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R. D. Allen
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April 1982

Prepared for the U.S. Department of Energy
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GEOTECHNICAL ISSUES AND GUIDELINES FOR STORAGE OF COMPRESSED AIR IN EXCAVATED HARD ROCK CAVERNS

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Pacific Northwest Laboratory
Richland, Washington 99352

FOREWORD

The Compressed Air Energy Storage Technology Program at the Pacific Northwest Laboratory (PNL) is sponsored by the Department of Energy, Division of Energy Storage Technology. The program scope includes a group of studies directed at developing a new energy storage technology to improve the cost and efficiency of electrical power utilization and reducing the dependence on petroleum fuels such as oil and natural gas.. The program has two major thrusts --- Reservoir Stability Criteria Studies and Second-Generation Concepts Studies. These have the following objectives:

- Reservoir Stability Criteria
  Develop stability criteria for long-term operation of underground reservoirs used for compressed air energy storage (CAES) in order to accelerate the commercialization of the concept.
- Second-Generation Concepts Studies
  Develop and assess advanced CAES concepts that require little or no supplementary firing by petroleum fuels in order to eliminate the dependence of CAES on petroleum fuels.

The ultimate objective of this program is to reduce the consumption of natural gas and oil used for peak-power generation plants by about 100,000,000 barrels per year. This could be accomplished by replacing conventional gas turbine peaking plants currently being used by utilities with CAES plants.

The following documents have been issued or are in process by PNL or by subcontractors to PNL, reporting the results of the work toward these objectives.

- Technical and Economic Feasibility Analysis of the No-Fuel Compressed Air Energy Storage Concept, D.K. Kreid, BNWL-2065, May 1976.

- FY-1977 Progress Report - Stability and Design Criteria Studies for Compressed Air Energy Storage Reservoirs, G.C. Smith, J.A. Stottlemyre, L.E. Wiles, W.V. Loscutoff and H.J. Pincus, PNL-2443 March 1978.

  FY-1977 Progress Report Compressed Air Energy Storage Advanced Systems Analysis, D.K. Kreid and M.A. McKinnon, PNL-2464, March 1978.

- Preliminary Stability Criteria for Compressed Air Energy Storage in Porous Media Reservoirs, J.A. Stottlemyre, PNL-2685, June 1978.

- Preliminary Long-Term Stability Criteria for Compressed Air Energy Storage Caverns in Salt Domes, R.L. Thoms and J.D. Martinez, PNL-2871, August 1978.

- Numerical Analysis of Temperature and Flow Effects in a Dry, One-Dimensional Aquifer Used for Compressed Air Energy Storage, G. C. Smith, L.E. Wiles, and W.V. Loscutoff, PNL-2546, February 1979.

- CAES and UPHS in Hard Rock Caverns: III. Preliminary Stability and Design Criteria for Compressed Air Energy Storage Caverns, P. F. Gnirk, Re/Spec Inc., Rapid City, SD, PNL-2916 (RSI-0079), draft February 1979.

- The Effects of Water on Compressed Air Energy Storage in Porous Rock Reservoirs, L.E. Wiles, PNL-2869, March 1979.

- Numerical Modeling of the Behavior of Caverns in Salt for Compressed Air Energy Storage (CAES), Vol 1-2, Serata Geomechanics, Inc., Berkeley, CA, May 1979.

- Pacific Northwest Laboratory Annual Report for 1978 to the DOE Division of Energy Storage Systems - Compressed Air Energy Storage Technology Program, W. V. Loscutoff, PNL-2935, June 1979.

- Incremental Cost Analysis of Advanced Concept CAES Systems, C.A. Knutsen, Knutsen Research Services, Bothell, WA, PNL-3118, September 1979.

- Numerical Analysis of Temperature and Flow Effects in a Dry, Two-Dimensional, Porous-Media Reservoir Used for Conipressed Air Energy Storage, L.E. Wiles, PNL-3047, October 1979.
- Potential Petrophysical and Chemical Property Alterations in a Compressed Air Energy Storage Porous Rock Reservoir, J.A. Stottlemyre, R.L. Erikson and R.P. Smith, PNL-2974, October 1979.
- The Economics of Compressed Air Energy Storage Employing Thermal Energy Storage, S. C. Schulte and R. W. Reilly, PNL-3191, November 1979.
- Structural Analysis of Porous Rock Reservoirs Subjected to Conditions of Compressed Air Energy Storage, J.R. Friley, PNL-3231, January 1980.
- CAES and UPHS in Hard Rock Caverns: IV. Preliminary Stability and Design Criteria for Underground Pumped Hydro Storage Caverns, P. F. Gnirk, Re/Spec Inc., Rapid City, SD, PNL-3262 (RSI-0110), draft, February 1980.
- Compressed Air Energy Storage Technology Program Annual Report for 1979, W. V. Loscutoff, Staff Members and Subcontractors of Pacific Northwest Laboratory, PNL-3395, June 1980.
- Porous Media Experience Applicable to Field Evaluation for Compressed Air Energy Storage, R. D. Allen and P. J. Gutknecht, PNL-3294, June 1980.
- Technical and Economic Assessment of Fluidized Bed Augmented Compressed Air Energy Storage System, A. J. Giramonti, R. D. Lessard, D. Merrick and M. J. Hobson, United Technologies Research Center, East Hartford, CT, PNL-3686 (UTRC R80-954490-20), Volumes 1-3, June 1980.
- An Experimental Study of the Response of the Galesville Sandstone to Simulated CAES Conditions, R. L. Erikson, J. A. Stottlemyre, and R. P. Smith, PNL-3399, July 1980.
- Numerical Modeling of Behavior of Caverns in Salt for Compressed Air Energy Storage with Elevated Temperatures, S. Serata and J. F. McNamara, Serata Geomechanics, Inc., Berkeley, CA, December 1980.

CAES and UPHS in Hard Rock Caverns: I. Geological and Geotechnical Aspects, D.S. Port-Keller and P.F. Gnirk, Re/Spec Inc., Rapid City, SD, PNL-2886 (RSI-0076), January 1981.

Technical and Economic Assessment of Fluidized Bed Augmented Compressed Air Energy Storage System - System Load Following Capability, R. D. Lessard, W. A. Blecher and D. Merrick, United Technologies Research Center, East Hartford, CT, PNL-3895 (UTRC R80-954490-29), January 1981.

Conceptual Design and Engineering Studies of Adiabatic CAES with Thermal Energy Storage, M. J. Hobson et al, Acres American, Inc., Columbia, MD, PNL-4115, March 1981.

• Water Coning in Porous Media Reservoirs for Compressed Air Energy Storage, L. E. Wiles and R. A. McCann, PNL-3470, June 1981.

• Annual Report for 1980 Compressed Air Energy Storage Technology Program, L. D. Kannberg, Staff Members and Subcontractors of Pacific Northwest Laboratory, PNL-3804, June 1981.

Line by Line Assessment of Principal Subsystems, Equipment Items and Components of Compressed Air Energy Storage (CAES) Systems, D. L. Chiang and C. K. Jee, ENTEC Research Associates, Inc., Rockville, MD, August 1981.

The Exploration and Characterization of an Aquifer CAES Site Near Pittsfield, Illinois, T. E. Jensen, Dames and Moore, Park Ridge, IL, Vol. I-II, September 1981.

Technology Assessment Report for the Soyland Power Cooperative, Inc. Compressed Air Energy Storage System (CAES), Environmental Science and Engineering, St. Louis, MO, September 1981.

Siting Selection Study for the Soyland Power Cooperative, Inc. Compressed Air Energy Storage System (CAES), Environmental Science and Engineering, St. Louis, MO, September 1981.

- Laboratory Testing of Hard Rock Specimens Subjected to Loadings and Environmental Conditions Appropriate For a Compressed Air Energy Storage Cavern, A. F. Fossum, Re/Spec Inc., Rapid City, SD, February 1982.

Geotechnical criteria for the design and stability of CAES caverns in hard rock formations are developed and summarized in this document. The intended audience includes technical people with the architect-engineering industry who would be involved in CAES construction. Thus, the writers assume normal competence in geotechnical engineering. The document is not an all encompassing text on either CAES or reservoir engineering, but rather a reference on CAES reservoir stability research work.

Three recent publications will also be of interest to the reader.

- Geotechnical Basis for Underground Energy Storage in Hard Rock, O. C. Farquhar, Electric Power Research Institute, EPRI EM-2260, Project 1199-11, Final Report, Palo Alto, CA, March 1982
- Preliminary Design Study of Underground Pumped Hydro and Compressed-Air Energy Storage in Hard Rock, Volumes 1 through 13: CAUPH Preliminary Licensing Documentation, EPRI EM-1589, Project 1081-1 and DOE/ET 5047-13, Final Report, April 1981 (Prepared by Acres American, Inc., Columbia, MD, Project Manager D.C. Willett)
- Factors Influencing the Design of a CAES Facility in Hard Rock, R. H. Curtis and D. C. Willett, Proceedings of the International Conference on Seasonal Thermal Energy Storage and Compressed Air Energy Storage, Vol. 2, pp. 501-510, October 19-21, 1981, Seattle, WA, Pacific Northwest Laboratory, Richland, WA.

ACKNOWLEDGMENTS

This document was compiled from recent literature dealing with the stability of excavated hard rock caverns. The authors are especially indebted to the staff of RE/SPEC Inc., of Rapid City, South Dakota, for their substantial contributions in the areas of numerical modeling and rock mechanics experimentation. Paul F. Gnirk, Arlo F. Fossum, Debra S. Port-Keller, Terje Brandshaug, G. D. Callahan and J. L. Ratigan authored these studies and provided considerable assistance with the literature search.

David C. Willett of Acres American, Inc., Buffalo, New York and Leif G. Eriksson of ERTEC Western, Inc., Long Beach, California, provided valuable suggestions during the final synthesis.

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SUMMARY

The primary objective of this report is to develop and present geotechnical criteria for the design and stability of CAES caverns in hard rock formations. These criteria involve geologic, hydrological, geochemical, geothermal, and in situ stress state characteristics of generic rock masses. Their relevance to CAES caverns, and the identification of required research areas, are identified throughout the text. The design criteria must be established from the viewpoint of 1) the type of CAES system, 2) the desired air volume and pressure, and 3) the thermal/rock mechanics/hydrologic constraints appropriate to the rock mass. These constraints must be used in determining of optimal cavern shape, cavern orientation, dimensions and spacing, and excavation methods.

Because of the high excavation costs associated with the larger volumes required by other storage schemes, a CAES reservoir in hard rock would probably be a constant pressure, water-compensated cavern. The daily temperature cycling and wall wetting occurring during the operation of a CAES reservoir, having never been encountered before in rock masses, lead to the following design concerns:

- cavern geometry and size
- cavern orientation
- thermal response
- low frequency fatigue, coupled with temperature cycling and wetting
- air penetration of rock mass
- hard rock properties at nonambient conditions
- residual strength of hard rock after failure
- mineralogical alteration of hard rock under CAES conditions.

The following conclusions regarding air storage in hard rock masses have been identified:

- Rocks must be competent (with high structural strength and stability), and capable of sustaining openings with minimal support and rock improvement measures. Candidate rock types include granite/granodiorite, quartzite, massive gneiss, dolomite and limestone.

- Rock masses must be characterized by overall hydraulic conductivities less than 10^{-8} m/sec for water.
- Long-term containment of stored air may not be possible in tightly folded, heavily fractured, jointed, or faulted rocks.
- Cavity geometry and orientation must be selected for minimal support requirements. Important geologic parameters include extent of fracturing, nature of joint surfaces, permeability in zones of weakness, and hydrologic conditions.
- Storage cavern design and construction must minimize slaking, spalling, and loss of ground water above the cavern.
- Cyclic temperature and humidity variations must not significantly decrease rock strength by fatigue.
- Geologic formations with high horizontal in situ stresses are unfavorable. Maximum horizontal stress should not exceed vertical stress by more than a factor of 1.5.
- The construction cost of CAES caverns can be seriously impacted by conditions such as degree of jointing and faulting and incompetence of the overburden. Access shafts for CAES caverns should not be sunk in areas with more than 50 m of incompetent, water-bearing overburden.

Additional guidelines that should be considered when constructing a CAES reservoir in hard rock include:

- Hydrostatic pressure within the host rock must balance the pressure of stored air (and the equal pressure of the water-compensating column).
- Surface water must be available for pressure compensation.
- The "champagne effect", rapid evolution of air in the water column connecting the cavern to the surface lake, must be considered in the design of CAES plants with compensated caverns.

- Unconfined compressive strength must exceed 25 MPa over the cycling life.
- The nearest dissimilar geologic formation contact should not be closer than 100 m.
- Areas of active volcanism, faulting, seismic activity, excessive subsurface solution and subsidence are to be avoided.
- Long axes of caverns should be oriented with respect to structural discontinuities and in situ stress fields to maximize stability and minimize construction costs.
- The most likely cavern depth is 750 to 850 m due to operating requirements.
- Air loss should not exceed 1% during the storage period.
- Operating pressures, essentially constant, will fluctuate within a very narrow range with differences attributable to variations in the effective height of the water-compensating column. The most likely design range for operating pressures is 7.35 to 8.33 MPa. Maximum charging pressure will be 12.0 kPa/m of depth.
- Compressed air will enter the cavern at 30 to 80°C.
- Compensating water temperature may fluctuate between 0 and 30°C.
- Cavern depressurization should be gradual not exceeding 1 MPa per hour.

In conclusion, this literature survey and analysis strongly suggests that the chief geotechnical issues for the development and operation of CAES caverns in hard rock are impermeability for containment, stability for sound openings, and hydrostatic balance.

CONTENTS

FIGURES

TABLES

GEOTECHNICAL ISSUES AND GUIDELINES FOR STORAGE OF COMPRESSED AIR IN EXCAVATED HARD ROCK CAVERNS

1.0 INTRODUCTION

Compressed air energy storage (CAES) is a method for storing off-peak surplus energy from electric power plants, then readily recovering and using the stored energy to generate electrical power during peak demand periods. The CAES method requires underground caverns to store the air compressed during off-peak demand periods. Cavern storage may be either:

- compensated - Nearly constant air pressure with varying volume (wet system with hydraulic compensation), or
- uncompensated - Constant air volume with varying pressure (dry system).

Over the 30-year proposed operational lifetime of a CAES system, the rock mass that contains the caverns and inlet/outlet shafts will be subjected to some 10,000 loading cycles. Although the internal pressure in the storage caverns of a compensated system is nearly constant, the rock is subjected to a daily temperature cycle caused by alternating contact with hot air (compression cycle) and water at "ambient" temperature (generation cycle). These temperature cycles will induce cyclic thermal stresses in the rock. Under these conditions, the selection of the rock mass and the quantification of its material properties and geotechnical characteristics are important factors in designing the caverns to minimize the risk of either structural or air/water leakage instabilities (Port-Keller and Gnirk 1981).

Rocks suitable for the excavation of underground CAES caverns must be competent and capable of sustaining openings supported by the pillars and walls left during mining. Hard rock formations offer one potential geologic medium. Hard rock is defined as rock with relatively great strength and resistance to large ductile and creep deformation (Gnirk and Port-Keller 1978). Candidate hard rock masses also must have the following key characteristics: homogeneous lithology, low porosity and permeability, widely spaced joints and fractures, no significant faulting, and mineability.

Underground hard rock storage has several precedents. The first underground powerhouse was built in 1908 at Mockfjard in northern Sweden with an installed capacity of 12 MW. In 1910 a 50-MW underground plant was built at Porjus in Lapland. Utilization of underground openings in hard rock for power facilities began in Norway in 1916. In the late 1930s, unlined rock caverns for storing liquid hydrocarbons were developed in Sweden. Although several hard rock storage caverns have long been used in the United States for the storage of petroleum products, interest in the use of excavated hard rock chambers for CAES is relatively recent. In this respect feasibility studies have been or are being conducted in California, Kansas, Illinois, Maryland, and the New England states (Weinstein et al 1978).

Also in Sweden many reservoirs of oil, gas, and air have been contained in excavated hard rock chambers (see Appendix A).

Currently, no CAES facility has yet been constructed in hard rock. However, Potomac Electric Power Company (PEPCO) developed preliminary designs and costs for a CAES hard rock cavern facility to be excavated at a suitable site in or near the PEPCO power system in Maryland. The plant design study was completed in 1980 (see Appendix B). Soyland Power Cooperative, Inc., of Decatur, Illinois, has designed a 220-MW CAES power plant that will store air within caverns excavated from Cambrian dolomite at a depth of about 700 m. 'The caverns are to be water-compensated.

This document describes the geotechnical issues affecting CAES in excavated hard rock caverns. Most issues relate to geologic environments, hydrology, rock mechanical properties, cavern excavation, and predicted operating conditions.

Section 2 treats the geologic issues and emphasizes candidate rock types, structural characteristics, hydrological requirements, and thermal/ mechanical rock properties. Results of tests conducted on candidate rocks are also included.

Section 3 embraces all main aspects of cavern stability. Types of instability that could affect compressed air storage are explained and various mechanisms are identified. Numerical modeling is applied to cavern design, post-excavation stability, thermal penetration of walls, and air leakage. Cavern excavation experience is included in subsection 3.4.

Section 4 addresses site qualification. Procedures for initial screening, rock characterization, drilling and in situ testing, geophysical methods, joint surveys and permeability measurement are suggested.

In Section 5 operational issues are identified. These include injection parameters, potential impacts within the host rock, reservoir monitoring and champagne effect.

Design and stability criteria are tabulated in Section 6. Although some are assigned numbers, others are handled with qualitative statements. Most criteria are concerned with the prevention of instabilities: local, general, and air leakage.

2.0 GEOTECHNICAL ISSUES

A wide range of geologic factors must be considered in choosing the site for a CAES hard rock cavern. Among these are candidate rock types, structural characteristics, hydrology, and thermal/mechanical rock properties. Each is discussed in the following subsections.

Rocks must be sufficiently competent to sustain caverns without supports other than the pillars and walls left after excavation (Chang et al 1980). Cavity geometry must be selected to minimize support requirements. Heavily fractured, jointed, or faulted rocks will be eliminated from consideration because they may not sustain underground reservoirs and may not contain compressed air, especially over the longer term. The degree of jointing and faulting and competence of the overburden must be limited so as not to seriously increase the construction cost of CAES caverns.

Candidate rock types include granite/granodiorite/diorite, gabbro, quartzite, massive gneiss, dolomite, and limestone. High structural strength, adequate depth and volumetric extent, and absence of high in situ lateral stress are important. The candidate site will be about 750 to 850 m below the water level of the compensating surface lake to ensure adequate hydrostatic head. Maintenance of capillary water within open pores and fractures is also a necessary condition during all stages of construction and operation. Rock masses should be characterized by overall hydraulic conductivities less than 10^{-8} m/sec for water to assure containment of air.

Cyclic exposures of reservoir walls to temperature, pressure, oxygen and water must not significantly decrease rock strength by fatigue or geochemical reactions. Slaking and spalling must not occur in sufficient degree to allow increased air loss or structural instability.

If more than 50 m of incompetent water-bearing overburden overlies an otherwise attractive site, it may not be economically feasible to construct access and water compensation shafts.

Impermeability for air containment (and compensating water containment) and stability for maintenance of sound caverns and shafts are the chief geotechnical issues (Milne et al 1977; Walia and McCreath 1977; Duffant 1977).

2.1 CANDIDATE ROCK TYPES

Intrusive igneous rocks, massive chemically precipitated sedimentary rocks and relatively massive nonfoliated metamorphic rocks are the most suitable hosts for excavation of CAES caverns. Favorable CAES sites within intrusive igneous and metamorphic rocks can be found in large areas of the eastern, north-central, and western United States (Figure 1). Siting possibilities for limestones and dolomites are confined principally to the midwestern region including Illinois, Indiana, Iowa, Kentucky, Minnesota, Missouri, Ohio, and Wisconsin.

2.1.1 Igneous Rocks

Igeous rocks originated by crystallization of minerals and/or solidification of glasses from magma. These rocks are classified as either intrusive or extrusive, depending upon where they hardened in relation to the earth's crust.

2.1.1.1 Intrusive

Intrusive igneous rocks are those which hardened beneath the overlying unconsolidated sediments and within crustal rock. These rocks, which crystallized as masses (plutons) in various sizes and shapes, are generally "phaneritic", meaning grain diameters are greater than 1 mm and easily seen with the unaided eye. Figure 2 presents a typical igneous rock mineralogical classification (Port-Keller and Gnirk 1981).

This study has concentrated on rocks of the granite-granodiorite, diorite, gabbro, and peridotite groups. The other groups, although they may be locally favorable for CAES, occur with much less frequency and in limited extent compared to the rock types mentioned (Jackson 1970; Travis 1955; Dietrich and Skinner 1979).

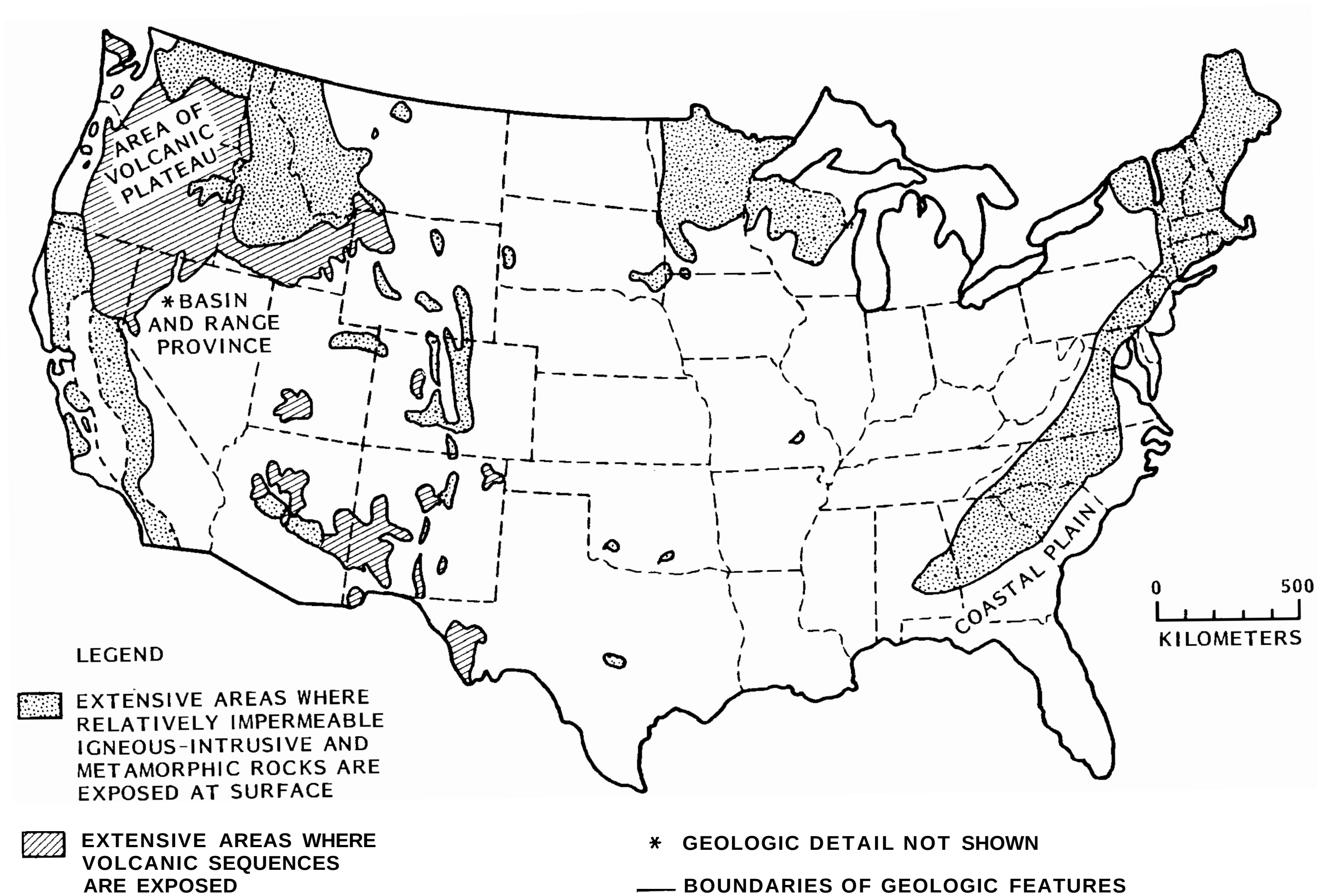

FIGURE 1. Distribution of Igneous and Metamorphic Rock in the United States (Aamodt et al 1975)

<table>
<tr><th rowspan="2">ESSENTIAL MINERALS</th><th rowspan="2">PREDOMINATELY POTASSIUM FELDSPAR</th><th rowspan="2">POTASSIUM AND PLAGIOCLASE FELDSPAR</th><th colspan="3">PREDOMINATELY PLAGIOCLASE FELDSPAR</th><th rowspan="2">IRON AND MAGNESIUM MINERALS</th></tr>
<tr><th>SODIUM PLAGIOCLASE</th><th>INTERMEDIATE</th><th>CALCIUM PLAG.</th></tr>
<tr><td>QUARTZ >10%</td><td colspan="3">GRANITE - GRANODIORITE
(RHYOLITE - DACITE)
GROUP</td><td rowspan="2">DIORITE
(ANDESITE)
GROUP</td><td colspan="2"></td></tr>
<tr><td>QUARTZ <10%</td><td colspan="3">SYENITE
(TRACHYTE)
GROUP</td><td>GABBRO
(BASALT)
GROUP</td><td>PERIDOTITE
GROUP</td></tr>
<tr><td>FELDSPATHOIDS REPLACE QUARTZ</td><td colspan="4">FELDSPATHOIDAL SYENITE
(FELDSPATHOIDAL TRACHYTE)
GROUP</td><td>FELDSPATHOIDAL GABBRO
(FELDSPATHOIDAL BASALT GROUP)</td><td></td></tr>
</table>

NOTE: PARENTHESES INDICATE EXTRUSIVE EQUIVALENTS

FIGURE 2. Igneous Rock Classification (Port-Keller and Gnirk 1981)

Magma may be emplaced in the surrounding rock (country or host rock) forcefully or gently. The depth, nature of emplacement, and geochemistry influence the kinds of rock types and associated structures (folds and faults) that may be formed. These factors also determine the chemical alteration of country (host) rock and textural features of the solidified pluton (Port-Keller and Gnirk 1981).

