1 and the locations are shown in Figure 2.

The macroscopic load-displacement response of the specimen is summarized in Figure 5, which plots axial strains against the applied axial stress. The data are from three LVDTs (Nos. 19, 20 and 21 on Figure 2) that measured the displacements between points 1.3m apart at the top and

bottom of the specimen. Axial strains at the center of the specimen were calculated from these readings and are shown as the solid curve in Figure 5. The curves exhibit the nonlinear form at low stress levels that is typical of many rocks, but also reflect a number of more complex features. Creep deformation occurred when the axial stress was held constant during permeability testing and the magnitude and rate of creep increased with increasing applied stress. The failure kinematics involved tilting of the top of the specimen and nonlinear shearing motions. During failure new fractures were developed in the previously intact portions of the specimen and pre-existing fractures were opened, as shown in Figure 2. A peak stress of 7.4 MPa was attained at 0.06% strain before the onset of generalized failure. This peak stress is much lower than the unconfined compressive strengths on the order of 100 MPa measured from small 5.2cm diameter samples of granitic rock containing healed fractures obtained from other locations in the Stripa mine. However, the tangent modulus of deformation measured prior to failure of the large specimen was 52.3 MPa which is within the range obtained from small diameter samples.

The conditions prevailing in an unconfined compression test are not generally representative of in situ conditions and a single laboratory test cannot account for all the complex factors such as geologic history, magnitudes and orientations of stresses, fracture geometry and boundary conditions that influence the behavior of a fractured rock mass. It would therefore be misleading to attribute the relative weakness of the large specimen solely to a "size-effect". However, the experimental data does illustrate the significant differences in measured properties that may be found when results from tests on very large specimens are compared with those obtained from samples of conventional size.

Results from the permeability tests are plotted on Figure 6 in the form of hydraulic conductivity versus applied axial stress. The data from the large specimen of Stripa rock are compared with results of previous laboratory and in situ tests in which normal stresses were applied to a single fracture. Although the permeability of the pervasively fractured specimen was much greater than that of the others, it initially exhibited

the characteristic reduction in permeability with increasing axial loading. As the axial stress was increased from zero to 5.56 MPa, the permeability decreased from 5.25cm/sec to 0.5cm/sec. However, prior to failure of the sample, there was a rapid increase in permeability to some 2.0cm/sec associated with shear dilatancy and fracture opening. This latter effect was not unexpected, but has not previously been observed in laboratory tests.

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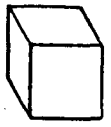
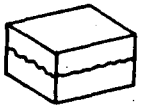
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TABLE 1

PRINCIPAL TYPES OF LARGE ROCK SPECIMENS AND LABORATORY TESTS
(after Watkins, 1981)

SPECIMEN		TYPICAL TEST APPLICATION
CUBIC		CONSTITUTIVE PROPERTIES IN THREE-DIMENSIONAL STRESS STATE (TRULY TRIAXIAL TESTS)
RECTANGULAR PRISM (JOINTED ROCK)		DIRECT SHEAR TESTS, SHEAR STRENGTH AND DEFORMABILITY OF JOINTS AND FRACTURES, FRACTURE CONDUCTIVITY
INTACT CYLINDER		TRIAXIAL TESTS, STRENGTH AND CONSTITUTIVE PROPERTIES OF ROCK MATRIX, THERMO-MECHANICAL PROPERTIES
INTACT CYLINDER WITH AXIAL BOREHOLE		MATRIX PERMEABILITY, HYDRAULIC FRACTURING, DOWNHOLE GEOPHYSICS
CYLINDER WITH DISCRETE FRACTURE NORMAL TO AXIS AND AXIAL BOREHOLE		FRACTURE CONDUCTIVITY, FRACTURE CLOSURE (NORMAL STRESS)
CYLINDER WITH DISCRETE FRACTURE INCLINE TO AXIS (WITH OR WITHOUT AXIAL BOREHOLE)		TRIAXIAL TESTS, CONSTITUTIVE PROPERTIES OF JOINTS, FRACTURE CONDUCTIVITY
PERVASIVELY FRACTURED CYLINDERS (WITH OR WITHOUT AXIAL BOREHOLE)		TRIAXIAL TESTS, ROCK MASS CONSTITUTIVE PROPERTIES, THERMOMECHANICAL BEHAVIOR, ROCK MASS PERMEABILITY, DOWN HOLE GEOPHYSICS