After emplacement, other external forces may impose structural features on the pluton such as folds, faults, and joints. In addition, country rock may erode, exposing the pluton itself to weathering, erosion, and other mechanisms of joint formation (Port-Keller and Gnirk 1981).

Igneous intrusive rocks are generally considered to be the most likely geologic terrane for CAES cavern sites. So long as joints, faults, or folds have not seriously impaired the integrity of the rock mass, such rocks may be considered favorable for several reasons:

- They may be found in masses sufficiently large to accommodate the CAES systems.

- The intact rock is often homogeneous and nearly isotropic on a coarse scale.
- The petrography., particularly the interlocking crystalline texture, imparts generally high strength and favorable elastic, thermal, and weathering properties to the rock.
- Porosity and primary permeability are generally very low.

2.1.1.2 Extrusive

Extrusive igneous rocks are formed after ejection or outpouring at or very near the earth's surface in a variety of ways. They may be formed from lava gently extruded onto the earth's surface; the lava may solidify in layers as much as several hundred feet thick. Extrusive rock may also be formed from violent volcanic eruption. Rock textures (specifically numbers, sizes, and shapes of cavities) depend on the type of lava, the nature of emplacement, and the method or rate of cooling, as in water or air. Such rocks may vary widely and be highly permeable and not very competent, or they may be compact, impermeable and very competent.

Extrusive features associated with high permeability and incompetence include scoriaceous, vesicular, brecciated, and shrinkage-cracked structures. Scoriaceous describes an extremely vesicular rock in which many vesicles open at the surface. A vesicle is an ellipsoidal, cylindrical, or spherical opening formed in molten rock by the expansion of gas escaping from solution. Breccia zones are cavities and broken rock often occurring between lava flows. Shrinkage cracks are caused either by cooling of the outer lava surface or by cooling from beneath. In addition, after cooling, other structural features such as folds, faults, and joints may be superimposed.

Extrusive igneous rocks are generally considered to be less favorable for CAES development than intrusive rocks due to their inherent structural discontinuities. They are often found in masses of limited areal extent and layers of limited thickness. The rocks may be highly vesicular, with high porosity and permeability and generally low strength. Intact core specimens, however, may exhibit high strength. The rocks tend to occur near the surface.

The most favorable igneous extrusive rock is probably one formed by massive, nonviolent flow of lava upon the earth's surface. If such a flow occurs in sufficient mass to be a cavern host, other characteristics cited as favorable in igneous intrusives would also generally apply (Port-Keller and Gnirk 1981). In this respect, massive basalt flows or welded tuffs of sufficient density might be favorably considered as likely host rocks for CAES site development.

2.1.2 Metamorphic Rocks

Metamorphic rocks are igneous, sedimentary, or previously metamorphosed rocks that have been altered by temperature, pressure, and/or solutions. Metamorphic petrology is extremely complicated; however, the primary classification can be based on the presence or absence of laminated structures in the rock resulting from segregation of different minerals into layers. This parallel or nearly parallel structure, along which the rock tends to separate into flakes or thin slabs, is called foliation. Foliated metamorphic rocks include slates, phyllites, schists, and some gneisses. Nonfoliated metamorphic rocks include quartzites and marbles. Figure 3 presents a typical metamorphic rock classification (Port-Keller and Gnirk 1981).

Structure in metamorphic rocks is often complex, involving numerous faults and folds. In addition to foliation, other small-scale structures may include rock cleavage or schistosity. Rock cleavage[a] is the tendency to split along parallel surfaces of secondary origin whereas

[a] Foliation and rock cleavage are distinguished as follows: foliation is a general condition in which a rock exhibits some parallelism in mineral segregation and some tendency to split along these surfaces. Rock cleavage is foliation in which these surfaces are parallel, the resulting rock separations are parallel, and the planes are not indigenous within the original rock.

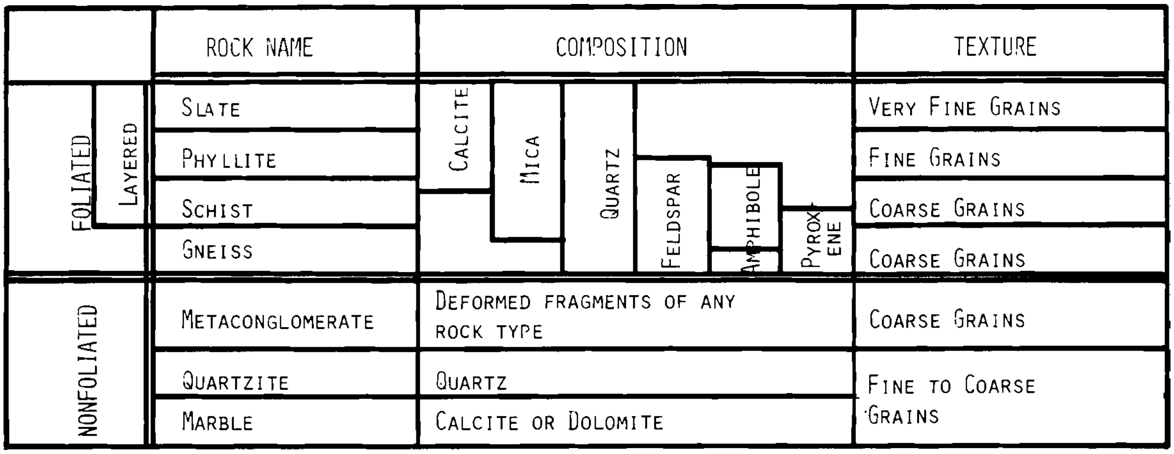

FIGURE 3. Metamorphic Rock Classification (Port-Keller and Gnirk 1981)

schistosity applies to that variety of rock cleavage found in rocks sufficiently recrystallized and laminated to be classified as schist or gneiss. As such rocks are often less competent, joints, partings, fissures, and fractures may be closed at shallower depths in foliated rocks than in igneous intrusive rocks (Brown 1975). However, joints in marbles and quartzites may have wide apertures at depth, similar to those found in igneous intrusive rocks (Port-Keller and Gnirk 1981). Particular gneisses with coarse irregular banding may not be associated with discontinuities or planes of weakness. These massive rocks may be quite suitable for CAES cavern construction.

Nonfoliated rocks, such as quartzites and marbles, when they occur in sufficiently large masses, may be as suitable for caverns as igneous intrusives. Such rocks are generally homogeneous and isotropic and have adequate strength characteristics. This is particularly characteristic of quartzites. Marble, however, may occur less frequently in large masses, have somewhat lower strength, and be subject to undesirable weathering and solution effects because it consists primarily of calcium carbonate. The desirability of high quality marble for other commercial purposes will tend to eliminate it as a potential host rock (Port-Keller and Gnirk 1981).

Major difficulties in CAES cavern stability in foliated rocks would probably arise from either foliation or rock cleavage. Foliation and cleavage planes can cause the strength, elasticity, and thermal properties of the rock to be both anisotropic and highly variable in particular directions. In addition, weathering of such rocks may be accelerated along planes of separation. The presence of easily altered metamorphic minerals such as chlorite and talc hastens chemical weathering (Port-Keller and Gnirk 1981).

2.1.3 Sedimentary Rocks

Sedimentary rocks are formed by weathering, transportation, and consolidation of fragments of other rocks and minerals of many kinds (clastic rocks), by precipitation of constituents from solution (chemical rocks), or by chemical secretion by organisms (also classed as chemical rocks). Clastic sedimentary rocks include shales, siltstones, numerous sandstone types, breccias, and conglomerates. Chemical sedimentary rocks include numerous limestone types, dolomites, gypsum, and rock salt. Figure 4 presents a typical sedimentary rock classification (Port-Keller and Gnirk 1981).

Most sedimentary rocks are bedded. Many are tilted, folded, or faulted after deposition. Sedimentary rocks can generally be considered much less favorable than igneous rocks for CAES caverns for several reasons:

- They are usually bedded and may not occur in sufficiently thick beds to be cavern hosts. Also, they may be interbedded with other, less competent rock types. Indeed, the bedding planes themselves may be considered planes of weakness.
- Strength is generally considerably lower than for igneous rocks (except for some dolomites and limestones).
- Porosity and permeability are much greater (particularly for clastic rocks) than for igneous rocks and may present insurmountable air and water leakage problems.

	Rock Name	Composition	Texture
Clastic	Conglomerate	Fragments of any rock type	Coarse rounded grains 2 mm)
	Breccia		Coarse angular grains 2 mm)
	Various Sandstone Types	Quartz, various amounts of feldspar, rock fragments	Medium grains (1/16 to 2 mm)
	Siltstone	Quartz and clay minerals	Fine grains (1/256 to 1/16)
	Shale	Quartz and clay minerals	Very fine grains (<1/256 mm)
Chemical	Various Limestone Types	Calcite ($CaCO_3$)	Micro to coarse crystals, fossil and fossil fragments
	Dolomite	Dolomite $CaMg(CO_3)_2$	Similar to limestone
	Chert	Chalcedony (SiO_2)	Cryptocrystalline, dense
	Gypsum	Gypsum ($CaSO_4 \cdot 2H_2O$)	Fine to coarse crystals
	Rock Salt	Halite (NaCl)	Fine to coarse crystals

FIGURE 4. Sedimentary Rock Classification (Port-Keller and Gnirk 1981)

- Weathering and solution characteristics are much less favorable than corresponding properties for igneous rocks.

Shales and siltstones disintegrate rapidly under exposure to cyclic moisture and temperature conditions (Mailhe et al 1977). Rock salt and gypsum are very water soluble. Limestone and dolomite are also soluble, and their carbonate terranes may form caves and numerous other solution features under the effects of ground water. Such solution features may reduce rock mass stability and greatly increase secondary permeability (Port-Keller and Gnirk 1981).

For the above reasons, this study has concentrated on competent limestones, dolomites, and a few sandstones as potential CAES hosts. Most other clastics, gypsum, and rock salt have been disregarded as inappropriate candidates for hard rock cavern development (Port-Keller and Gnirk 1981).

2.2 GEOLOGICAL CHARACTERISTICS

A rock that is stable or competent has the inherent physical and geologic characteristics to sustain openings (typically with spans of several meters) without any structural support except the pillars and walls that remain after mining. Competent rocks may be either massive or bedded. An ideal massive rock is assumed to be elastically perfect, isotropic, homogeneous and possessing an invariant strength (Obert et al 1960).

Any subsurface rock is in a state of stress because of the weight of the overlying rock and stresses created during tectonic or orogenic deformation. During excavation, additional stresses are produced in the rock mass surrounding the opening. These will cause the rock to fail if the stresses exceed the in situ strength.

Regardless of the stress situation in a rock mass, it is essential for stability that zones of weakness be avoided. These weak zones may be caused by jointing, faulting, fracturing, weathering, and mineralization (Selmer-Olsen and Broch 1977).

A meaningful stability analysis depends upon an adequate combination of geologic exploration and field testing, rock mechanics property testing, and numerical modeling.

Important physical and geologic parameters that establish overall rock mass quality include the following:

- extent of fracture (from remote sensing analysis and core examination) number of joint sets
- nature of joint surfaces (rough or smooth; clay-filled or unaltered) permeability and zones of weakness
- rock strength and in situ stress conditions
- hydrologic conditions.

Rock formations that have experienced severe tectonic deformation may be heavily folded, fractured, and faulted which can result in high permeabilities and low mechanical strength. The geological complexity of areas that have had a high degree of tectonic activity render subsurface conditions difficult and costly to evaluate. Because conditions can change very rapidly beyond the borehole, the structure and stratigraphy cannot be determined in detail until excavation has begun. Unforeseen vertical and lateral changes in rock strength and permeability can cause construction and containment problems (Wesslén et al 1977). Because very little, if any, accurate subsurface geologic information is likely to exist in these areas of complex geology, they should be avoided.

Rock masses may contain both primary and secondary structural features. Primary structure constitutes those features that were formed at the time of rock origin. Secondary structural features are those that were superimposed on the rock mass and its primary features after rock origin. It should be emphasized that neither primary nor secondary features are found exclusively in one rock type. Therefore, such structures as faults, folds, or joints, which possibly could be significant in the design and stability of CAES caverns, may be found in any rock mass. These features are discussed in more detail in the following subsections.

2.2.1 Relevant Structural Features

2.2.1.1 Bedding Planes

The most important primary structural feature is bedding, or the planes that separate the same or different lithologies in a rock mass. They are formed during deposition of sediment, which then consolidates to form sedimentary rock. Such bedding is evident in most sedimentary rocks, and remnant bedding may remain in metamorphic rocks. Bedding thicknesses range from only several millimeters (laminae) to hundreds of meters. Bedding planes are significant to underground structures in that they are often planes of weakness in a rock mass along which failure may occur. In addition, bedding planes may act as hydraulic conduits, and, when deformed, may increase or decrease the hydraulic conductivity of a rock mass. Figure 5 illustrates how undeformed sedimentary beds may overlie igneous rock.

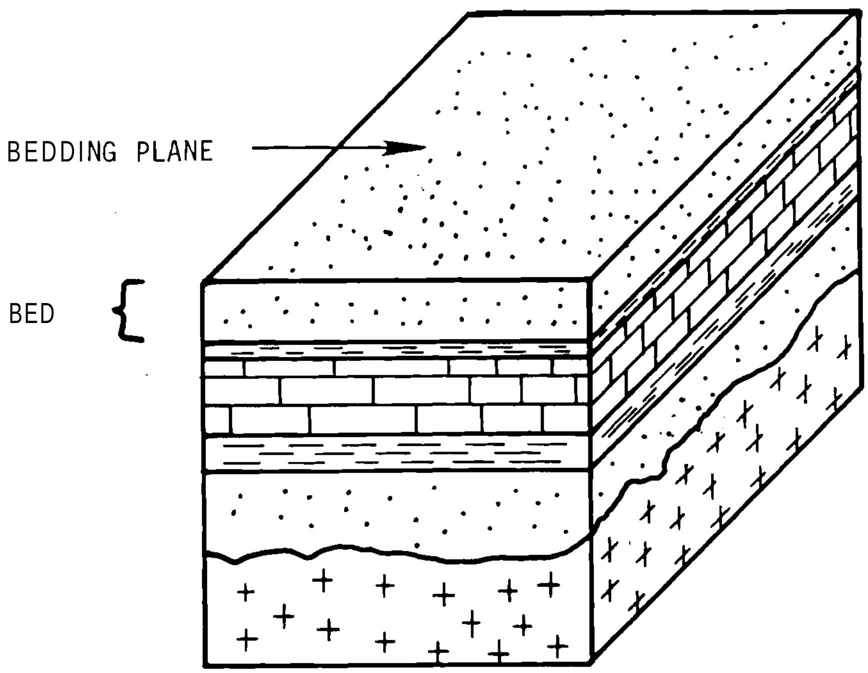

FIGURE 5. Sequence of Undeformed Sedimentary Rock Beds and Bedding Planes Overlying an Igneous Rock Mass (Port-Keller and Gnirk 1981)

2.2.1.2 Folding and Faulting

Major secondary structural features include folds, faults, and joints. Folds are warps or flexures in the earth's crust, and may occur on a broad regional scale of many kilometers or on a local scale of several centimeters. Folds may occur singularly, or in fold systems which may or may not be associated with fault development. Figure 6 shows a single fold in sedimentary and igneous rocks.

Folding in a rock mass is potentially significant in underground construction because it may indicate a region of high horizontal in situ stress. Folding, especially when "tight" and accompanied by faulting may also tend to undermine the integrity of a rock mass by producing zones of weakness. Finally, folding may produce extremely complex subsurface structure in which mapping of the contacts between different

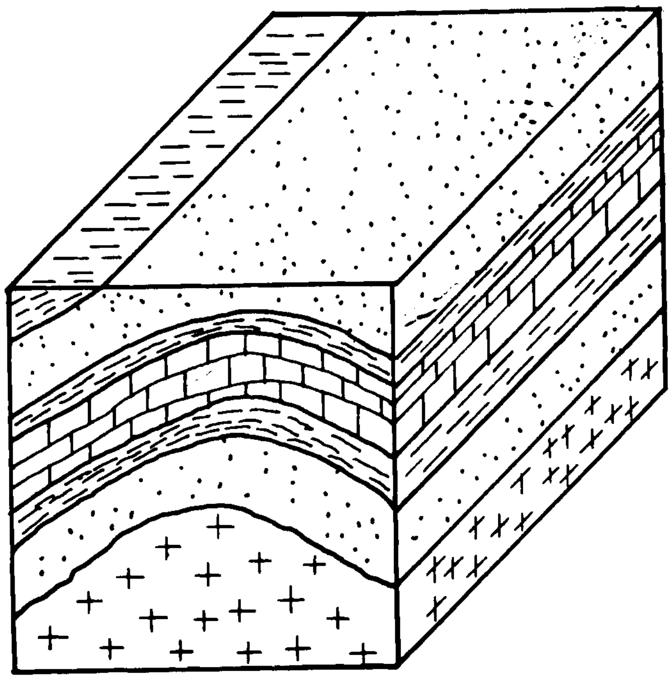

FIGURE 6. Simple Fold in Sedimentary Rock Formation Overlying an Igneous Rock Mass (Port-Keller and Gnirk 1981)

rock types is difficult. Folding may be so intense and complicated that a competent rock mass may be "split" into several sections too small to be of interest as a CAES cavern host.

Faults are fractures or fracture zones in the earth's crust along which there has been displacement of the sides relative to one another and parallel to the fracture. As in the case of folds, displacement in faults may be many kilometers or only several centimeters, and faults may occur singularly or in fault systems. Figure 7 shows a single "normal" fault displacing sedimentary and igneous rock. Faults may or may not be associated with other structural events such as folding or emplacement of igneous plutons.

The area between fault blocks (the masses of rock that have moved relative to one another) may be filled with some sort of unconsolidated earthy material (fault gouge), such as clay or finely ground rock material

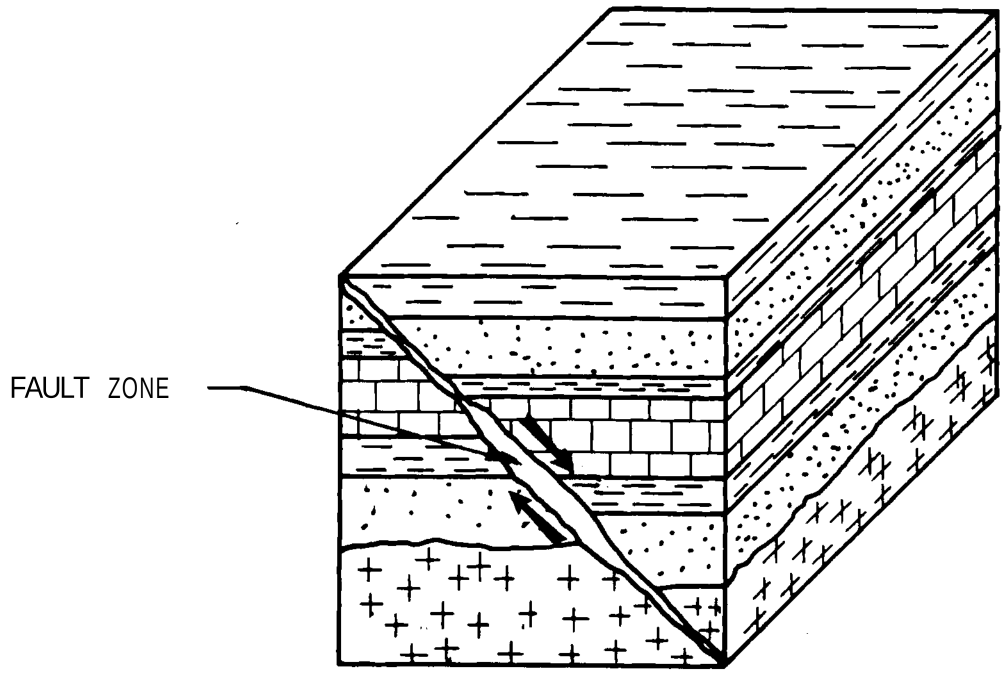

FIGURE 7. Normal Fault Displacing Igneous and Sedimentary Rock Formations (Port-Keller and Gnrik 1981)

formed during faulting. Fault zones may also be subsequently filled by other minerals such as calcite or metal sulfides. The so called fault "plane" is rarely a true smooth plane, but is more likely a zone of varying curvature and thickness.

Faults and fault zones are potential areas of weakness in rock masses. Because of such zones and the likelihood of accelerated weathering and water movements within them, rock masses are subject to failure along such planes. These zones also often increase the hydraulic conductivity of rock masses. Finally, faults may indicate an active seismic situation.

2.2.1.3 Joints

Joints are also classified as fractures in the earth's crust, but unlike faults, no appreciable displacement has' occurred in their formation. Joints are potentially the most significant geologic structural feature related to the stability of underground excavations, for three reasons:

They are a much more localized feature than are either faults or folds, and occur much more frequently.

- They provide planes of weakness along which failure is most likely to occur in a rock mass.

They act as hydraulic conduits, which increases the permeability of a rock mass.

The joint characteristics generally of most interest in underground construction are aperture or opening width, orientation, continuity, and frequency of occurrence. The length of a joint plane may range from about one meter to several hundred meters, and the spacing may vary from a few centimeters to tens of meters. Very closely spaced joints are often termed fracture cleavage. Generally, aperture width and frequency of joints decrease with depth in a rock mass. Joints in more competent rocks, such as granites and quartzites, usually have greater aperture width at depth than do joints in sandstones or shales.

Most joints are relatively plane surfaces, but some are curved. Most originate as tight fractures, but often they may be "opened" by weathering or tectonic forces. Joints may also be filled by such minerals as clay or calcite. The degrees of weathering and filling are related to rock type, hydrological regime and nearness to surface.

Joints may occur at any attitude, i.e., vertical, horizontal, or inclined at any angle. Most often, a large number of joints are parallel. A joint set is a group of more or less parallel joints, whereas a joint system consists of two or more joint sets with a characteristic mutual orientation.

Several kinds of primary joints are common in hard rock, particularly igneous rock. Columnar and transverse joints are caused by tensile stresses that develop during the cooling and crystallization of an igneous rock mass. They most often occur within the igneous body itself, but may also appear in the adjacent country rock (Lahee 1961) as shown in Figure 8. The columnar joints create hexagonal blocks (typical of basalt)

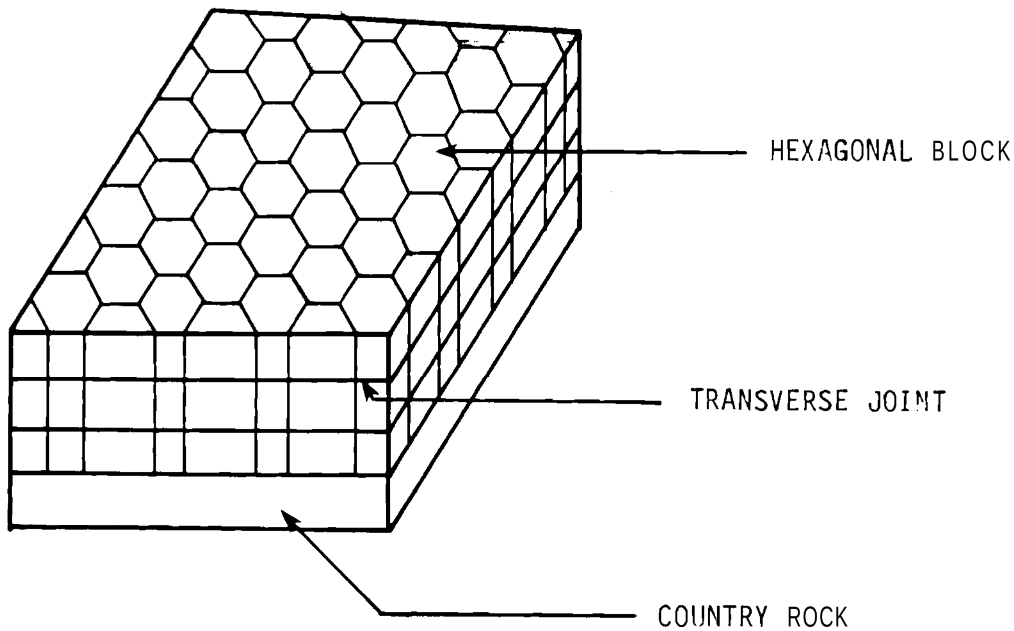

FIGURE 8. Columnar and Transverse Joints in an Igneous Rock Mass (Port-Keller and Gnirk 1981)

whereas the transverse joints are parallel to the country rock/igneous rock contact.

Other primary joints include cross joints, tension joints, and marginal fissures. Figure 9 depicts such joints in an igneous rock mass. Note that:

- Cross joints occur roughly at right angles to igneous flow lines (tension origin).
- Tension joints are cross joints at depth in a rock mass where flow lines may not be evident.
- Marginal fissures occur in the steep border regions of the intrusions and dip into the igneous body at angles between 20° and 45°.

Both cross joints and marginal fissures may transect the intrusive-country rock contact.

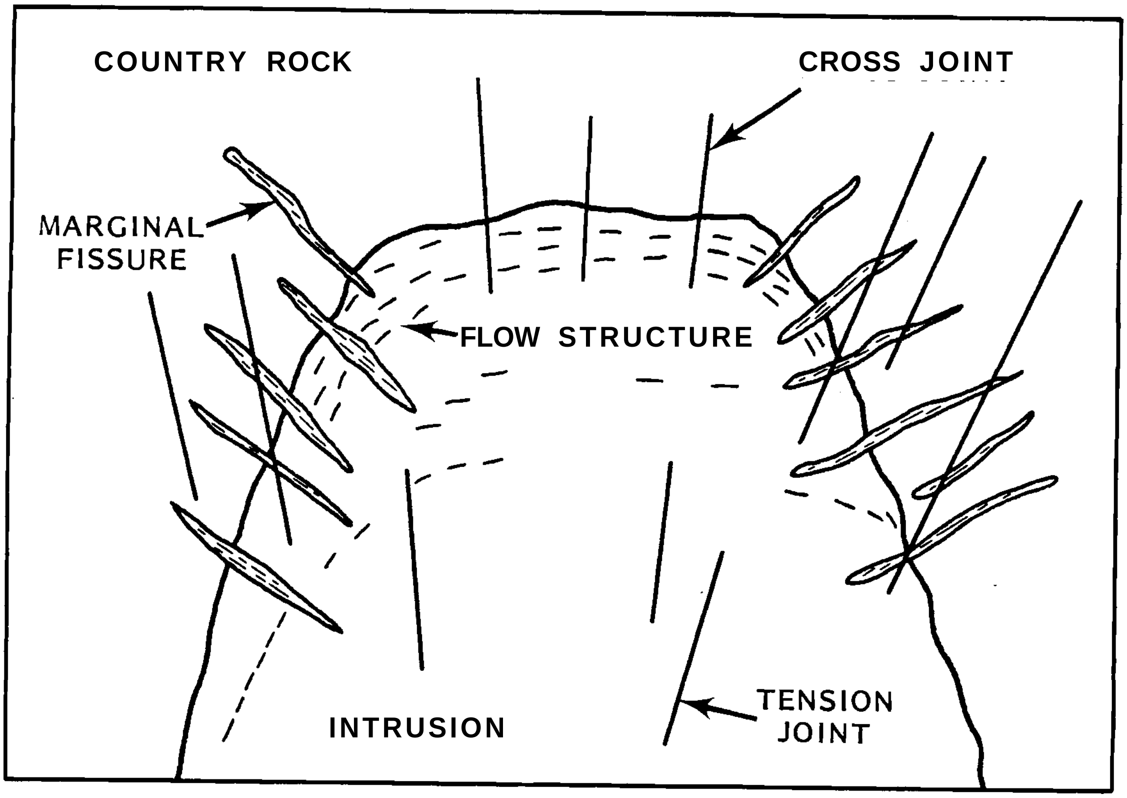

FIGURE 9. Joint System Associated with an Intrusive Rock Mass as a Consequence of Flow Structure (Port-Keller and Gnirk 1981)

Sheet joints develop in granite and other massive rocks. They are roughly parallel to the surface of the ground and divide the rock into flat sheets, as illustrated in Figure 10. Such joints are believed to result from pressure relief caused by erosion (or quarrying) of overlying rock and subsequent rock expansion in a nearly vertical direction.

In sedimentary and metamorphic rocks, joints may occur in all directions. Usually, however, a large proportion can be grouped into two or more distinct, or conjugate, sets. Figure 11 illustrates two geometries of conjugate joints. Joint sets tend to be approximately perpendicular to each other and to intersect the direction of compression at about 45°.

FIGURE 10. **Sheet Joints in a Massive Rock Mass (Port-Keller and Gnirk 1981)**

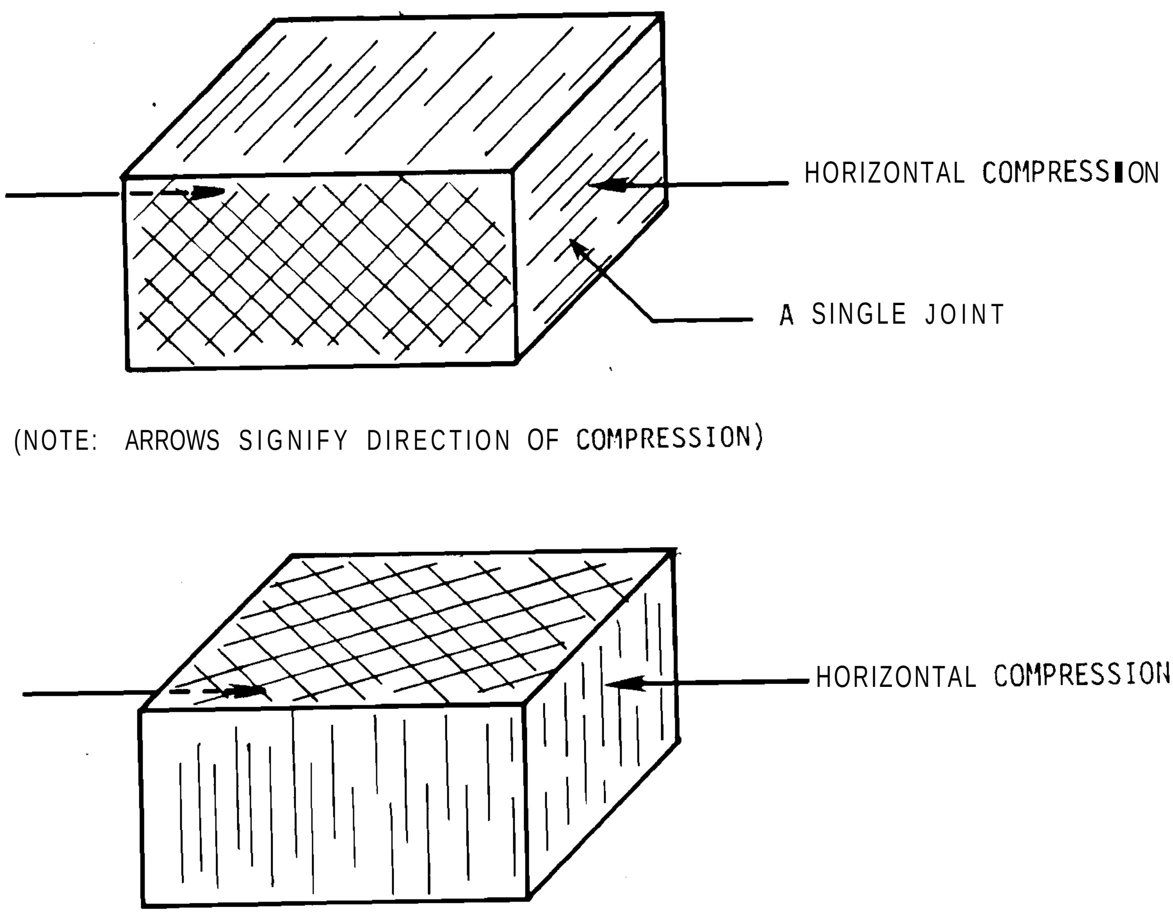

FIGURE 11. **Conjugate Sets of Compression Joints (Port-Keller and Gnirk 1981)**

The above discussion considers only a few major types of joints. Jointing in any rock mass is affected by the local and regional geologic history. The investigation and characterization of any potential host rock must necessarily include large-scale detailed mapping, possibly of every joint set visible in the rock.

2.2.2 In Situ Stress State

The in situ stress state in a rock mass is defined as that state of stress that exists before disturbance of the rock by excavation (Gnirk and Port-Keller 1978). This natural state of stress, in conjunction with the strength and structural characteristics of the rock, is important in determining the orientation, geometric shape, and dimensions of an underground excavation. Generally, the in situ vertical stress is taken to be that induced by the weight of the overburden, but it may be perturbed by regional or local tectonic features. As indicated in Figure 12, the average vertical stress to a depth of 3 km is of the order of 0.025 MPa/m (Hainison 1978) to 0.027 MPa/m (Brown and Hoek 1978) which corresponds to an average bulk density of 2,550 to 2,755 kg/m^3 (159 to 172 lb/ft^3). The approximate limits given in Figure 12 as deduced from the compilation of worldwide published data by Brown and Hoek (1978) are indicative of variations in the overburden density for different rock types and of the influence of tectonic features.

The ratio of the in situ horizontal stress to the in situ vertical stress is known as the coefficient of lateral earth stress. Figure 13 illustrates that this coefficient varies from less than one to greater than three at depths of a few hundred meters. For depths of several kilometers, the coefficient ranges from about one-third to one. Limiting curves due to Haimson (1978) obtained from hydrofracturing data indicate that the two orthogonal stresses in the horizontal plane are not necessarily equal. The curves by Brown and Hoek (1978) are upper and lower "average" limits deduced from the literature. The in situ principal stresses may not be aligned with the vertical and horizontal directions because of the

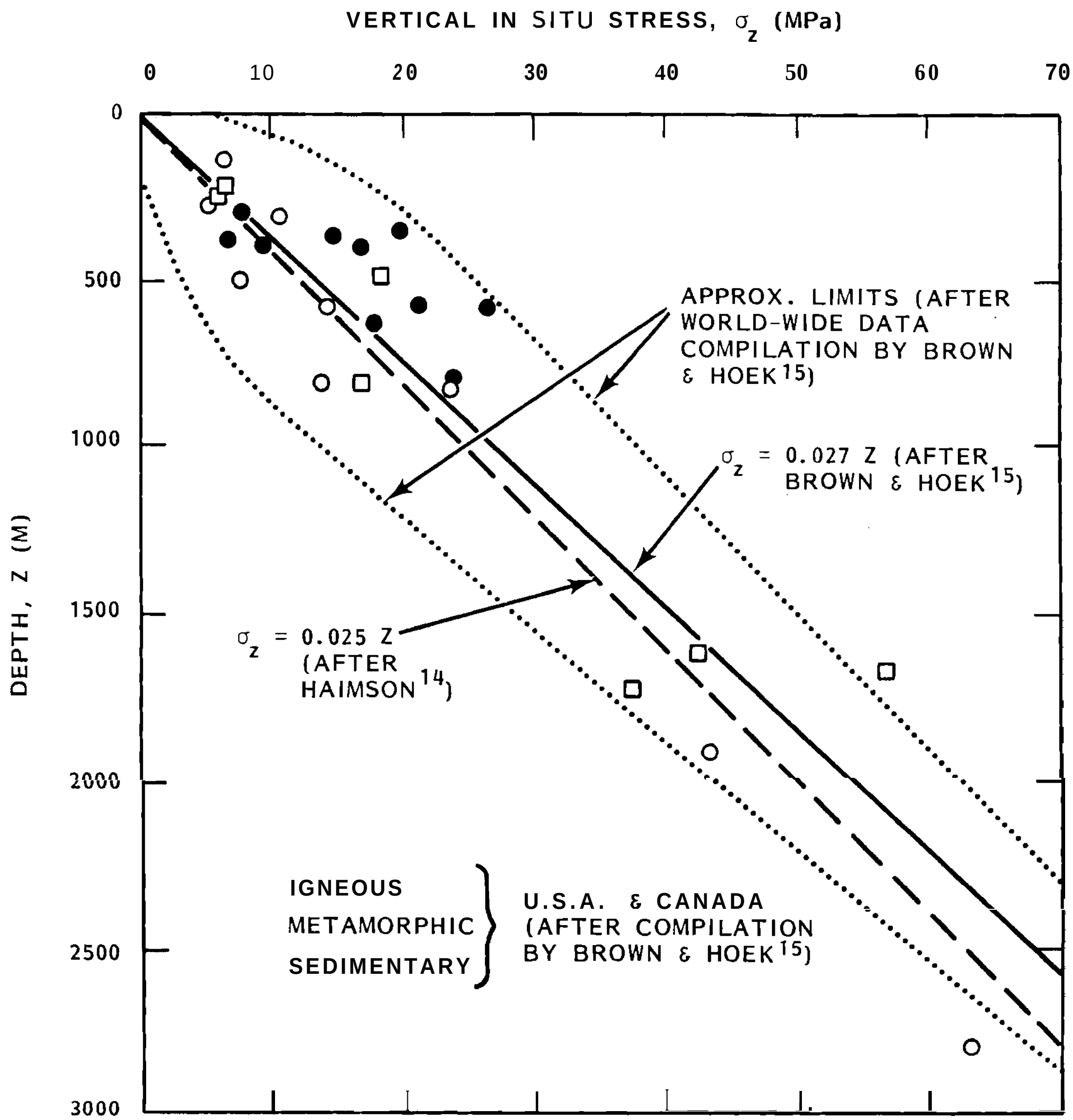

FIGURE 12. Vertical In Situ Stress as a Function of Depth (Gnirk and Port-Keller 1978)

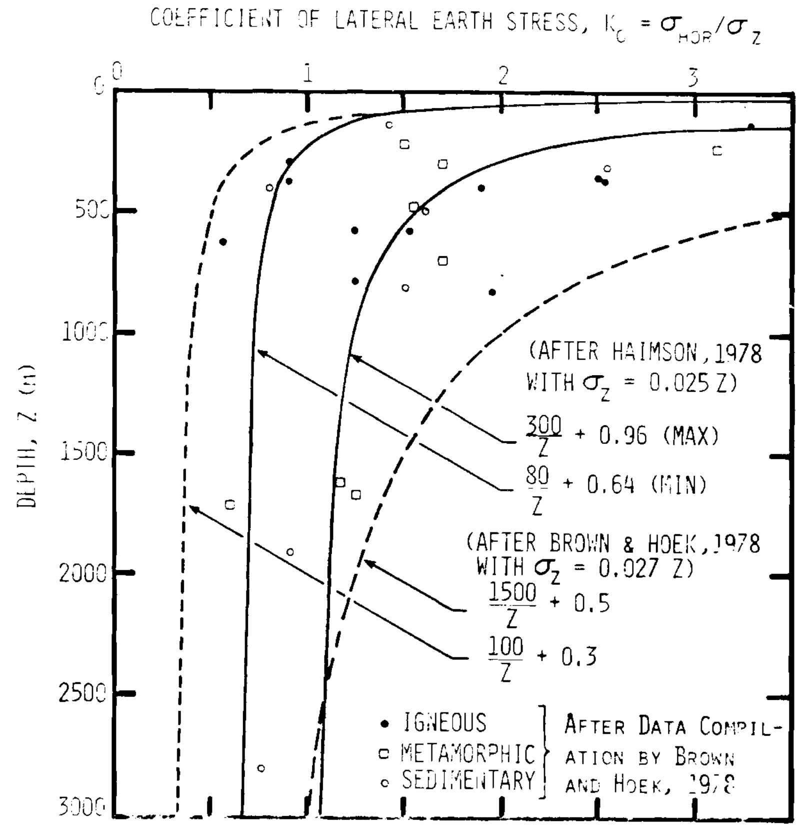

FIGURE 13. Coefficient of Lateral Earth Stress as a Function of Depth (Gnirk and Port-Keller 1978).

influence of tectonic features. No generalization can be assumed valid for in situ stress; it must be implied from onsite data or preferably measured at depth.

2.3 HYDROLOGICAL REQUIREMENTS

The literature survey of hydrology and case studies of various underground facilities, including underground storage and mines, suggests important hydrological considerations and criteria to be met for CAES hard rock cavern siting (Gnirk and Port-Keller 1978). Primary consideration must be given to such factors as the hydraulic characteristics of the host rock, ground water distribution and behavior, and ground water chemistry. These factors are described and discussed below.

2.3.1 Hydraulic Characteristics[a]

The important hydraulic characteristics of a rock mass include primary and secondary porosity, permeability and hydraulic conductivity. Porosity is the present volume available to contain fluids such as air and water. Porosity due only to pores or voids within the rock matrix is said to be primary, whereas porosity due to fractures, fissures, faults, or joints is said to be secondary. Likewise permeability, the ability of a rock to transmit fluids, may be either primary or secondary. Primary permeability depends on effective porosity, the assemblage of connected void spaces that allow fluid flow. Secondary permeability consists of through-going discontinuities in a rock mass. Low porosity, and more importantly, low permeability are essential in CAES host rock masses to 1) minimize ground water inflow during cavern construction and operation, and 2) minimize air leakage from the cavern during operation (Gnirk & Port-Keller 1978).

Table 1 summarizes the magnitudes of porosity, permeability and hydraulic conductivity generally found in various rock types (Gnirk and Port-Keller 1978). Igneous intrusives, some igneous extrusives (particularly flow basalts, rhyolites, or trachytes), high grade metamorphics, and

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[a] Intrinsic permeability, in units of length squared, L^2, is given by

$$k = -\frac{qv}{d\phi/d\ell}$$

where q is the specific discharge (flux) (LT^{-1})
v is the kinematic viscosity (L^2T^{-1})
ϕ is the fluid potential (L^2T^{-2})
ℓ is the length (L)

Hydraulic conductivity in units of length divided by time, LT^{-1}, is given by

$$K = -\frac{q}{dh/d\ell}$$

where h is the static head (L)

TABLE 1. RANGES OF HYDRAULIC ROCK PROPERTIES (MODIFIED FROM PORT-KELLER AND GNIRK 1981)

Rock Type	General Remarks on Hydraulic Conductivity	Porosity (%)	Permeability (m^2)	Hydraulic Conductivity (m/sec)
Igneous (Intrusive)	Primary: low. Secondary: generally low, but varies greatly depending on discontinuity dimensions and frequency	∿0.0 to 14	10^{-13} to 1	10^{-12} to 10^{-2}
Igneous (Extrusive)	Primary: low, except for some vesicular rocks. Secondary: low to high, depending on vesicular nature and on discontinuity dimensions and frequency	0.05 to 34	10^{-7} to 1	10^{-9} to 1
Sedimentary (Chemical)	Primary: low to high. Secondary: low to high depending on presence of karst features and on discontinuity dimensions and frequency	∿0.0 to 35	10^{-12} to 10^{-4}	10^{-8} to 1
Sedimentary (Clastic)	Primary: generally high for sandstones, breccias, and conglomerates, low for siltstones and shales. Secondary: low to high depending on discontinuities	∿0.0 to 25	10^{-15} to 10^{-4}	10^{-12} to 10
Metamorphic (Foliated)	Primary: low. Secondary: low to high and highly directional, depending on schistosity, foliation and other discontinuities	0.1 to 22	10^{-11} to 10^{-4}	10^{-10} to 1
Metamorphic (Nonfoliated)	Primary: low. Secondary: low to high depending on discontinuities and solution features	∿0.0 to 34	No data available	10^{-8} to 10^{-2}

some limestones and dolomites (those devoid of karst features) have the most favorable permeabilities for optimal CAES cavern operation. Most extrusive vesicular igneous rocks, some quartzites and marbles, and most clastic rocks have generally unacceptable permeabilities.

The permeabilities of massive high-quality igneous intrusive and metamorphic rocks are usually low. The primary porosities of granitic intrusive rocks are less than 2% because of their origins from the cooling and solidification of magmas intruded into the earth's crust. Because of high-confining pressures and crystal intergrowths, very little void space is developed in the rocks during solidification (General Electric Co. 1976; Aamodt et al 1975). However, secondary permeability is developed in granitic rocks as fractures, joints, and faults that originate from stresses created during crystallization, phase change and tectonic deformation (Walia and McCreath 1977; Aamodt et al 1975; Bergman 1977; Reinius 1977). In contrast, limestones and dolomites may contain secondary solution channels (Aamodt et al 1975; Galley et al. 1968).

Secondary permeability can cause severe air leakage if interconnected fractures and joints are drained of ground water. Ground water entering the mined opening must be closely monitored to ensure sufficient saturation of the rock mass during all stages of CAES development. Drained discontinuities will serve as conduits for air during CAES operation. Some sealing procedures may be required in excavated hard rock cavern CAES systems. Leakage through the fractured rock can be halted in some situations by special grouts that are effective under a variety of temperatures and pressures. Injected water curtains installed and operated in the rock mass that surrounds the storage cavern prior to any excavation will prevent future leakage.

Caverns should be excavated at a depth where the hydrostatic pressure of ground water equals or slightly exceeds the pressure of the stored air. Leakage of stored air may still occur if ground water does not have a downward flow toward the reservoir (Aberg 1977).