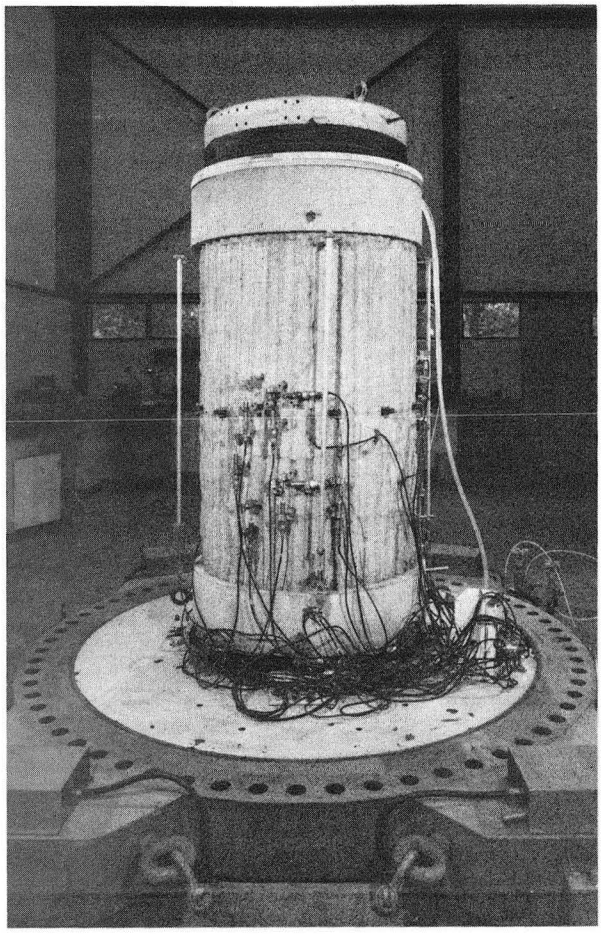


Figure 1
Instrumented Specimen
(CBB 802-2032)

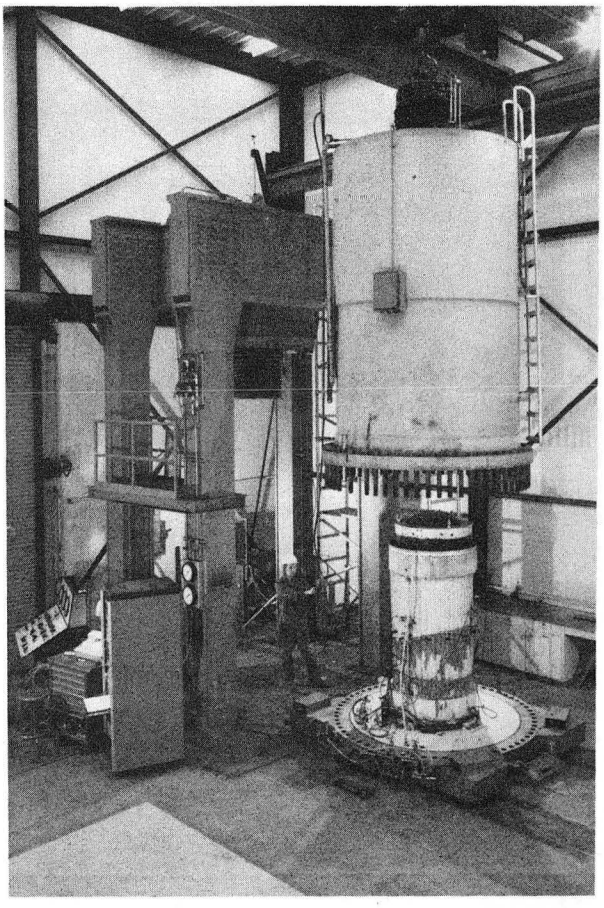


Figure 3
Large triaxial testing machine
(CBB-802-2028)