Komada et al (1980) have performed numerical studies of the effects of natural and artificial ground water pressures on storage caverns. A substantial decline in ground water level was predicted for a rock mass with permeability of 10^{-8} m/sec based on 20 years of operation.

2.3.1.1 Permeability Criteria

Because of its low viscosity, air will leak through a rock mass of relatively low permeability (Walia and McCreath 1977). Saturated rock masses with hydraulic conductivities of more than 10^{-8} m/sec may require improvement measures to prevent excessive future air leakage due to desaturation of the rock mass by water inflow into the storage cavern during the construction phase. Current techniques for measuring in situ rock permeabilities for water are not capable of defining hydraulic conductivities less than 10^{-8} m/sec. When the rock is excavated for CAES, the permeability could prove to be higher than predicted from laboratory measurements on core specimens because permeability is largely due to secondary joints not present in core samples (Walia and McCreath 1977).

Massive igneous plutonic and metamorphic rocks, and selected limestones and dolomites, are likely to meet these low permeability requirements. Secondary permeability derived from joints, fractures, and other fissures in the rocks can be controlled by constructing the storage chamber at a depth where the surrounding rock is saturated with water (Weinstein et al 1978). An acceptable air loss may be up to 2% of the total contained volume of air per day.

2.3.1.2 Physical Influences on the Hydraulic Conductivity of Joints

In situ hydraulic conductivities of rock masses decrease with increasing depth and stress. From limited field data for igneous and metamorphic rocks to depths of about 300 m, this decrease is nonlinear, and extrapolations to 1,000 m indicate conductivities of the order of 10^{-10} to 10^{-11} m/sec. Based on observations in hard rock mines, sustained ground water seepage below 1,000 m is characteristically absent. Extrapolation of water yield data from wells in crystalline rock to a depth of 1 km projects inflows of perhaps 40 to 400 ℓ/day/m (Port-Keller and Gnirk 1981).

Based on increasing in situ stress with depth, the above data and observations indicate that the hydraulic conductivity of rock is related to the stress state: Data from laboratory experiments confirm this supposition, but are generally inadequate for development of functional relationships for various rock types. In crystalline rock masses, joints are the predominant conduits for water. Hence, a relationship between hydraulic conductivity and stress state must include joint spacing, orientation, and aperture width. Limited experimental evidence indicates that the reduction of permeability of fractures with increasing stress also depends upon the primary permeability of the rock mass (Port-Keller and Gnirk 1981).

Data for the hydraulic conductivities of rock that are subjected to cyclic stress, or that experience dilatancy near the fracture stress limit, are also inadequate. Limited laboratory data suggest that the first loading cycle causes the most significant decrease in permeability, and this occurs before the onset of dilatancy. However, in the event that the deviatoric stress is sufficient to initiate dilatancy, permeability will be increased. Low-cycle fatigue could lead to eventual dilatancy and enhancement of permeability, particularly in some jointed rock masses (Port-Keller and Gnirk 1981).

Increasing temperature will lead to an initial increase in permeability of a rock, and subsequently to a decrease. Generally, this phenomenon is reversible when the temperature is decreased. Applicable information on the effects of cyclic thermal or cyclic thermal/mechanical loading on rock permeability is limited, with primary data available from CAES specific studies.

The overriding permeability factors in rocks generally considered favorable for CAES are the dimensions and frequencies of discontinuities of all types. As indicated in Figures 14 and 15, the literature suggests general reductions of permeability (caused by a combination of lesser

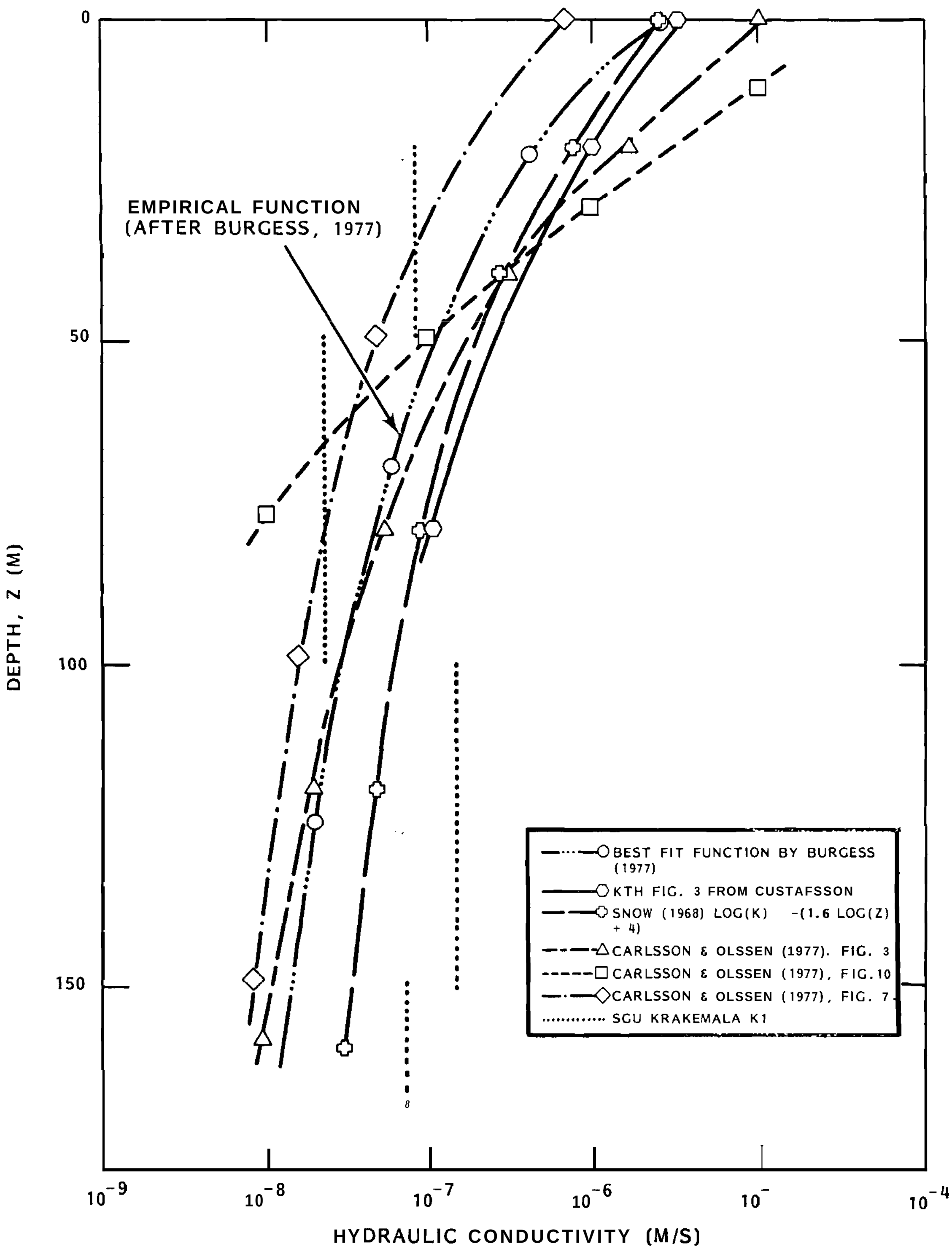

FIGURE 14. Field Data and Empirical Relationships for Hydraulic Conductivity as a Function of Depth for Igneous and Metamorphic Rock Masses (Gnirk and Port-Keller 1978)

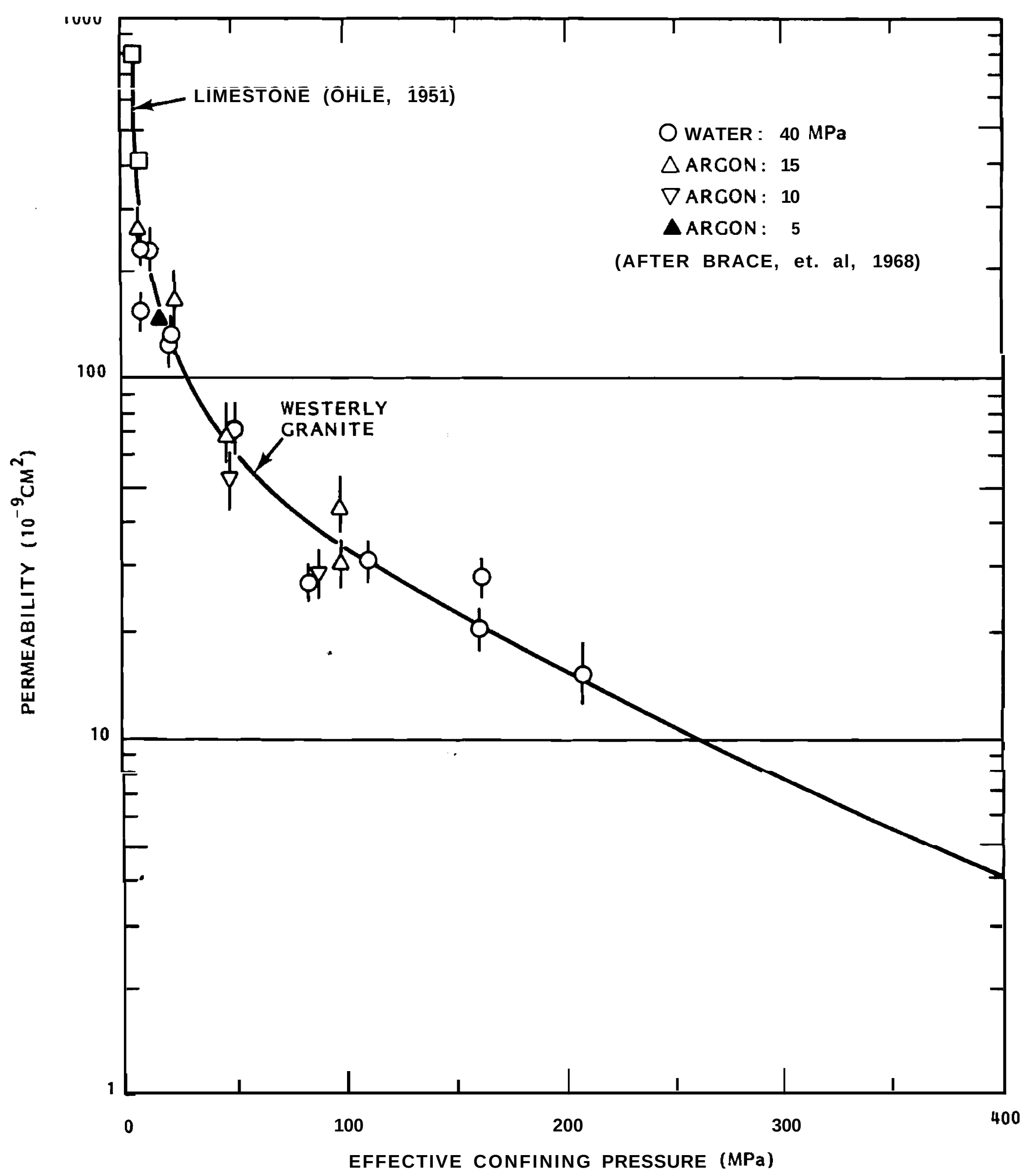

FIGURE 15. Permeability as a Function of Effective Confining Pressure for Limestone and Granite (Gnirk and Port-Keller 1978)

frequency and closing of discontinuities under pressure) with depth, and with confining pressure in laboratory experiments. The permeability-depth relation is not yet well defined for variations of rock types.

Witherspoon et al (1981) described a "void" and "asperity" model to predict fracture influence upon mechanical and hydrological properties. The fractional contact area of a fracture at an applied stress of 20 MPa was only about 0.15. Under this condition the fraction is definitely "open" to allow fluid transport even though Young's modulus becomes almost identical to that of intact rock. Very high normal stresses on the order of 200 to 300 MPa may be required to completely prevent hydraulic flow.

2.3.2 Ground Water Distribution and Behavior

The saturated zone is the region beneath the water table in which voids and fissures are filled with water under hydrostatic pressure. The optimal conditions for successful cavern operation are: 1) the cavern is located entirely within the saturated zone at a depth providing adequate hydrostatic head, and 2) a stable phreatic surface (upper boundary of the saturated zone) exists after possible fluctuations due to initial construction disturbances. Water-filled voids and capillarity should effectively retard cavern air leakage, while low permeability and discontinuities would prevent excessive water inflow to the cavern.

Generalizations about ground water stability are difficult because of the site specific nature of subsurface water distribution. Available literature does not permit the prediction of stability on the basis of rock type. Other factors, external to rock type and to the cavern situation, include precipitation, runoff, and withdrawal for consumption (Port-Keller and Gnirk 1981).

During either construction or operation of CAES systems, water inflow could be so great that mining or operation would be impossible. Conversely, in arid terranes or within the saturated zone, absence of both capillary water and water at hydrostatic head may preclude storage of compressed air within a mined cavern.

It is essential to CAES operations that the ground water not be completely drained out of the fissure system (Aberg 1977), as water-filled fissures and voids are necessary to help retard air leakage.

2.3.3 Ground Water Chemistry

Ground water chemistry may affect cavern operation (Gnirk and Port-Keller 1978). Chemical changes caused by presence of oxygen or water or by temperature fluctuations within the cavern could cause accelerated degradation of the rock mass and subsequent cavern instability, i.e., either undesirable surficial wall effects or gross structural instability. In addition, undesirable chemical constituents occurring naturally in the ground water could hinder successful equipment operation.

Chemical changes in ground water depend largely on rock type. Limestones, dolomites, and marbles are more soluble than igneous, metamorphic, or nonchemical sedimentary rocks, primarily because carbonates dissolve more readily than silicates. Ground water in noncarbonate terranes generally has low ion concentrations unless other mineralization is present, such as saline or sulfide minerals. Temperature changes due to CAES cavern operation can influence ground water chemistry.

2.3.4 Surface Requirements

Site selection depends upon the practicality of constructing a surface reservoir to provide water for the pressure compensation shaft. Initial fill and make-up water must be available from surface lakes or rivers or from shallow aquifers. A relatively impervious reservoir bottom and adequate natural or artificial lateral containment are needed; both conditions imply an impervious overburden above competent rock or a specially engineered reservoir liner.

2.4 THERMAL/MECHANICAL ROCK PROPERTIES AND TESTING

Thermal/mechanical rock properties are compiled in Table 2 (Gnirk and Port-Keller 1978). The properties are listed in terms of ranges of values, and are indicative of the relative strength and thermal

TABLE 2. RANGES OF THERMAL/MECHANICAL ROCK PROPERTIES (GNIRK AND PORT-KELLER 1978)

Rock Type		Tensile Strength (MPa)	Unconfined Compressive Strength (MPa)	Modulus of Elasticity (GPa)	Poisson's Ratio	Thermal Conductivity (W/m°K)	Thermal Diffusivity ($10^{-6}m^2$/sec)	Coefficient of Linear Thermal Expansion (10^{-6}/°K)	Specific Heat (J/kg°K)
Igneous	Max.	15.0	470	124	0.39	4.07	1.738	26.0	1001
Intrusive	Min.	3.4	6.3	3.5	0.05	0.76	0.261	4.5	754
Igneous	Max.	14.5	359	110	0.38	2.79	1.375	14.0	993
Extrusive	Min.	1.5	17.2	1.9	0.09	1.37	0.635	5.0	988
Sedimentary	Max.	10.1	400	98	0.37	5.50	1.678	28.5	1059
Chemical	Min.	0.8	21.4	1.1	0.09	1.20	0.629	4.5	796
Sedimentary	Max.	22.1	440	110	0.46	4.53	2.361	15.0	1030
Clastic	Min.	1.1	22.7	7.2	0.14	3.72	0.889	9.0	1030
Metamorphic	Max.	10.8	435	105	0.48	3.97	1.917	23.5	796
Foliated	Min.	0.6	6.9	11.0	0.02	2.02	0.569	5.0	754
Metamorphic	Max	11.0	505	104	0.40	5.33	2.80	30.0	1043
Nonfoliated	Min	1.2	62	13.0	0.06	0.75	0.219	4.5	837

characteristics of the various generic rock types, without differentiation for competency. In general, the data for jointed rock under conditions of elevated temperature and confinement stress are inadequate for cavern stability evaluations.

Rock thermal fatigue is essentially a type of weathering process. Cyclic heating of a rock in the presence of moisture generally causes some disintegration by decomposition of selected minerals and leaching of reaction products. The companion processes of thermal spalling and thermal cracking also cause disintegration. Thermal spalling and/or cracking of a rock surface occur when heating induces thermoelastic stresses above the fracture strength of the rock. Spalls are relatively thin, usually curved pieces of rock broken from the rock surface during or after heating, and may be several meters long or microscopic in size. Thermal cracking results in the production of rock fractures, often microscopic in width.

Many parameters affect the susceptibility to fatigue, spalling, and cracking (Gnirk and Port-Keller 1978). They include rock type (composition and fabric), maximum temperature change, rate of temperature change, presence of fluids, composition of fluids, and frequency of cyclic temperature and moisture changes. Thermal fatigue effects in a CAES cavern could conceivably cause severe leaching of compounds, which could alter ground water chemistry, make cavern water unsuitable for effective equipment operation, and cause elastic and strength properties of the rock mass to change. Major spalling effects could also produce gross cavern instability, while microspalling could produce significant quantities of particulate matter in the cavern air that could have destructive effects on turbine operation. Thermal cracking could result in irreversible changes in rock elastic modulus, fracture strength, porosity, and permeability.

More laboratory study is needed to quantify thermal effects in rocks, particularly with regard to the contribution of each factor listed above to long-term rock mass instability. Thermal effects combined with moisture effects are more degenerative than thermal effects alone. Abundant "microfissurization" promotes more disintegration (Aires-Barros 1975) as does higher porosity and higher permeability.

Carbonates such as limestones and dolomites may be much more susceptible to thermal and moisture effects than igneous rocks (Mailhe et al 1977). Igneous rocks with abundant mica minerals and calcium plagioclase feldspar may be more susceptible than other igneous rocks (Aires-Barros 1975).

A CAES cavern is a major underground structure that will be subjected to conditions never before encountered by large rock masses (Fossum 1982). The response of the rock mass to the associated induced stresses is of utmost concern and an accurate assessment is required of not only the uniaxial compressive strength but also of the triaxial strength under the pertaining conditions. In addition, the elastic properties under known applied stress conditions will be required as input for design analyses. An experimental program, described below, was instituted to determine which properties are significant, what tests should and can be performed, and what kind of results can be expected. RE/SPEC, Inc., of Rapid City, South Dakota, conducted this program, emphasizing CAES-specific conditions as they affect hard rock.

Operating conditions in a CAES cavern will likely include maximum air pressures of 7.35 to 8.33 MPa, requiring a facility depth of 750 to 850 m, air temperatures on the order of 30°C to 80°C, and water temperatures from just above 0°C to 30°C. The significant rock properties include physical, hydrological, geochemical, and thermomechanical properties. Generally speaking, the greatest inadequacies in physical, hydrological, and mechanical properties data exist in the areas of laboratory and in-situ values for nonintact rock testing at nonanibient conditions. Some types of data for intact rock at ambient and nonambient conditions are also marginal to somewhat inadequate (Fossum November 1980).

It was found that satisfactory results could be obtained from composite material mechanics for estimates of upper and lower bounds of thermal conductivity but that less satisfactory results could be expected for predicting thermal expansion coefficients.

2.4.1 Test Program Description

Test specimens included granite, granite gneiss, quartzite, and dolomite. Tests were conducted at room and elevated temperatures with dry and saturated moisture conditions. These included tests of indirect tension, unconfined conipression, triaxial compression, anisotropic triaxial compression, mechanically cycled fatigue, thermally cycled fatigue, and thermal conductivity. The tests were designed 1) to determine the amount of strength and stiffness reduction caused by the non-ambient conditions associated with CAES, 2) to determine if the methods for estimating the shear strength-normal strength relationship are adequate over pressure ranges appropriate to CAES, 3) to determine if the methods for estimating thermal conductivity are adequate, and 4) to establish the amount and type of testing needed to adequately define the properties of the rock for CAES application.

2.4.2 Results and Conclusions

The testing program revealed that rocks, representing the rock types typical of those that will be selected for CAES development, will experience tensile and conipressive strength reductions caused by the expected CAES cavern environment. Mean tensile strength reductions of 13 to 35% were observed after the rock specimens were saturated with water (Fossum 1982). This strength degradation is attributed to a weakening of bonding strength of the rock structure and occurs regardless of the initial degree of saturation or pore-water pressure. The angles of internal friction and cohesion, however, appeared to be unaffected by water.

Temperatures from 50° to 150°C had negligible effect on the strength properties of all rocks tested. However, it was found that when a rock was heated and then quenched in cold water, a significant reduction was observed in the rock's cohesion. After the first thermal shock, no subsequent damage was noticeable upon additional thermal cycling. In addition, no significant difference was apparent between the cohesive strength of rock specimens cycled between 0° and 50°C and those cycled between 0° and 100°C or between 0° and 150°C. After the first thermal cycle, the angle of internal friction either remained unchanged or increased slightly. The loss in cohesion was about 30 to 50%. Figure 16 shows the Mohr failure envelope for Milbank Granite after one thermal cycle between 0° and 50°C together with the failure envelope obtained at ambient conditions. Figure 17 shows similar results for Sioux Quartzite. Note that because the friction angle either remains constant or 'increases, the percentage of shear strength reduction decreases with increasing confining pressure.

Mechanical fatigue testing showed that, for loading that alternates between tension and compression, over 50% of the tensile strength could be lost within 10,000 cycles, corresponding to a CAES cavern life of approximately 30 years. The damage appears to occur in the tensile portion of the cycle (Fossum 1982). Figure 18 shows cyclic stress-strain curves for Milbank granite cycled between ± 62% of its tensile strength. Damage from micro-cracking can be seen by the change in Young's modulus in the tensile region of the stress-strain plot of cycle 2500 (Figure 18). Figure 19 shows how this modulus decays with cycling. Figure 20 shows S (strength) - N (cycles) plots for Milbank granite cycled in tension-compression, tension-tension, and compression-compression loading modes.

.Regardless of the mode of loading, the rock was weakened by repeated cycling even though the magnitudes of loading were less than those necessary to cause static failure. The most favorable type of cyclic loading from the standpoint of fatigue strength was compression-compression. Over the projected lifetime of a CAES cavern (about 10,000 cycles) this rock would lose approximately 16% of its compressive strength for this type of loading.

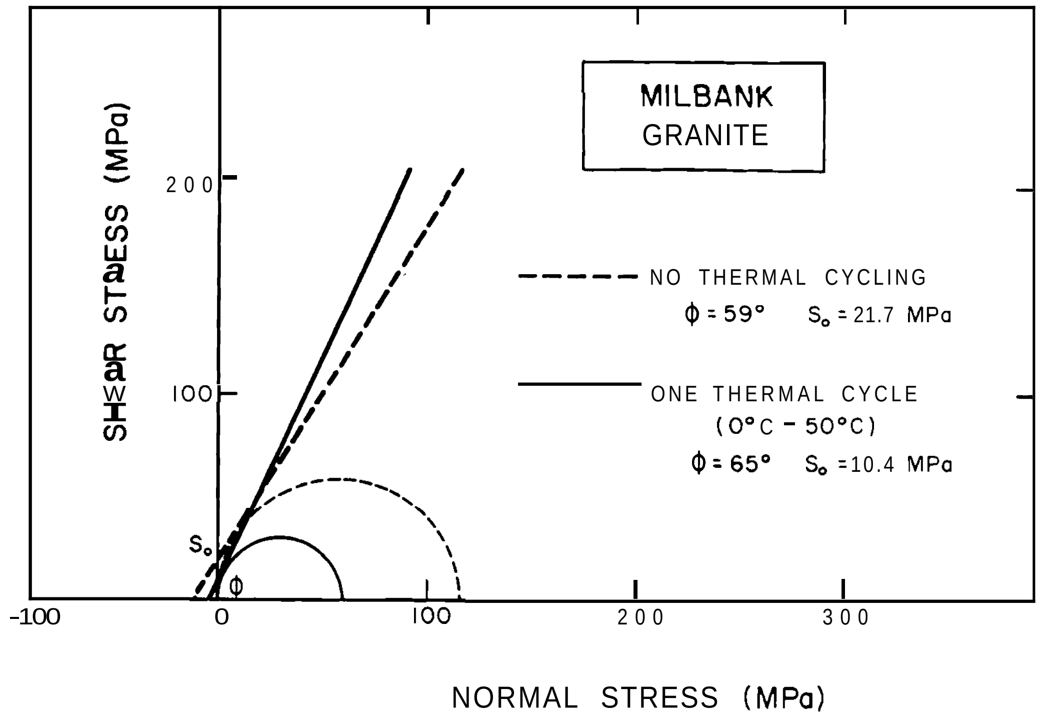

FIGURE 16. Mohr Failure Envelopes for Milbank Granite (Fossum 1982)

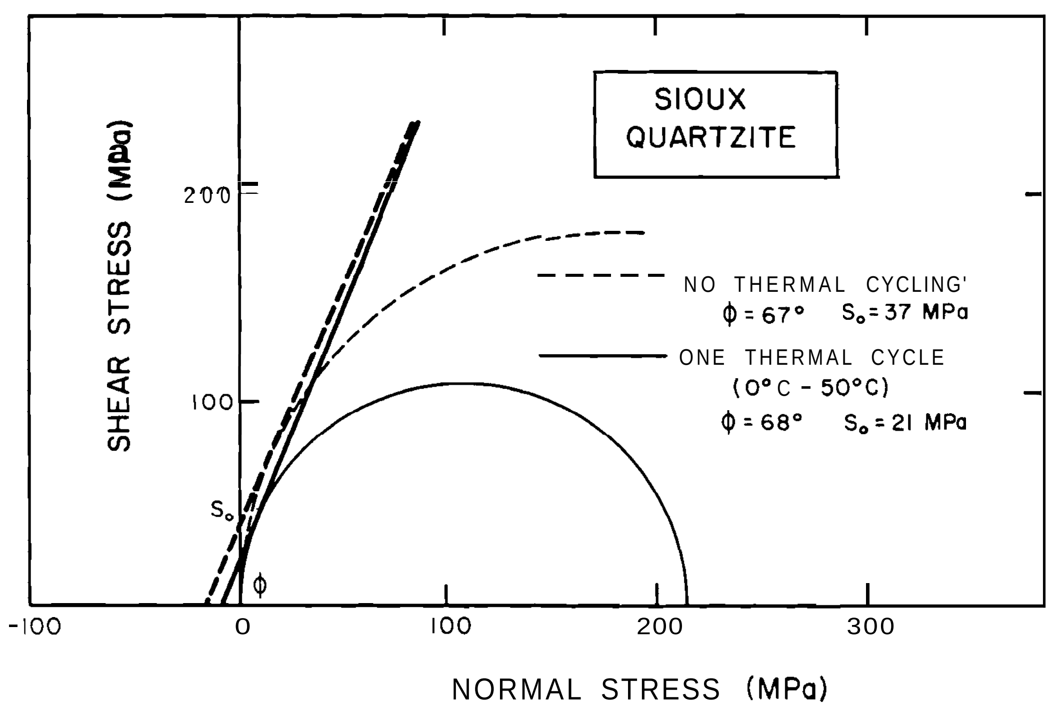

FIGURE 17. Mohr Failure Envelopes for Sioux Quartzite (Fossum 1982)

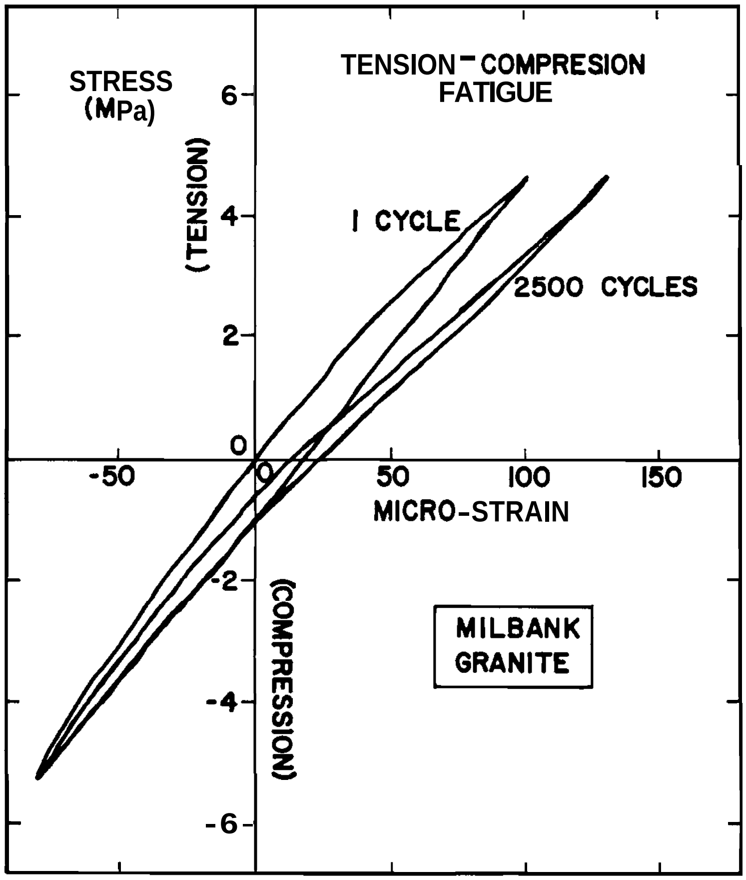

FIGURE 18. Cyclic Stress-Strain Curves for Milbank Granite (Fossum 1982)

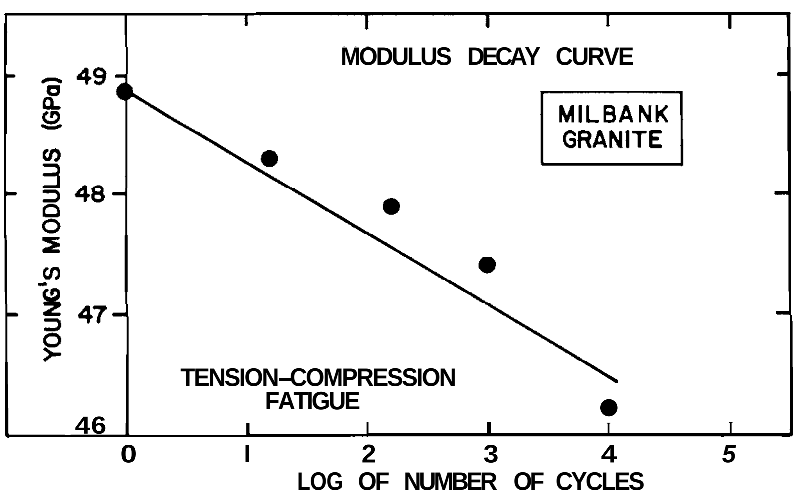

FIGURE 19. Modulus Decay Curve for Milbank Granite (Fossum 1982)

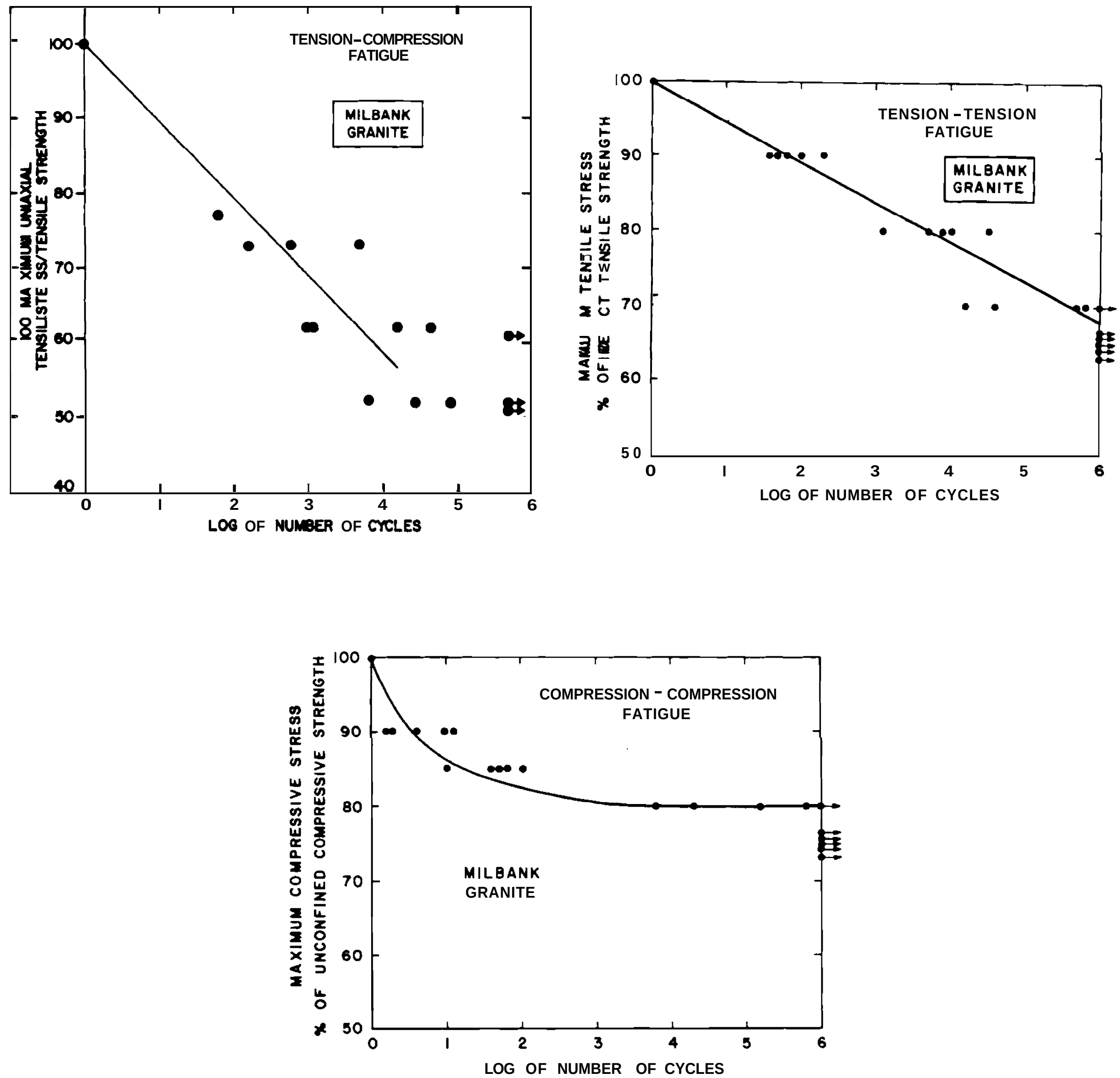

FIGURE 20. S (strength) - N (Cycles) Plots for Milbank Granite Cycled in Tension-Compression, Tension-Tension, and Compression-Compression Loading Modes

Planes of foliation in a rock mass have a pronounced effect on ultimate strength, reminiscent of jointed rock behavior as shown in Figure 21. When the direction of the major principal stress is oriented at 30" to 50" from the foliation plane, the strength can be reduced by as much as 50% as compared to that for orientations of 0" and 90" (Fossum 1982).

Strength reductions can be expected in the tensile and compressive strengths on rock masses subjected to a CAES cavern environment. The loss of tensile strength is not a major concern, as this strength is generally quite low to begin with. For design purposes, it would be wise to assume that the host rock has no tensile strength (Fossum 1982), because an already low tensile strength will be reduced by moisture, fatigue, and thermal shock. The nearly 50% loss of cohesion caused by

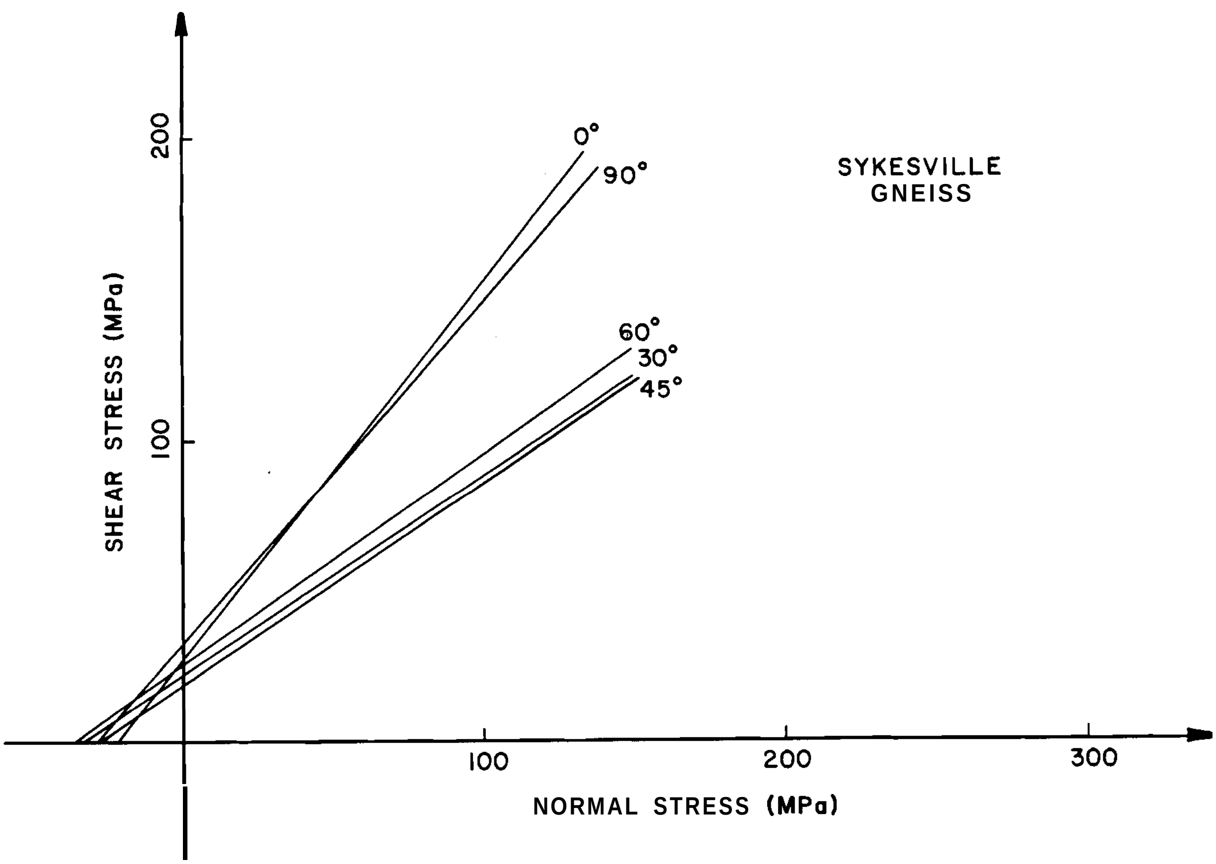

FIGURE 21. Variation of Strength Envelope of Sykesville Gneiss with Foliation Angle (Fossum 1982)

the presence of moisture and thermal shock is tempered by the fact that the frictional coefficient remains relatively constant. Thus, the percentage loss of failure strength of the rock decreases with increasing mean stress. Most of the damage will be caused in the first few cycles.

For design purposes, it is advisable to assume that the angle of internal friction does not increase and that the cohesion is reduced by 50%. In light of the experimental results of this program, this assumption would be conservative (Fossum 1982).

If the rock is foliated or jointed, the orientation of the cavern with respect to the angles of jointing or of foliation planes must be duly considered. The shear strength of the rock is a strong function of the orientation of the potential planes of weakness with respect to the principal stresses.

Over the confining pressure and temperature ranges of interest to CAES application, the failure envelopes were linear for the stronger hard rocks and parabolic for dolomite. Because the friction coefficient was not influenced by the nonambient conditions, future reasonable estimates of the failure envelopes could be made with indirect tension and unconfined uniaxial compression tests. After the failure envelope has been established, it would be advisable to truncate the tensile portion of the envelope to zero, because an already low tensile strength will be reduced by mechanical fatigue and thermal shock.

The RE/SPEC testing program has demonstrated that, although the compressive and tensile strengths are adversely influenced by a CAES cavern environment, the reduced failure strength of initially intact hard rocks is sufficiently high to indicate that a CAES plant could be operated satisfactorily (Fossum 1982). The program also demonstrated that foliated and/or jointed rock masses require special attention; assessment of strength requirements and strength values become quite site-specific.

The results of this program and other PNL sponsored programs were used to establish stability and design criteria. These conclusions are incorporated within Section 6.

2.4.3 Creep of Jointed Rock Material

One thermo-mechanical property not tested by RE/SPEC is rock creep, the continuing movement of rock with time under relatively constant stress. This can be a significant engineering problem whenever large loads must be sustained for long durations. As is true in almost all rock engineering phenomena, the creep of rock masses in situ will be governed primarily by the behavior of the discontinuities--the bedding planes, faults, and, in particular, joints. However, nearly all previous research on rock creep has been aimed at determining the form of the creep law for small, unjointed laboratory specimens. Although this essential first step has defined the mechanisms and variables for the intact rock case -- which will also be some of the mechanisms and variables for the jointed rock case.-- it is not sufficient to develop a comprehensive quantitative creep model for rock masses (Schwartz and Kolluru 1981).

Time-dependent deformations of rock can be categorized as either 1) creep (often called "squeezing" in tunneling), which most generally refers to any time-dependent rock behavior but which in practice usually connotes time-varying, primarily shear deformations, or 2) consolidation and/or swelling, more restrictive terms referring to purely volumetric time-dependent deformations. These two types of behavior involve fundamentally different mechanisms in the rock. Consolidation/swelling is usually associated with the flow of water out of or into the rock pores or geochemical response of the rock to changed environment. Creep, on the other hand, is primarily the product of time-dependent microfracturing of the rock: if the rock is dilatant, this microfracturing will produce both shear and volumetric strains.

Time-dependent microfracturing is the dominant, but not the only, mechanism in rock creep. Other secondary mechanisms include 1) twinning and translation gliding in individual mineral crystals, 2) recrystallization, especially at high temperatures, 3) dislocations at grain boundaries, and 4) viscoelasticity of the matrix material in aggregated rocks (e.g., sandstone, shale). However, the microfracturing mechanism has been most thoroughly investigated.

Limited laboratory comparisons of creep in jointed and intact rock show that instantaneous strains for jointed specimens exceed the intact rock creep deformations by several orders of magnitude. This indicates that rock creep must be evaluated on the basis of jointed rock mass behavior. In situ field evaluation will be required for any candidate CAES site.

3.0 CAVERN STABILITY

All aspects of cavern stability known to have potential effects on compressed air storage are treated in this section. Mechanisms of instability, numerical modeling, cavern excavation, and precedent stability considerations are included.

3.1 TYPES OF INSTABILITY

Three types of CAES cavern instability have been suggested by Gnirk and Port-Keller (1978):

- general rock instability - identified by massive roof falls, wall slabbing, and floor heave, leading to the loss of structural integrity of the cavern or its entrance, or both
- local rock instability - identified by localized thermomechanical spalling and thermochemical disintegration of the rock over the cavern periphery, leading to particulate transport during compressed air withdrawal to the turbine system
- air leakage instability - identified by unacceptable air leakage from the cavern during compressed air injection and storage (due to greatly enhanced hydraulic conductivity as a result of drained discontinuities or induced fracturing or joint dilation).

In practice, we may define the time periods of instability concern for a system of CAES caverns as 1) excavation, 2) operation, and 3) decommissioning. Both general and local rock instability apply to the excavation and decommissioning periods, whereas all three instability concerns apply to the operational period.

3.2 MECHANISMS OF INSTABILITY

Clearly, the development of stability criteria involves the specification of limits on the thermal mechanical/hydrological behavior of the rock mass, such that stability exists when the limits are not exceeded. Thus, a thorough understanding of the mechanisms and modes of each instability type is of primary importance.

3.2.1 General Rock Instability

General rock instability may be caused by any of four mechanisms: brittle fracture, ductile fracture, creep rupture, and fatigue fracture (Gnirk 1979). Each is described below.

In brittle fracture initiation, the required magnitude of differential stress characteristically

- increases with increasing confinement stress, commonly in a linear fashion for compressive mean stresses up to about 50 MPa
- increases with an increasing rate of deviatoric stress application
- decreases with increasing temperature, often exponentially
- decreases with decreasing effective stress (difference between the confinement stress and the pore pressure)
- decreases with increasing percentage of water saturation
- may be 10 to 20 times greater in uniaxial compression than in uniaxial tension.

In general, the magnitude of strain at the initiation of ductile fracture:

- increases with increasing confinement stress
- increases with increasing temperature
- decreases with increasing rate of deviatoric stress application
- decreases with decreasing effective stress
- decreases with increasing percentage of water saturation.

Constitutive laws of rock failure are commonly used to evaluate conditions of potential fracture or flow in rock mechanics analyses of underground structures (Gnirk 1979). However, these laws do not include time-dependence and cannot predict either creep rupture or fatigue fracture resulting from accumulated rock degradation.

Rock creep under constant stress and temperature is characterized by three consecutive stages of deformation:

- transient creep-decreasing strain rate
- steady state creep-constant strain rate
- tertiary creep-increasing strain rate leading to rupture or fracture.

In general, creep is increased with increasing shear stress, temperature, and moisture content and retarded with increasing mean stress and decreasing effective stress. For confinement stresses and temperatures over the ranges of 0 to 100 MPa and 25 to 150°C, the amount of creep deformation in most dense intact hard rocks will be small, i.e., about 0.1% to 0.2% before rupture (Gnirk 1979).

Fatigue failure involves cyclic differential stress that progressively weakens the rock until fracture occurs. In a CAES cavern the rock is subjected to cyclic stress and temperature perturbations. The notion of "time to failure" must be adjusted to include the number of cycles to failure. The mechanism of fatigue fracture under cyclic temperature is related to the differential thermal expansion of the mineral constituents of the rock mass. This mechanism would tend to weaken the rock relatively rapidly if the temperature during a cycle varied extensively. As in the case of creep rupture, the fatigue strength of a rock is increased with increasing confinement stress. Of considerable interest, however, is the observation that the fatigue strength of rock subjected to alternating compression-tension cycles is considerably less than that for purely cyclic tension. Limited data suggest that mechanically stronger rocks have a higher fatigue strength than weaker rocks (Gnirk 1979).

Applicable generic rock types for CAES caverns include igneous intrusive (granites, granodiorites, gabbros), certain metaniorphic (quartzites and gneisses), and a few sedimentary (dolomites, and limestones and sandstones of low porosity and high relative strength) rocks. Within the ranges of stresses and temperatures anticipated for CAES caverns, the predominant mode of general instability will be characterized by loss of cohesion along joint planes or brittle fracture of intact rock. Planes of weakness (i.e. , joint systems and bedding planes) in rock masses will strongly influence the generation and orientation of fractures. In fact, the global strength of a rock mass with planes of weakness, i.e., a nonintact rock, will be substantially less than that of the intact rock. The strength obviously depends upon the orientation of the major deviatoric stress with respect to the plane of weakness. In effect, the situation may be compared to that of anisotropy, where the strength is a function of direction.

For brittle and ductile fracture, the influence of joints can be incorporated into the constitutive failure conditions. This procedure permits evaluation of the potential for global instability in the rock mass around a cavern for quasi-static loading conditions. However, the data are not generally available for calculations of the potential for fatigue fracture in jointed rock in even a broad generic sense.