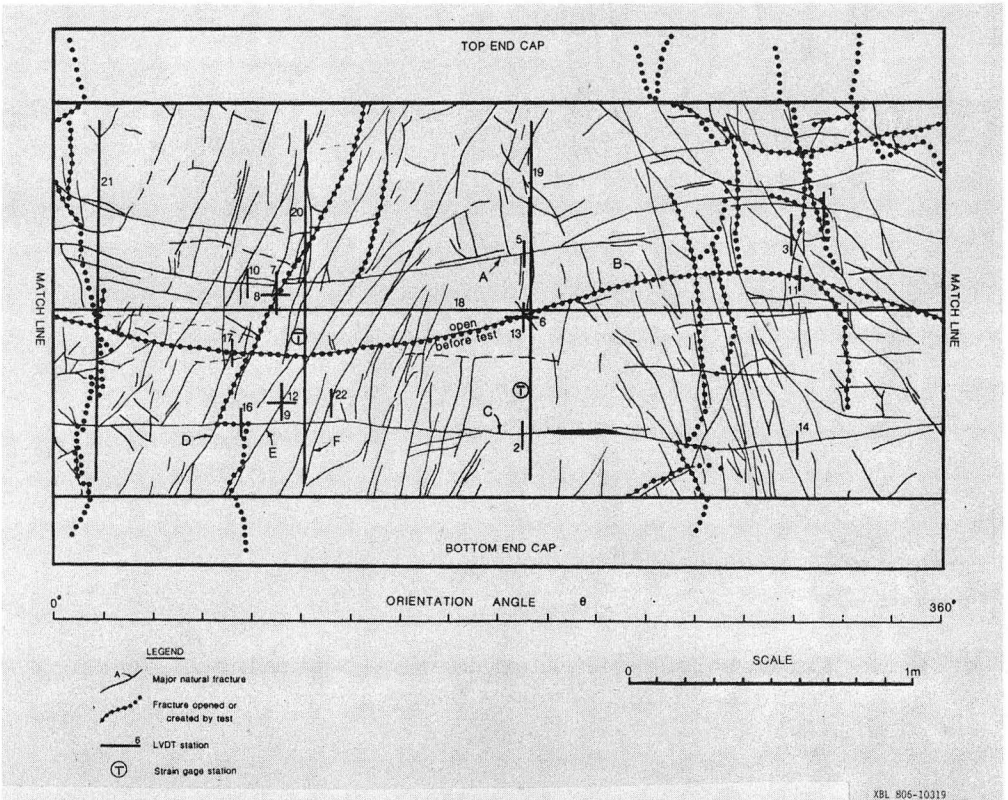


Figure 2. Fracture Map Showing Instrumentation

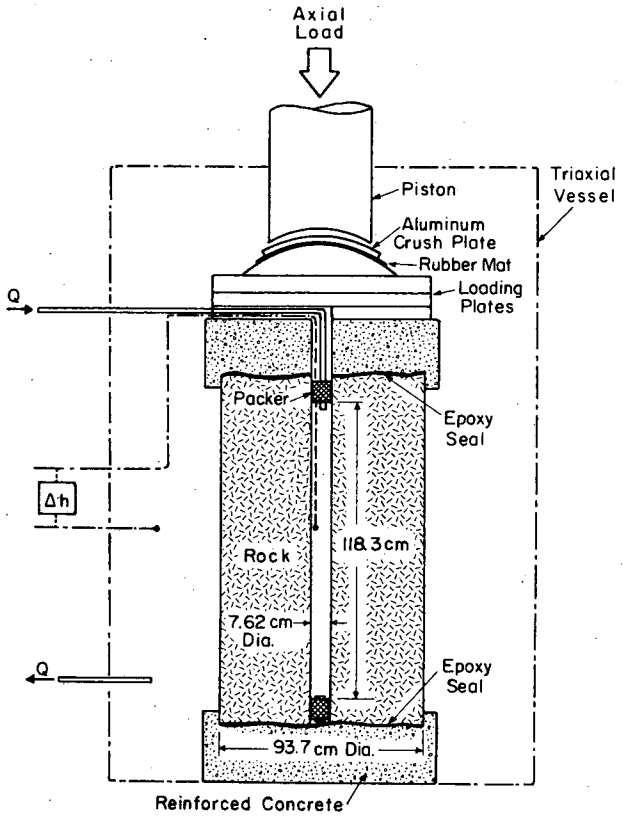


Figure 4. Schematic of test arrangement.