3.2.2 Local Rock Instability

Local rock instability of a cavern involves spalling and degradation of the rock along the periphery (Gnirk 1979). The main consideration is whether or not its occurrence will detrimentally affect the storage and use of compressed air. Detrimental effects could be anticipated if rock particles were entrained in the compressed air stream during withdrawal to the surface turbine system. For conditions of wet cavern walls and relatively low air stream velocities, particle entrainment seems unlikely, except for perhaps extremely fine particles of dust size. Local instability is important, however, if it compromises the global stability of the cavern, or contributes to blockage of the cavern entrance.

The primary mode of local instability will probably be thermally-induced fracturing of the microstructure with associated weakening of the rock by weathering like processes. Thermally-induced fracturing will probably be restricted to rock surfaces in direct contact with the compressed air. Conversely, both the exposed rock surfaces and any open planes of weakness will be subjected to degradation by alternating water and hot air contact. General quantitative evaluation of either phenomenon, or their coupled influence on local rock instability, will be difficult because experimental data and appropriate constitutive laws are lacking. Thermally-induced fracturing and weathering are likely to be site-specific.

3.2.2.1 Thermal Spalling

Thermal spalling occurs when a rock surface is subjected to a substantial temperature change. Air temperatures as low as 60°C could conceivably initiate spalling. The abrupt heating of a polycrystalline rock surface gives rise to a nonhomogeneous field of thermal expansion caused by the contrasting differences in expansion coefficients of the mineral constituents (Gnirk 1979). The induced state of stress in the plane of the rock surface is highly compressive. Because the outward displacement of the rock surface is effectively unrestrained, the stress state is analogous to that of an extension test. Fracture is initiated in the rock parallel to the heated surface, resulting in a planar fragment or spall. The spall thickness is a function of the magnitude and duration of the heat flux on the surface and the thermal/mechanical properties of the rock. The thickness of spalls may range from 1 to 3 cm, with maximum lateral dimensions of 0.1 to 0.6 m. Fracture surfaces are determined primarily by the thermal stress pattern rather than by the rock structure.

To evaluate thermal spalling potential in a CAES cavern, the transient state of induced thermoelastic stress in a rock surface could be calculated by conventional finite-element procedures. By use of the potential failure index evaluation an assessment could be made of spalling for given temperature histories at the fluid-rock interface. The strength parameters in the failure criteria must reflect the results of triaxial extension tests for particular generic rock types. This procedure would not reflect any dynamic characteristics of the spalling phenomenon, but should provide some limits on the rate of spalling with time (Gnirk 1979).

Hood (1979) discussed possible mechanisms for thermal deterioration of rock. These include: 1) dehydration of clay minerals, 2) differential thermal expansion of individual crystalline units, and 3) gross rock failure which is probably stress related. Witherspoon et al (1980) reported thermal decrepitation of vertical borehole walls near 300°C.

3.2.2.2 Weathering

Rock on the cavern periphery is weathered or degraded by the cyclic action of water and hot air at high relative humidity. The weathering mechanism is chemical in nature, and accelerated with increasing temperature. Essentially, certain mineralogical components of a rock, especially parallel to planes of permeability within a rock mass, may be susceptible to dissolution, hydration, and leaching. The net effect is a gradual weakening of rock micro- and/or macrostructure, which leads to a reduction in fracture strength (Gnirk 1979).

Relatively high stress perturbations in the rock around the periphery of a cavern, and the cyclic water-warm air contact, may accelerate weathering. Materials filling joint planes or other planes of weakness in igneous and metamorphic rocks will also be subjected to weathering. Currently, quantitative evaluation of the rate of weathering under CAES cavern conditions does not seem possible. However, standard weathering tests for aggregate use in pavements and concretes may apply.

3.2.3 Air Leakage Instability

Air leakage instability is defined as either an unacceptable pressure decline in uncompensated CAES caverns, or an unacceptable air volume loss in compensated CAES caverns (Gnirk 1979). The mode of instability is the outward migration of air from the cavern into the surrounding rock mass through fractures, fissures, joints, faults, and connected pores or voids. If these conduits are water-saturated, the air must either migrate upward as bubbles, dissolve within the cavern water, or displace the water.

The permeability of a rock or rock mass is a measure of its capacity for transmitting a fluid. Permeability, when referenced to the original pores and voids of a rock, is designated as being primary; and, secondary when referenced to subsequent fractures, joints, and other discontinuities or planes of weakness. In general, primary permeability is a characteristic feature of sedimentary rocks, and secondary permeability is more representative of metamorphic and igneous rocks. However, sedimentary rocks are also frequently jointed, contributing to secondary permeability. The hydraulic conductivity of a rock or rock mass is basically a measure of the rate at which water is transmitted, and involves a combination of porosity, permeability, and fluid viscosity. Laboratory and field data demonstrate that rock mass permeability

- decreases with increasing depth
- decreases with increasing compressive stress
- generally decreases with increasing temperature
- increases as the level of the fracture stress is approached and volume dilatation of the rock is augmented.

3.3 NUMERICAL MODELING OF CAVERN STABILITY

Cavern stability is affected by cavern design, post-excavation stress factors, cyclic thermal penetration of the cavern walls and cavern capability to contain compressed air. A thorough assessment of these factors is necessary in determining the suitability of a cavern for CAES applications. Laboratory measurements of rock properties under cavern in situ and CAES conditions will be needed to determine mechanical and geochemical changes for various rock types.

SPECTROM, a bank of computer programs for geotechnical analyses assembled by RE/SPEC of Rapid City, South Dakota, is particularly applicable to problems in rock mechanics, involving discrete or coupled phenomena in the thermal, mechanical, and hydrological realms of rock behavior. These programs use isoparametric elements, with capabilities for handling nonlinear and anisotropic material properties, time-independent and time-dependent nonlinear material response through post-failure, and two- and three-dimensional geometries. Each code within the series has been extensively validated with closed-form analytical solutions and available commercial finite-element codes on CDC CYBERNET, and with laboratory and in situ (or field) case history data where available. The SPECTROM codes were used to numerically model CAES cavern stability factors.

3.3.1 Cavern Design

Parametric analyses of the excavation phase of CAES caverns produced a catalogue of stress concentration factors for single and multiple caverns with different shapes and extraction ratios under a range of initial in situ stress states (Brandshaug and Fossum 1980). (A conceptual drawing of underground CAES caverns is shown in Figure 22.) This information was related to case history data to determine artificial support requirements for caverns situated at a depth of 750 m in igneous rock. An example of such a guide is given in Figure 23. Quantitatively it was found that

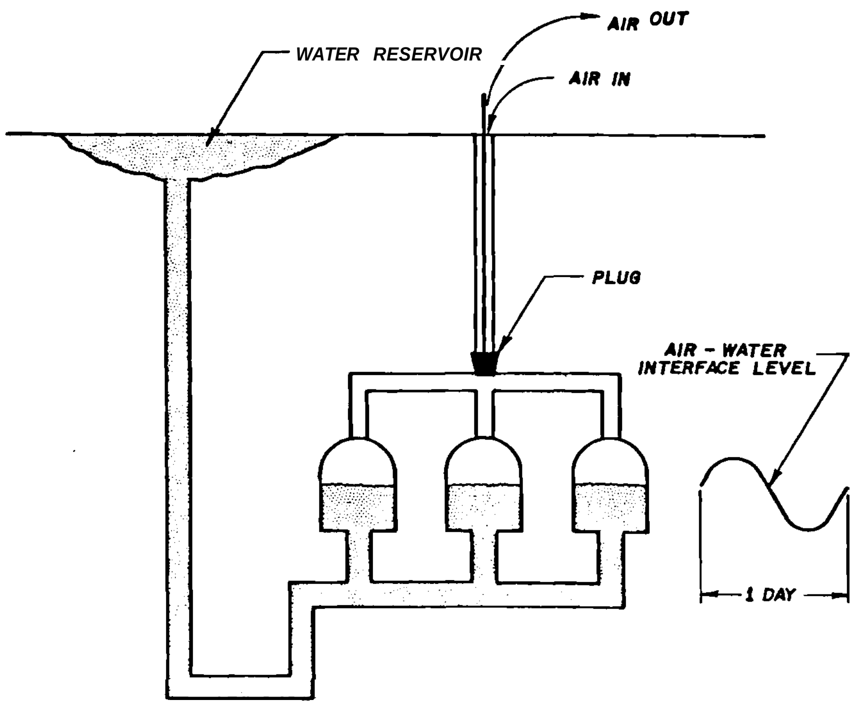

FIGURE 22. Compensated CAES Caverns

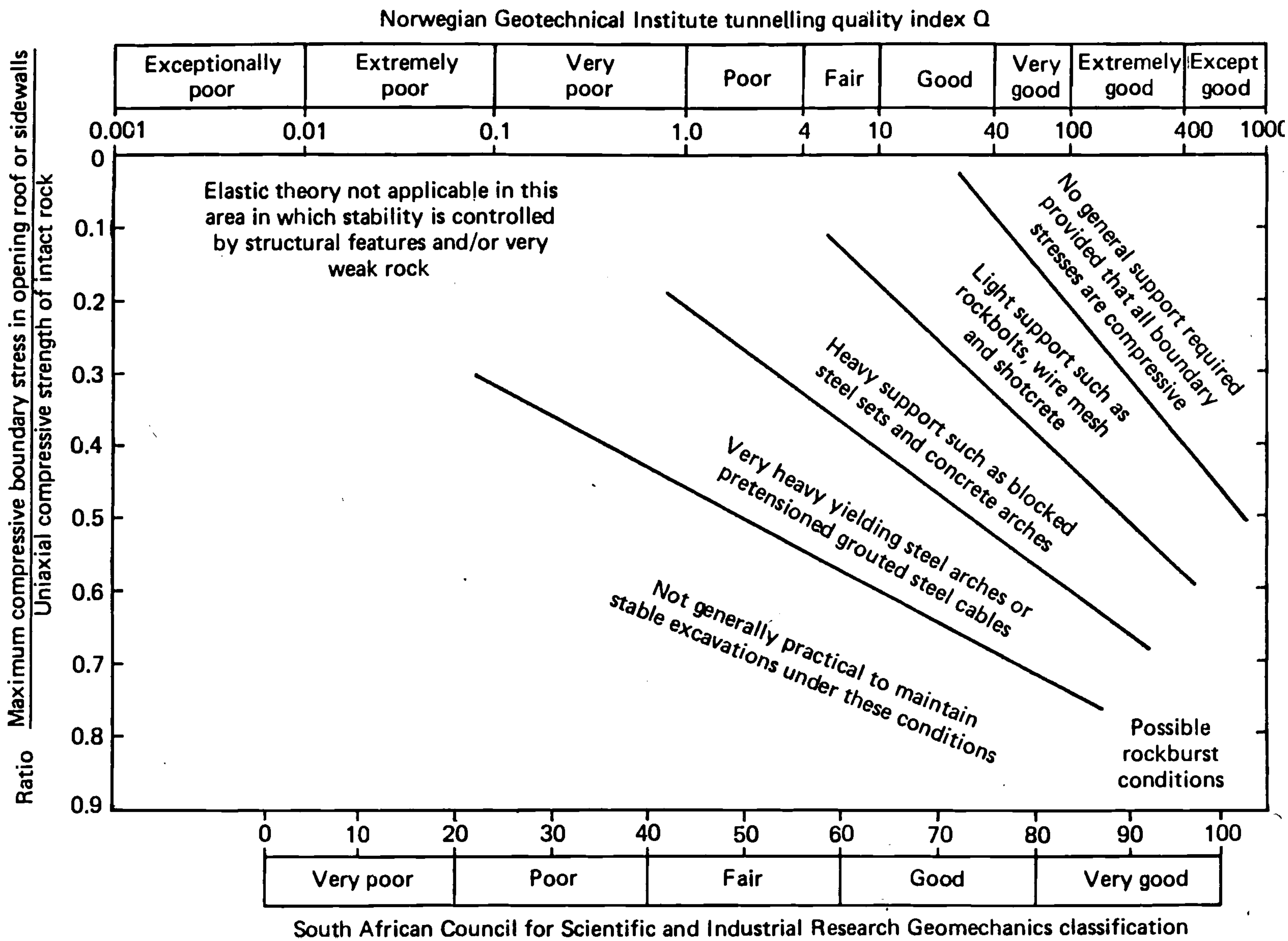

FIGURE 23. Underground Excavation Stability as a Function of Induced Stress and Rock Mass Quality (after Hoek 1979)

the rock around a single cavern experiences higher stresses than those induced in the rock around parallel caverns until the extraction ratio is less than 20%. For extraction ratios less than 20% percent a multiple cavern array behaves as a single cavern.

The air and water temperatures assumed in the thermal analyses were 40°C and 10°C, respectively. It was found that after 50 operational cycles, the rock temperatures approached a steady oscillatory behavior within the first meter into the cavern walls. Because both the roof and the floor were maintained at constant temperatures of 40°C and 10°C respectively, the temperature gradient extended much farther into the rock at these two locations than into the walls. Figure 24 shows the temperature isotherms around a CAES cavern for different operational cycles. Relatively high thermal gradients exist in the rock all around the cavern. The thermal gradients in the roof and the floor diminish as operation continues; however, the gradient in the walls does not change substantially from the initial stage of operation.

The oscillatory behavior of the temperatures in the cavern walls leads to oscillatory behavior of the stresses as well. Figure 25 shows three types of cyclic behavior that can result, namely tension-tension, tension-compression, and compression-compression cycling (Brandshaug and Fossum 1981). For multiplc caverns, extraction ratios were found for which the tension-tension, and tension-compression cycling could be eliminated.

In the failure analyses performed for the excavation phase, it was found that caverns could be located in a competent igneous rock at 750 m without failure for a wide range of initial in situ stress ratios and cavern height-to-width ratios. However, when two sets of joints were introduced situated at ± 45° to the horizontal, joint failure could occur as shown by the shaded regions in Figure 26. Decreasing the depth to 500 m decreased the failure zone to that shown in Figure 27. When the cavern of Figure 26 was pressurized, corresponding to a static head

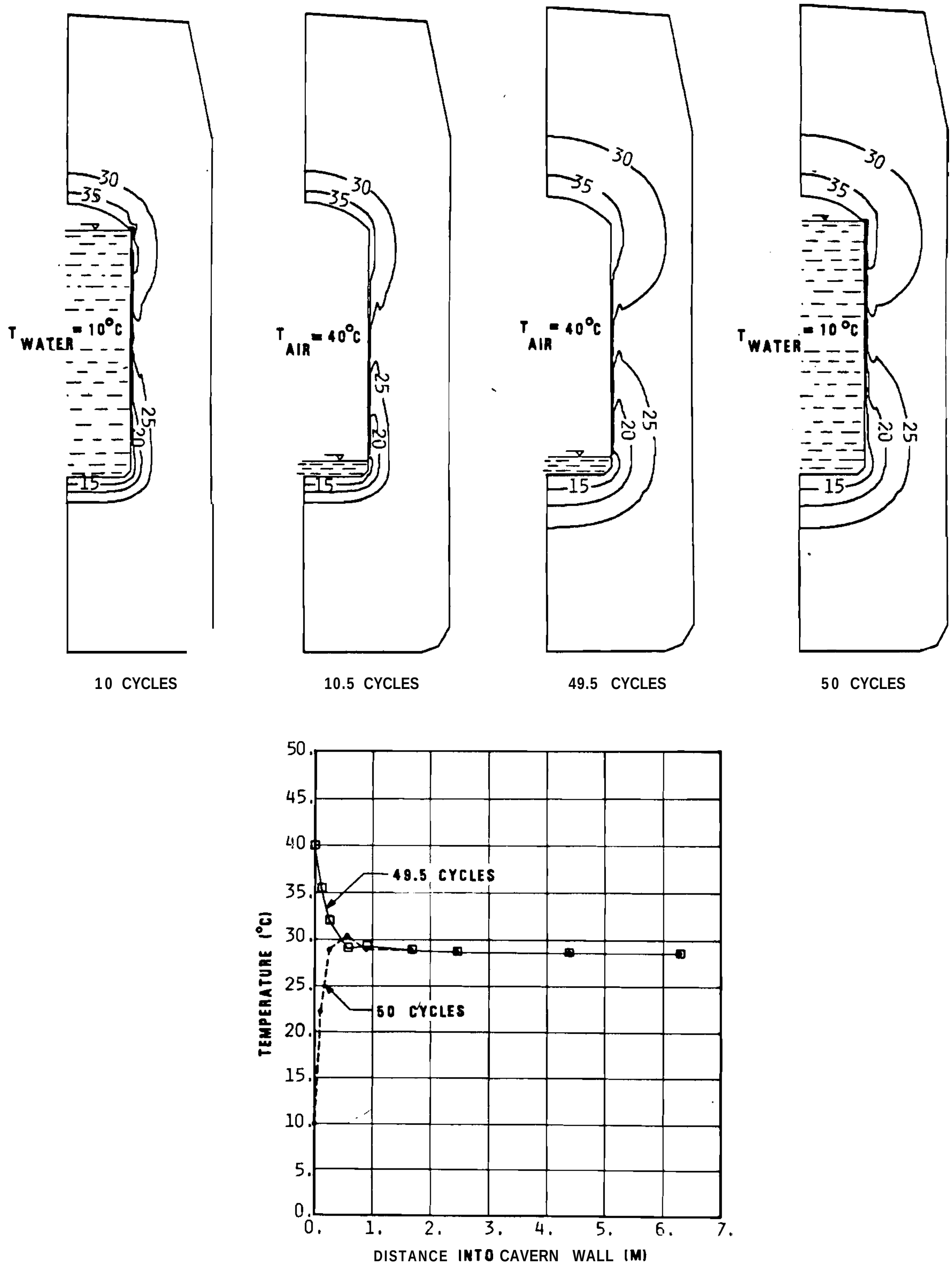

FIGURE 24. Isotherms (°C) and Temperature Distribution into a Single-Cavern Wall for Various Cycles (Brandshaug and Fossum 1981)

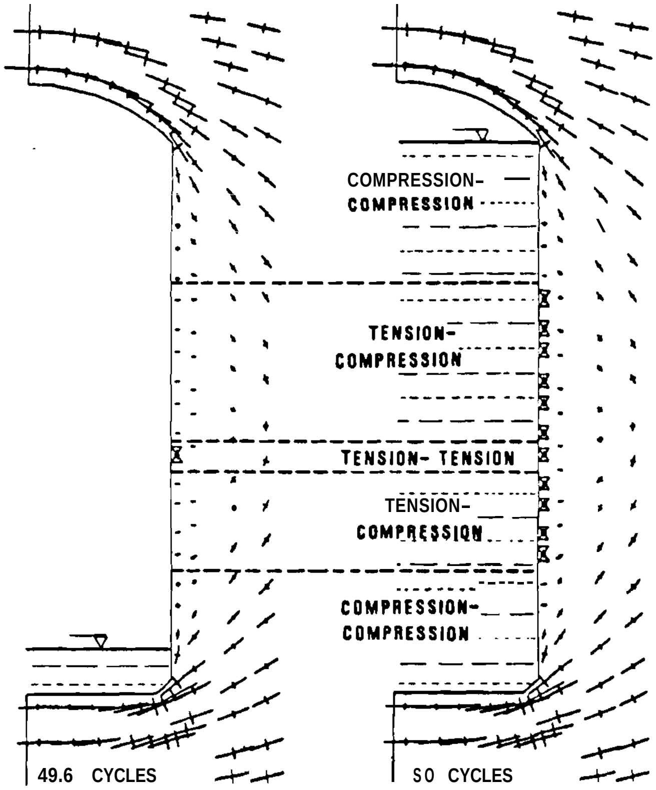

FIGURE 25. Principal Stress Trajectories Around a Single CAES Cavern After 49.5 and 50 Cycles (Brandshaug and Fossum 1981)

of water of 750 m, joint failure was limited to the zones shown in Figure 28. Thus, the cavern pressurization is an unloading process.

The failure analyses performed for the operational phase showed that multiple caverns could be located in a competent igneous rock at 750 m depth without failure, even after substantial strength reductions caused by fatigue or thermal shock were simulated. However, for the cases considered, no tensile stresses had developed.

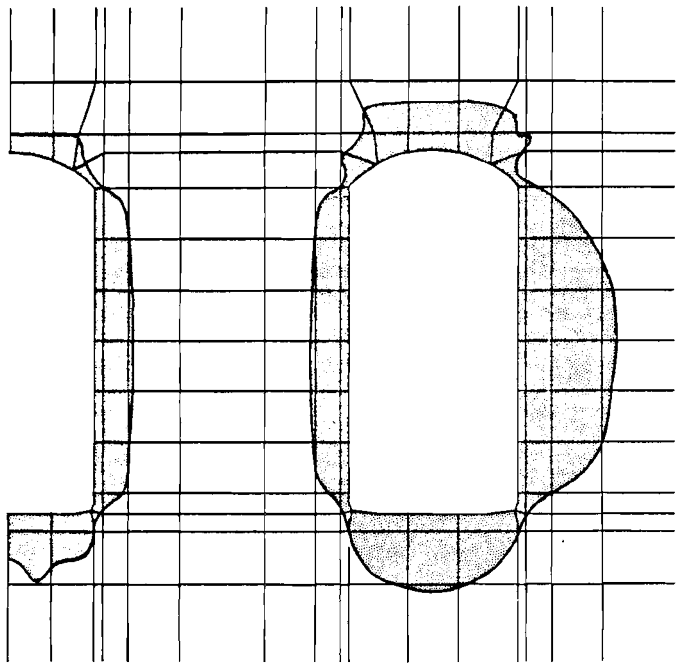

FIGURE 26. Failure Zones Around CAES Caverns After Initial Excavation. Depth 750 m, H/W = 2.0, In Situ Stress Ratio = 1.5 (Brandshaug and Fossum 1980)

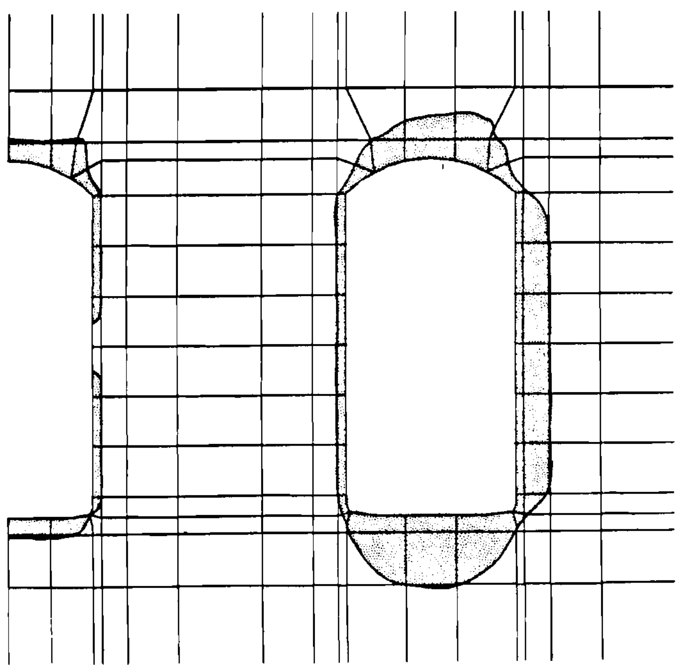

FIGURE 27. Failure Zones Around CAES Caverns After Initial Excavation. Depth = 500 m, H/W = 2.0, In Situ Stress Ratio = 1.5 (Brandshaug and Fossum 1980)

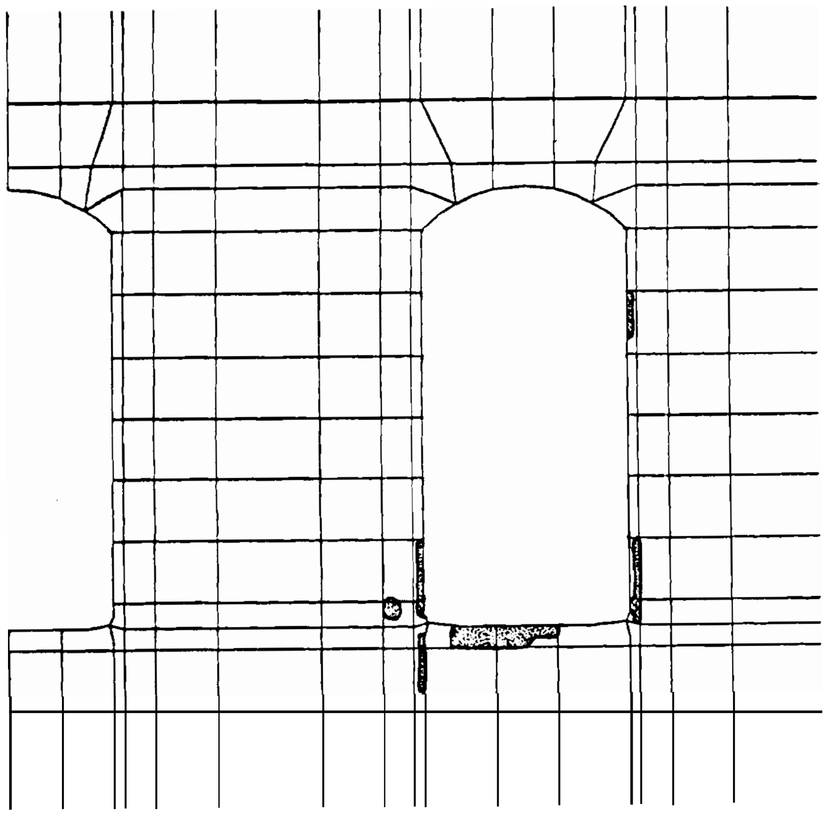

FIGURE 28. Failure Zones Around CAES Caverns During Operation. Depth = 750 m, H/W = 2.0, In Situ Stress Ratio = 1.5 (Brandshaug and Fossum 1980)

Rock bolts are used primarily to support blocks or pieces of rock at intersecting joints. This maintains sufficient fracture confinement to prevent failure by propagation. However, simulations of reinforcement measures indicated that artificial support such as rock bolts provided little or no increase in structural stiffness or strength.

Air leakage simulations performed using stress-dependent permeability relationships showed that although the in situ stress ratio had only minor influence on the volume of air leakage, the location of leakage in the cavern was strongly dependent on this ratio, with higher values giving rise to a greater loss of air through the cavern walls.

3.3.2 Post-Excavation Cavern Stability

The influences of cavern geometry and spacing and coefficient of lateral earth stress on the stability of caverns were evaluated (Gnirk et al 1979). The computed stress fields in the rock mass following excavation are applicable strictly to an elastic medium that is devoid of joints or other structural discontinuities. When inelastic deformation of the medium is not considered, or does not occur, the stress perturbation is, for all practical purposes, independent of the excavation sequence. In addition, the solutions are strongly statically determinate, implying negligible dependence on elastic properties.

The cavern excavation was assumed to be situated at a floor depth of 760 m, and the vertical in situ stress gradient was taken to be 0.0269 MPa/m. Consideration was given to single caverns with height-to-width ratios ranging from 0.5 to 3.0, and to a sequence of three caverns in parallel with height-to-width ratios of 1, 1.5, and *2* and extraction ratios of 20, 40, and 60%. In all instances, the cavern width was chosen as a constant 10 m, with the radii of the arched crown and rib-floor intersections 10% of the cavern height and 5% of the stress algebraically positive.

3.3.2.1 Single Caverns

Figure 29 illustrates the influence of height-to-width ratio (H/W) and in situ stress ratio (K_o) on the stress concentration factor in the central rib boundary (side wall) of a single cavern (Gnirk et al 1979). The stress concentration factor is defined as the ratio of the tangential (in this case, vertical) stress in the rib and the in situ (pre-mining) vertical stress. Clearly, as both H/W and K_o increase, the vertical stress in the rib changes from compression to tension.[a] For high

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[a]As a point of qualitative validation of the results given in Figure 29 as regards caverns stability, we draw attention to the Flygmotor CAES caverns in Sweden which have a height to width ratio of approximately 0.3 and are situated at a depth of about 90 m (Bergman et al. 1979). For the generally high horizontal in situ stresses-at that depth the ribs of the caverns have remained exceptionally stable over many years of daily operation. The temperature fluctuations in the caverns are nominal. For this case history situation, the boundary stress concentration in the rib is probably compressive, but of relatively low magnitude.

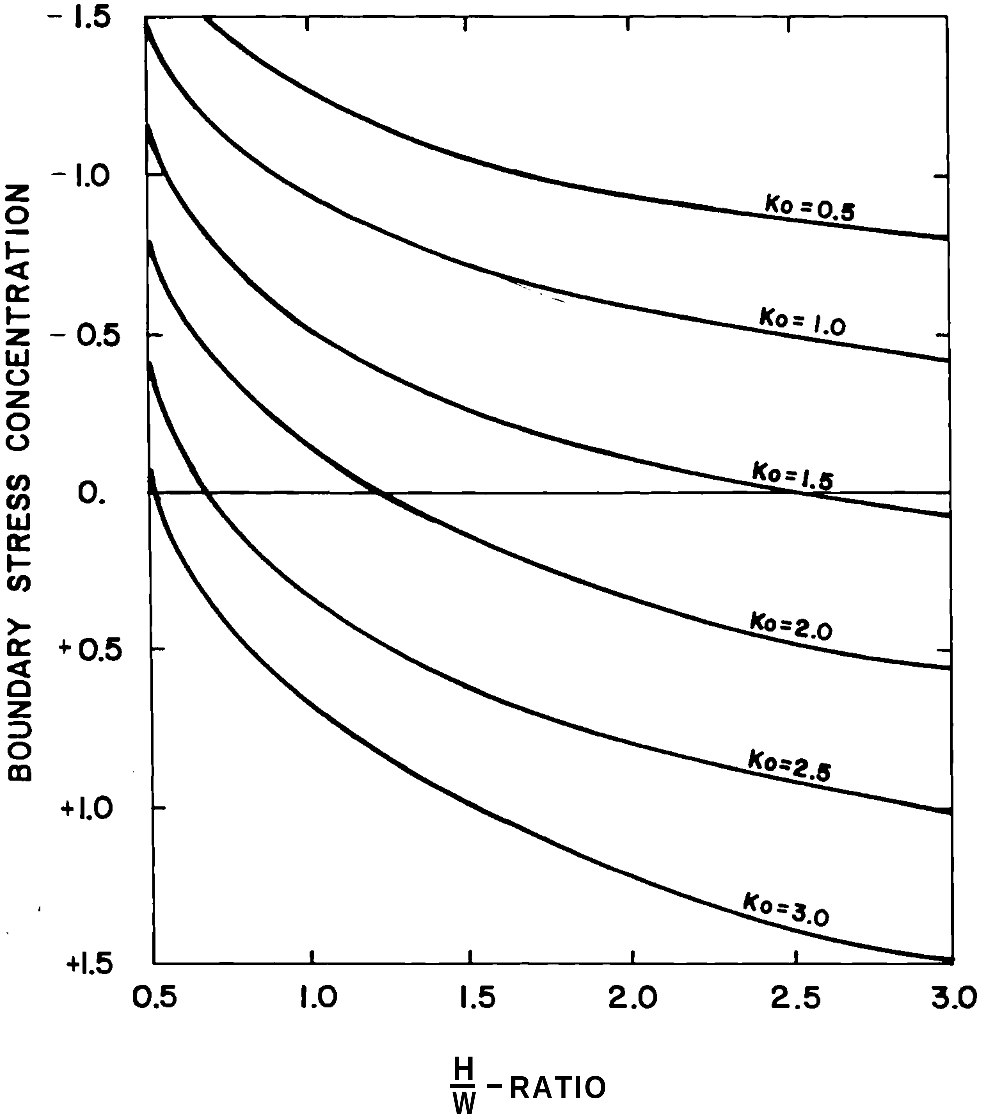


FIGURE 29. Boundary Stress Concentration in the Rib of a Single Cavern for a Range of Height-to-Width (H/W) and In Situ Stress (K_o) Ratios (Gnirk et al 1979)

values of K_o, the induced arching action above and below the cavern tends to "stretch" the rib. This stretching action will give rise to zones of predominant air leakage in the rib sections of the cavern.

Figure 30 illustrates the application of the elastic stress results to practical design considerations in terms of need for artificial support. We have plotted the ratio of the maximum compressive boundary stress in the cavern roof and the uniaxial compressive strength of a competent igneous rock (200 MPa) to K_o for a range of values of H/W. Superimposed on this plot are the artificial support needs, as deduced from correlations by Hoek (1979) based on extensive case history data. For an H/W ratio of 1.5 to 2, which is of interest to CAES cavern design, the in situ stress ratio must be less than 1.3 to 1.5 in order to avoid the use of heavy artificial roof support, such as blocked steel sets and concrete arches.

This may be a fairly conservative assessment as there are many stable openings in this range having only light support. A CAES site should not be eliminated on the basis of this assessment; however, support requirements may be significant, and will be determinable only through site-specific study.

3.3.2.2 Multiple Caverns

Figure 31 illustrates the influences of extraction ratio and in situ stress ratio on the vertical stress concentration in the rib of the central cavern in a three-cavern array, for a cavern height-to-width ratio of 2 (Gnirk et al 1979). For extraction ratios of 20 to 40% and a range of K_o from 1 to 2, the vertical pillar stress is compressive and on the order of 50 to 140% of the in situ vertical stress. For extremely

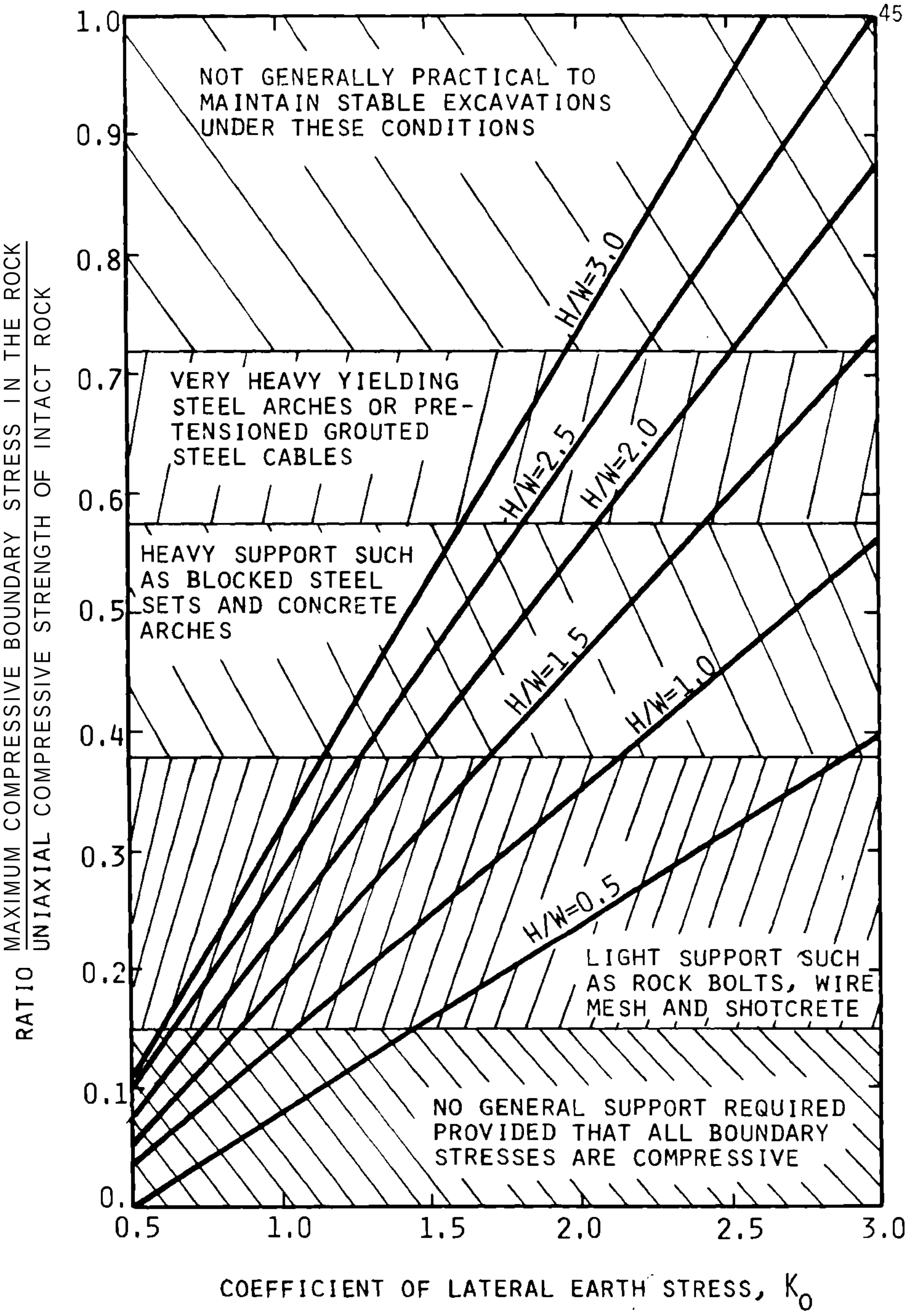


FIGURE 30. Post-Excavation Stability of a Single Cavern in Competent Igneous Rock at a Depth of 760 m (Gnirk et al 1979)

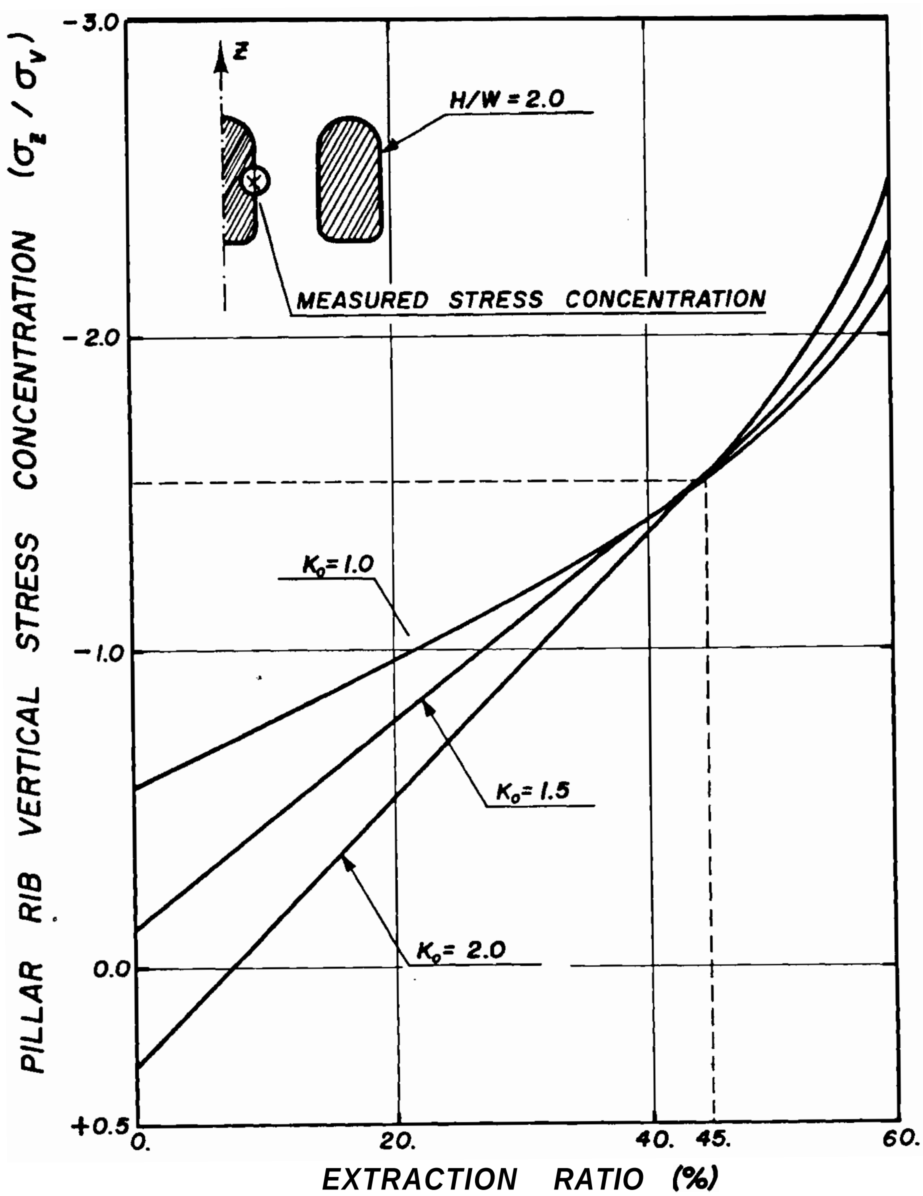


FIGURE 31. Boundary Stress Concentrations in the Pillar Rib of the Central Cavern in a Three-Cavern Array (Gnirk et al 1979)

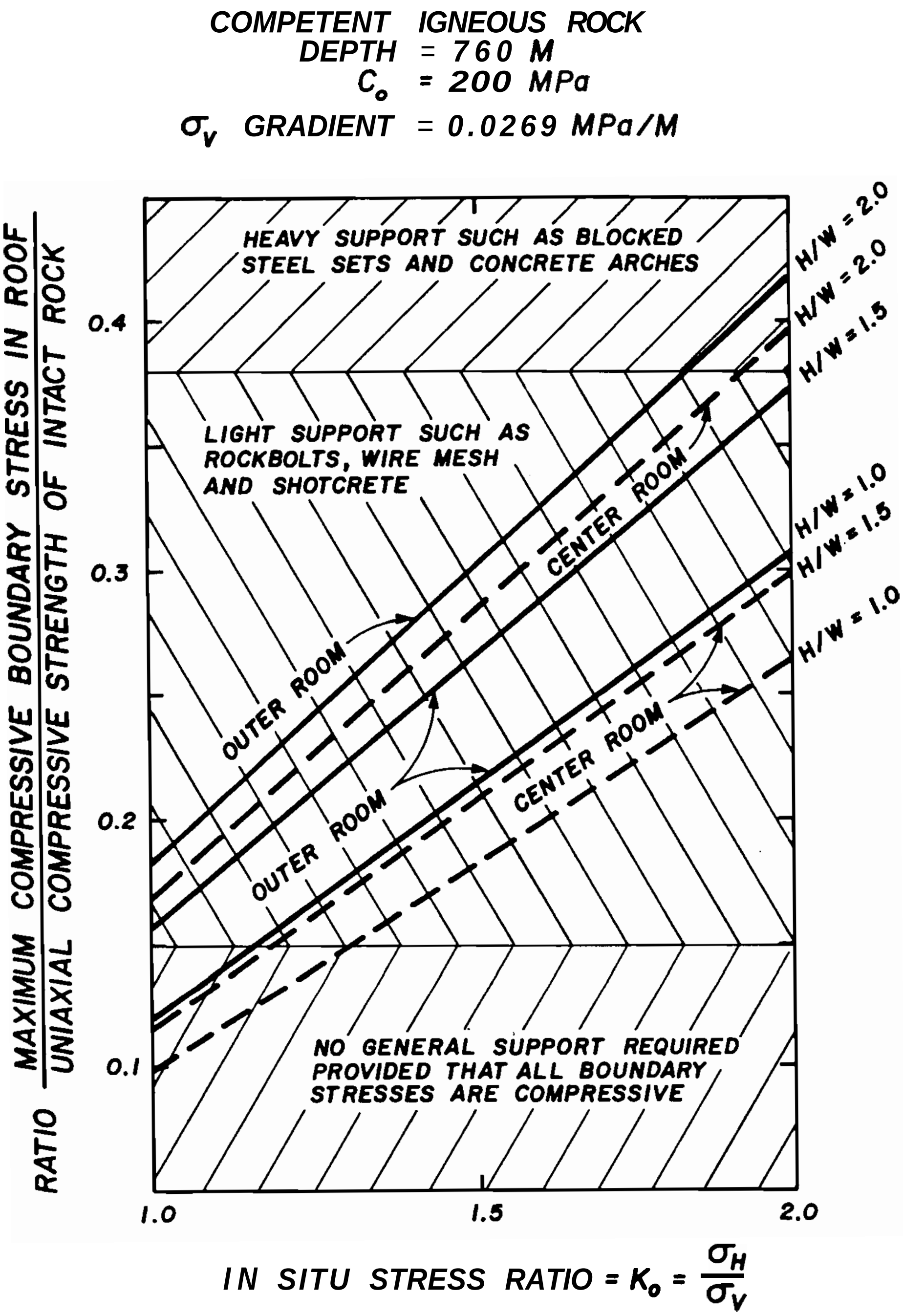


FIGURE 32. Post-Excavation Stability of an Array of Three Caverns in Competent Igneous Rock at a Depth of 760 m (Gnirk et al 1979)

high horizontal stresses, such as may occur at shallow depth, one could anticipate a vertical tensile stress in the pillar rib.[a]

Figure 32 combines the results for the stress concentrations in the cavern roofs with the uniaxial compressive strength of a competent igneous rock to illustrate the need for artificial roof support for ranges of H/W and K_o and a constant extraction ratio of 40%. The artificial support needs have been deduced, as before, from case history correlations by Hoek (1979). For an H/W of 1 to 2, only light support in the form of rock bolts and wire mesh is required to maintain cavern roof stability after excavation, for a value of K_o between 1.25 and 1.75.

The preceding results clearly indicate the need for site-specific in situ stress data for the design of CAES caverns. A slight alteration in cavern geometry in relation to the artificial support needs can be perhaps an economic trade-off, and deserves thoughtful attention from the viewpoints of rock stability and air leakage. In general, the results indicate that enhanced rock stability can be achieved with an array of caverns as compared to a single cavern, at least from the viewpoints of pre-operational considerations for CAES in hard rock.

3.3.3 Thermal Penetration Into Cavern Wall

To investigate the cyclic thermal effects in the rock during CAES operation, a cavern was chosen with a height and width of 20 m and 10 m, respectively (Gnirk et al 1979). The compensated cavern was considered to be initially full of water at 10°C. The top 2 m were assumed to

[a] As a point of qualitative validation, we draw attention to the case history described by Anttikoski and Saraste (1977) for three oil storage caverns in parallel near Helsinki. The caverns have a H/W of 2 and an extraction ratio of 30 to 40%, and are situated in an in situ horizontal stress field of about 15 MPa (K_o = 10 to 15). Through a combination of field observation and finite element analysis, they concluded that perhaps 60 to 90% of the pillars were in a state of tension and the cavern roofs were heavily stressed in compression. The formation of horizontal fractures across the pillars was deduced from the adjustment of oil levels between adjacent caverns to a common elevation.