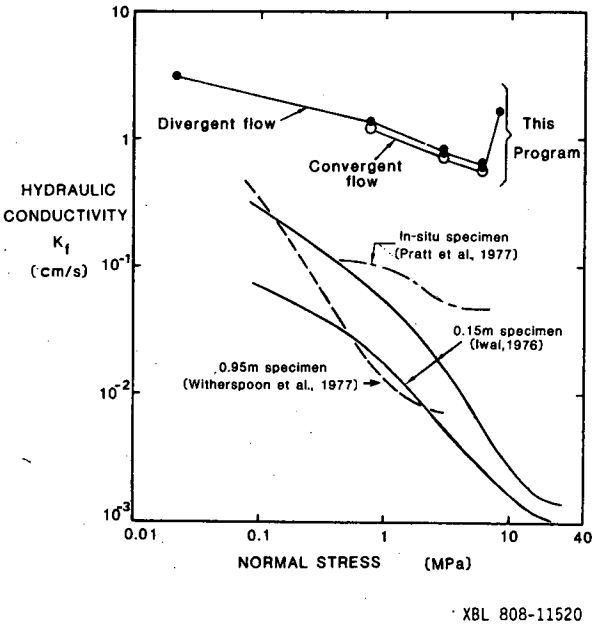


Figure 6. Changes in conductivity in response to applied stress.

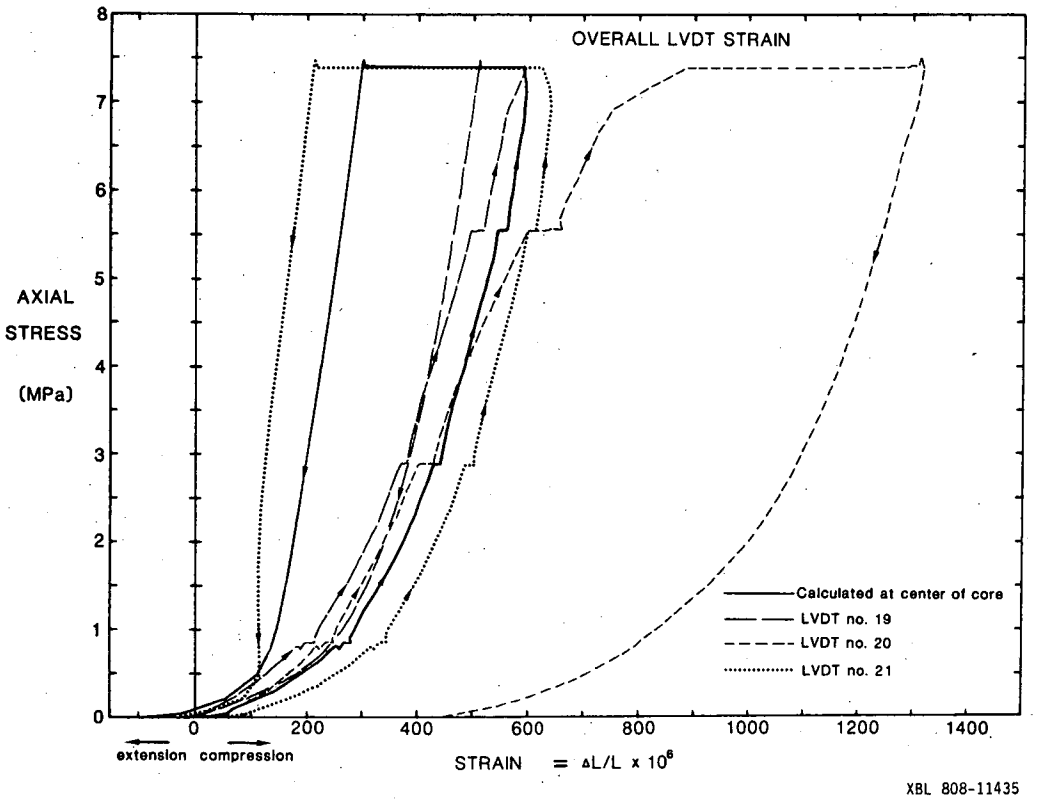


Figure 5. Axial stress-strain record with strains calculated for center of specimen.

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