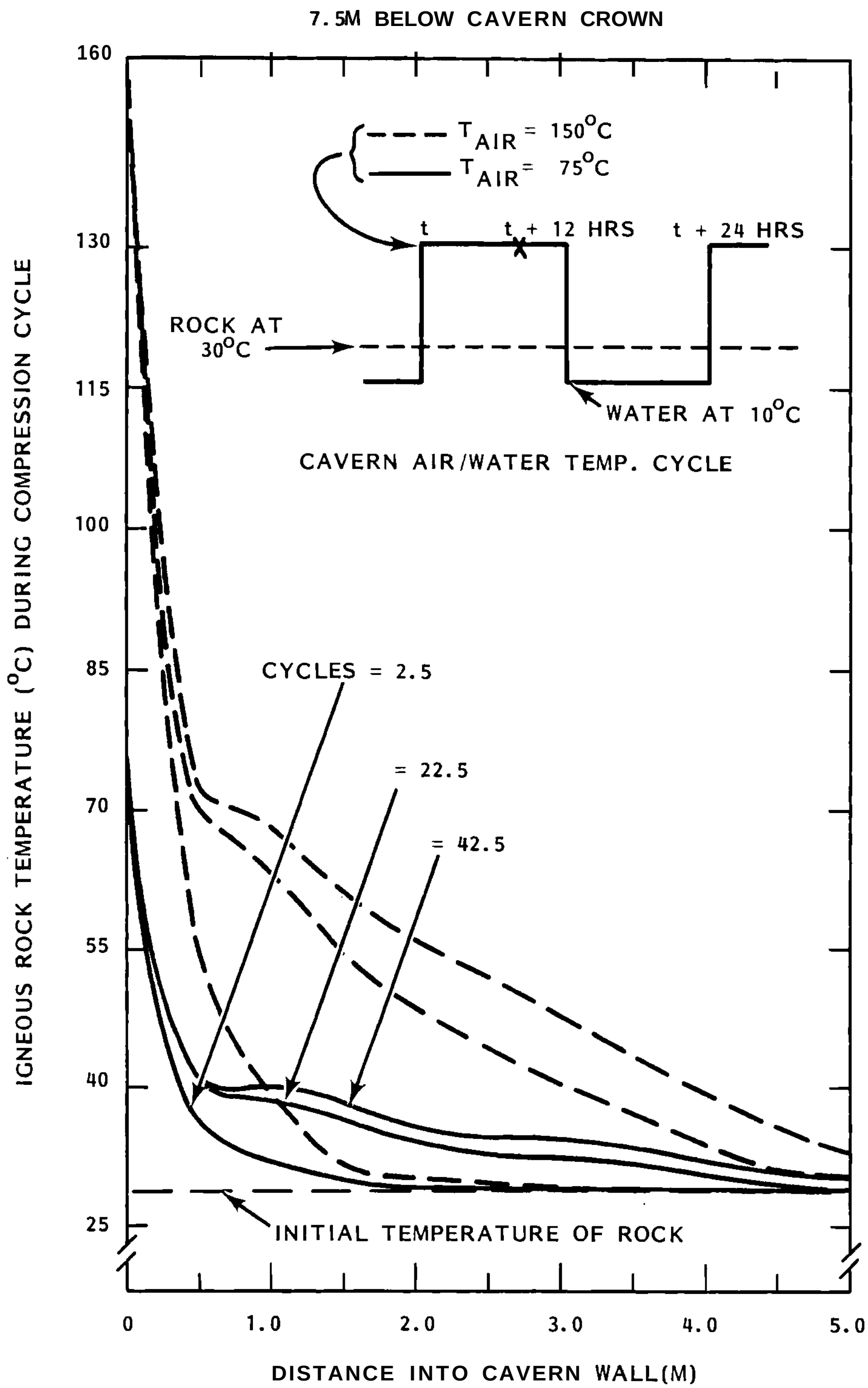


FIGURE 33. Temperature Distribution in a CAES Cavern Wall in Igneous Rock During the Compression Cycle (Gnirk et al 1979)

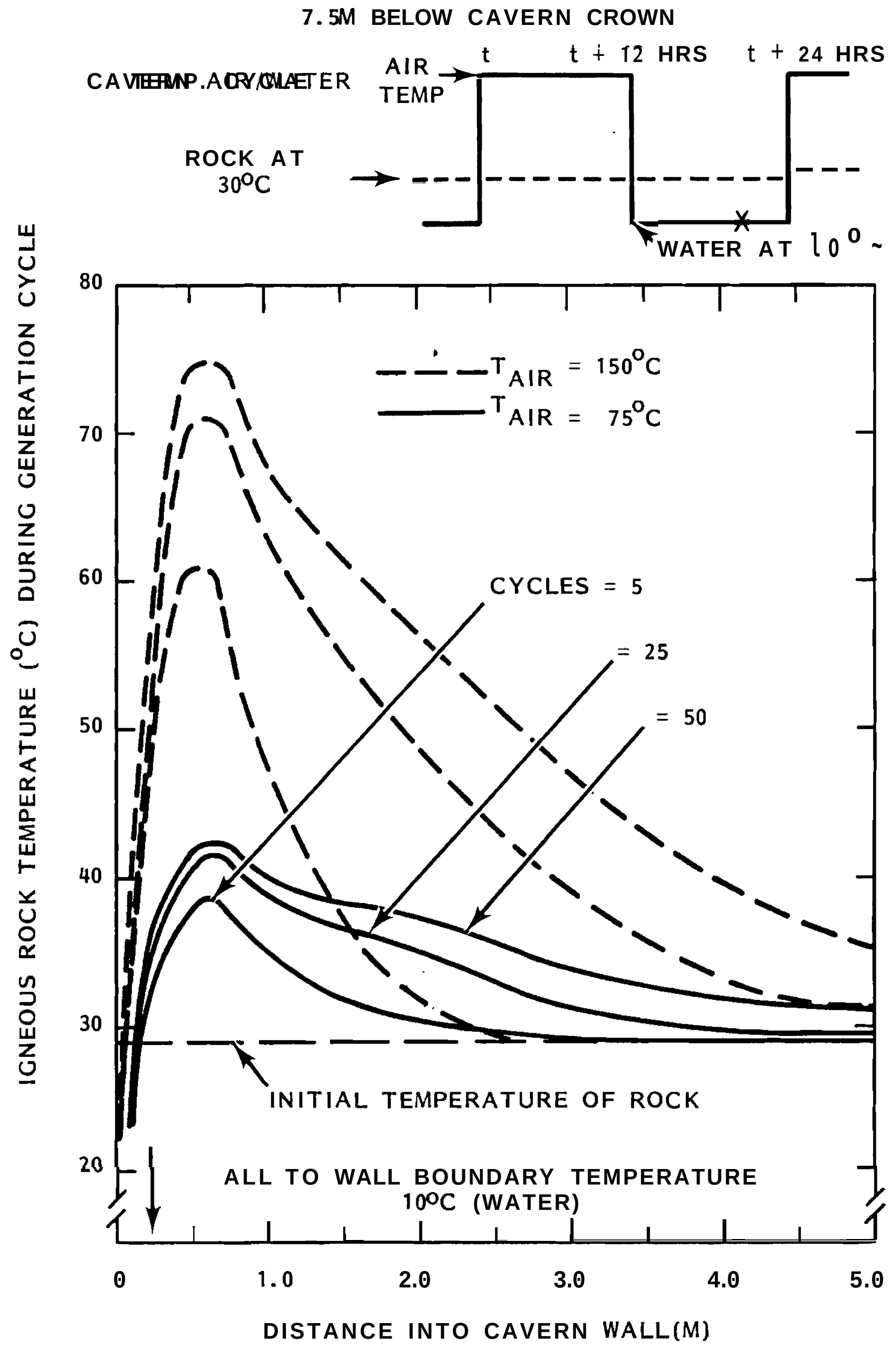


FIGURE 34. Temperature Distribution in a CAES Cavern Wall in Igneous Rock During the Generation Cycle (Gnirk et al 1979)

contain compressed air at all times and the bottom 0.5 m to contain water at all times. Thus, the operational height of the cavern over the compression and generation cycles was 17.5 m. The cycle time was taken to be one day, with the compression and generation cycles separated into alternating 12-hour periods. The air/water interface was supposed to move at a constant velocity over the operational height of the cavern. Compressed air inlet temperatures of 75°C and 150°C were considered. The cavern was assumed to be 750 m deep and the geothermal gradient was taken to be 25°C/km. Finally, the rock type was chosen to be igneous, with the following typical thermal properties: thermal conductivity = 2.9 W/mK; specific heat = 935 J/kgK; thermal diffusivity = 1.3 E-06 m^2/s.

Figures 33 and 34 illustrate the temperature distributions along a horizontal line into the rock 7.5 m from the crown of the cavern. The cavern wall (depending on elevation) is defined to be at the water temperature (10°C) or the compressed air temperature (75°C or 150°C) throughout the compression and generation cycle periods. The temperature in the first meter of the cavern wall is seen to rise substantially within the first five cycles of cavern operation. The region beyond one meter is relatively unperturbed by the cyclic temperature boundary on the cavern wall and undergoes continuous heating. The temperature rises between the twentieth and fiftieth cycles are much less. At the fiftieth cycle, temperature increases are quite small and the first meter of the cavern wall is near a steady oscillatory temperature distribution with small temperature increases.

If the initial temperature of the material is the same as the mean of the oscillatory surface temperature, the temperature distribution reaches a steady oscillatory motion much more rapidly than if the initial temperature of the medium is above or below the mean of the harmonic surface temperature. For the CAES system discussed above, the analytical solution indicates that the transient portion of the solution will contribute to thermal oscillation within five meters of the cavern wall for the life of the cavern. However, the majority of the transient is damped within the first 100 cycles.

One of the more important aspects of this simulation is the location and magnitude of the thermal gradients. The largest thermal gradients occur early in time and within the first meter of the cavern wall, indicating that this region will be the most critical in terms of thermally induced stresses (Brandshaug and Fossum 1981). High cyclic thermal gradients persist in the cavern walls as operation continues. It appears that only the first meter into the cavern wall will experience oscillating temperatures. The oscillating temperatures approach a steady behavior after 50 compression/generation cycles. Thermal gradients in the roof and floor diminish as operation continues. For the time span analyzed (50 days), the thermal response of the rock is similar for a single cavern and an array of three caverns.

For the single CAES cavern the thermally-induced stresses in the wall show a cyclic tension-compression behavior with tensile stresses approaching the tensile strength of igneous rock. For the array of three caverns no tensile stresses occurred. Although cyclic stresses are present also in the case of multiple caverns, the state of stress is cyclic compression-compression,' a state that is preferable to tension-compression with regard to fatigue failure. Thus, from a stability standpoint, a multiple cavern layout is preferred. The cyclic temperature-induced stresses appearing in the cavern walls may cause degradation of the rock, affecting its strength to a degree that may compromise the structural integrity of the caverns. Any structural failure is, of course, dependent on the initial rock quality and may be eliminated or contained within limits by choosing a host rock possessing an acceptable structure and sufficient strength.

For the conditions modeled, the analysis predicting inelastic behavior or rock failure shows that such failure develops strictly along joints and is fully developed upon completion of the initial cavern excavation. Even substantial strength reduction of the rock because of fatigue or thermal shock during operation does not alter or add to the failure already existing after cavern excavation. The internal cavern pressure during operation has the ability to unload the rock structure surrounding the caverns. This explains why failure during operation does not progress beyond that predicted after initial cavern excavation.

Rock failure predicted around caverns located at 500 m depth is approximately half of that experienced at 750 m depth, all other conditions being equal. Rock failure increases as the coefficient of lateral earth stress decreases. Lowering the cavern height-to-width ratio from two to one has the effect of decreasing the failure zones, especially around the outer caverns.

3.3.4 Air Leakage

Air leakage simulations using stress-dependent permeability relationships showed that the coefficient of lateral earth stress had only minor influence on the volume of air leakage. However, the location of leakage in the cavern is strongly dependent. on the in situ stress ratio with the higher value giving rise to a greater loss of air through the cavern walls.

3.3.4.1 Evaluation Procedures

The assessment of air leakage from, or water inflow into, a CAES cavern is commonly based on the theory of fluid flow in porous or fissured media, with invariant rock and fluid properties. For fissured media, the aperture width and spacing of the fissures must be specified. These types of data are clearly site-specific, which implies that a fissured medium for generic purposes must be treated from the viewpoint of an equivalent permeable medium. In this respect, the hydraulic conductivity, K, of the medium can be expressed in terms of stress, and the fluid viscosity in terms of temperature:

$$K = f(\sigma,\mu) = k\rho g/v$$

where

K = hydraulic conductivity

μ = fluid viscosity (function of temperature)

a = stress (function of temperature)

k = intrinsic permeability

ρ = density

g = gravitational constant

v = kinematic viscosity.

By developing a functional form of the equation from laboratory and field data, one can evaluate the perturbation in the rock mass permeability arising from excavation and cyclic temperatures in a CAES cavern (Gnirk 1979).

The development of the functional form of the equation is based on in situ hydraulic conductivity tests over a range of depths in boreholes, and a knowledge of the in situ stress state and joint set orientation in a particular locality. In general, the horizontal permeability of a rock mass, measured in field tests with vertical boreholes, decreases with increasing depth. This phenomenon may also be incorporated into the function form of the hydraulic conductivity relationship. Additionally, a vertical hydraulic conductivity relationship can be developed in situations where inclined boreholes are used.

The hydraulic conductivity of the rock mass following construction of the CAES cavern and during operation can be assessed in the above manner. However, the amount of air leakage cannot be evaluated without consideration of the degree of saturation of the rock mass.

For an uncompensated cavern to be air-tight, the undisturbed surrounding rock mass prior to construction must be saturated with water at higher pressure than the operating air pressure within the cavern, creating water flow toward the cavern. This condition may require use of a water curtain. In a water-compensated cavern the hydrostatic heads of the water column and the saturated zone will be nearly identical.

3.3.4.2 Enhancement of Cavern Tightness

Air leakage from a CAES cavern can be substantially reduced by:

1) implementation of a water curtain in the rock mass over the'cavern;
2) grouting of discontinuities in the rock mass surrounding the cavern;

The first technique involves a method for ensuring continued saturation of the rock mass during CAES cavern construction and operation. The second procedure effectively reduces the gross permeability of the rock mass during CAES cavern construction and operation. The first technique was used in an underground Swedish LPG facility (Lindblom 1977).

3.4 CAVERN EXCAVATION

Adequate information is available to design an underground opening with the size, shape, and depth required of a CAES cavern with regard to its excavation and construction, based on a combination of numerical modeling and precedent considerations (Brandshaug and Fossum 1980). The design methodology used in the referenced study for the excavation phase of a CAES cavern can provide a guide in the evaluation of support requirements for caverns of different shapes when excavated in a rock mass with relatively well quantified rock properties and geotechnical characteristics. Since experience is non-existent or limited on the behavior of large rock masses subjected to the operating conditions of CAES caverns, it is recommended that numerical methods be used in combination with in situ tests in developing a cavern design methodology.

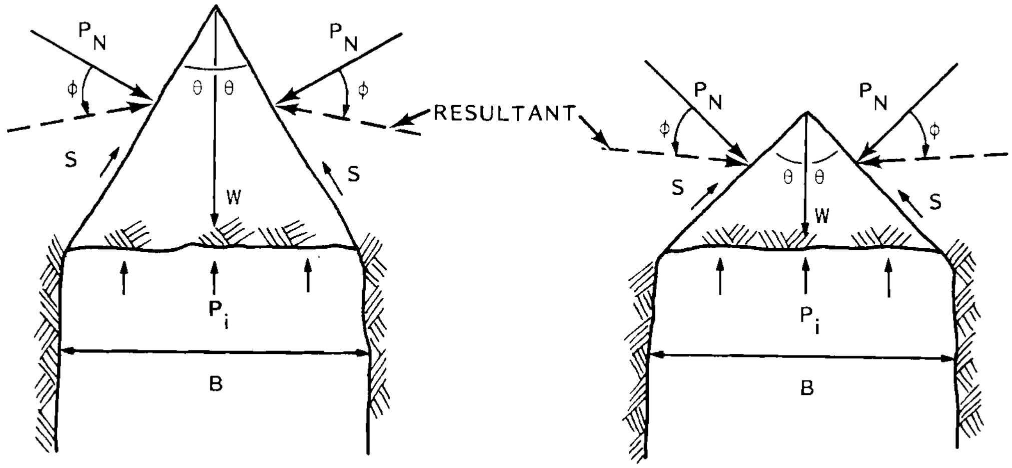
The method of excavation/construction is influenced by a combination of economics, geotechnical considerations, and contractor availability. The underground layout and cavern shape are also influenced by cost factors. In conventional heading and benching operations, excavation of headings is more costly than excavation of benches. Thus, from this standpoint, cavern height-to-width ratios should be as large as possible. Also, a single cavern may be more economical than multiple caverns of the same volume. However, additional consideration must be given to the length of haulageways, ventilation requirements, utility locations, and schedule leading to an optimal design.

3.4.1 Excavation Stability

We shall assume that the geotechnical and thermal/mechanical properties of a rock mass for a potential CAES site can be properly characterized and quantitatively defined (Gnirk and Port-Keller 1978). For a choice of cavern depth, shape dimensions, and spacing, we may compute, by say the finite-element method, the state of stress in the rock mass during a simulated excavation. The appropriate criterion for evaluating the surrounding stability of the excavation is the Mohr-Coulomb failure criterion. This criterion, which mathematically relates the tensile and compressive strengths of a rock to a state of applied stress, permits evaluation of the potential for incipient rock failure. Specifically, if the state of stress at some location around the excavation violates the Mohr-Coulomb criterion, failure within the rock mass is indicated. The criterion is applicable to both the intact rock and the joints, with appropriate characterization of the strength properties in each case, and to a failed rock mass in the sense of residual strength. By altering the cavern geometry and dimensions, and the sequence of excavation, the cavern stability may be effectively optimized for a given rock mass and state of in situ stress.

The actual measure of instability resulting from some amount of failure must be quantified in terms of loss of cavern serviceability. Regions of failure within the rock mass around the periphery of the cavern, as indicated by finite-element modeling, do not necessarily imply general instability if the regions are localized and reasonably disconnected, or can be "hardened" by use of artificial support. Thus, a certain degree of "contained" rock failure may be acceptable in the sense that the future serviceability of the cavern is not impaired.

The maximum piece of rock that must be stabilized in the crown of an underground opening depends directly on the nature and orientation of the discontinuities and the rock pressure across them. As illustrated in Figure 35, a block of rock will not fail if the rock pressure normal to the possible failure plane is great and the sum of the friction values (in degrees) is greater than the angle included between the



STABLE
WHEN $\theta < \phi$
AND $P_N \gg W$

$$P_i = P_N \left(1 - \frac{\tan \phi}{\tan \theta}\right) + \frac{\gamma B}{4 \tan \theta}$$

UNSTABLE
WHEN $\theta \geq \phi$
OR $P_N \ll W$

P_i = INTERNAL PRESSURE REQUIRED FOR STABILITY
P_N = NORMAL PRESSURE ON WEDGE
ϕ = ANGLE OF INTERNAL FRICTION
B = WIDTH OF OPENING

FIGURE 35. Rock-Wedge Stability (adapted from Cording et al 1971)

planes (Cording et al 1971). (Asperities will enhance the stability.) Rockbolts must be designed accordingly. At shallow depths, or where the rock pressure normal to the discontinuity planes is low, friction will not stabilize the block, regardless of its value.

Wall stability also depends on the orientation and strength of the discontinuities, and on the tangential and radial stresses around the opening. Stereographic, planar failure, and three-dimensional wedge analyses--techniques developed for surface vertical cuts--are applicable to vertical cuts underground if the blocks are bounded by discontinuities at the top and on the sides.

The general rock stability of a CAES cavern may be enhanced by a number of construction techniques, including: controlled excavation methods such as smooth blasting and presplitting; reinforcement methods such as rock bolting; and operational shakedown.

Clearly, the fractures generated in the solid rock during blasting act as planes of weakness and result in increased rock permeability and decreased rock mass strength (Gnirk 1979). Smooth blasting involves the use of an explosive with low charge concentration and a large number of closely spaced blastholes. This technique reduces back-break into the solid rock. Time-spread among ignitions along each row of holes should be minimized. In presplitting, cracks for the final contour are created by blasting prior to the drilling of the rest of the holes for'the blast pattern.

Operational shakedown of a CAES cavern is a relatively novel idea that appears to have no published precedence. Basically, the procedure involves initial cavern pressurization with air injection over extremely long cycles of temperature (compensated and uncompensated) and pressure (uncompensated). The cycle time is gradually increased to the desired operational cycle. The purpose of this procedure is to gradually strain-harden the rock over time and, in effect, to stiffen the structure before full-scale operation (Gnirk 1979).

Fracture control procedures can reduce the number of perimeter holes as compared with smooth blasting while maintaining equivalent or better control of overbreak and improved preservation of the structural integrity of the remaining rock (McKown and Thompson 1981). Geologic features such as joints, faults, or bedding planes can influence the perimeter control achieved with fracture control by arresting, diverting, or bifurcating the driven crack or by venting the explosive gases.

3.4.2 The Q-System in Design Decisions

Estimates of support are required at three stages in a project: for the feasibility studies, for the detailed planning and design, and finally during excavation itself (Barton et al 1980). In view of the potential economic importance of support costs the support estimates should be as accurate as possible for all three stages. The accuracy will depend partly on the effectiveness of the geological investigations, and partly on the ability to extrapolate past experiences of support performance to new rock mass environments. Underground excavations are constructed with some confidence primarily because of all their successful predecessors.

A practical method of extrapolating past experiences of support performance to new rock mass environments is the Q-system (Barton et al 1980). Several years experience by a number of users have shown it to be a useful aid in making design decisions. It has been used during feasibility and detailed planning work, and particularly during construction. The Q-system is essentially a weighting process in which the positive and negative aspects of a rock mass are assessed quantitatively by evaluating six factors: rock quality designation, number of joint sets, joint roughness, type of clay fillings, water inflow, and stress levels. Experience in 200 cases was used to find the most appropriate support measures, taking into account the rock mass quality, the excavation dimensions (span or height), and the safety requirements.

3.4.3 Decommissioning Stability

After CAES cavern operation ceases, consideration must be given to the eventual collapse of the cavity, leading to possible surface subsidence. It would appear that an appropriate evaluation of "long-term" stability would involve consideration of a creep rupture criterion in conjunction with the stress state around the cavern.

3.5 PRECEDENT STABILITY CONSIDERATIONS

This section treats cavern stability with respect to 1) recognizing factors that affect roof spans and 2) minimizing the impacts of joints and in situ stress.

3.5.1 Cavern Roof Span Determination

In 1976, a Symposium on Exploration for Rock Engineering was held in Johannesburg; one session was devoted to the relationship between the global stability of an underground cavern and the quality of the host rock mass. The quality of a rock mass is rated on a combination of geological, mechanical, and ground water factors. Each factor is numerically weighted according to a quantitative scale based on a range of favorable to unfavorable characteristics. The importance of a scale and its subdivisions is generally based on the experience and judgment of the particular author. Two papers relate to the dependence of the span (width) of an underground cavern on rock quality. This approach permits one to establish a reference with which to compare the results of numerical modeling efforts (Gnirk 1979).

As illustrated in Figure 36, Bieniawski (1976) utilizes six factors to classify a rock mass from a geomechanics viewpoint:

1. uniaxial compressive strength
2. drill core quality (RQD)
3. joint spacing
4. joint condition
5. ground water conditions
6. joint orientation

The upper and lower bounds of the relationship between the unsupported span width of a cavern and standup time (or period of stability) of the roof section are presented in Figure 37 for various ranges of rock quality, Q. The graph also includes case history results from Austria and South Africa, as well as the bounds of similar relationships that were developed in Austria and Scandinavia. In essence, Figure 37 indicates that a cavern span of 4 to 20 m will stand unsupported for one to 20 years in a virtually dry rock mass that exhibits joints with rough surfaces, apertures less than 1 mm, a generally favorable orientation, and a compressive strength, RQD, and joint spacing in excess of 100 MPa, 75%, and 1 m, respectively. In situ stress is not included, but probably reflected to some extent in the case history data points.

Barton (1976) uses basically the same parameters as described above for evaluating rock quality, as well as a stress reduction factor, which is defined as the ratio of the unconfined compressive stress to the major principal in situ stress. The parameters are combined as

A CLASSIFICATION PARAMETERS AND THEIR RATINGS

	PARAMETER		RANGES OF VALUES						
1	Strength of intact rock material	Point load strength index	> 8 MPa	4 - 8 MPa	2 - 4 MPa	1 - 2 MPa	For this low range — uniaxial compressive test is preferred		
		Uniaxial compressive strength	> 200 MPa	100 - 200 MPa	50 - 100 MPa	25 - 50 MPa	10-25 MPa	3-10 MPa	1-3 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core quality ROD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of joints		> 3 m	1 - 3 m	0.3 - 1 m	50 - 300 mm	< 50 mm		
	Rating		30	25	20	10	5		
4	Condition of joints		Very rough surfaces. Not continuous separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Soft joint wall rock	Slickensided surfaces OR Gouge < 5 mm thick OR Joints open 1-5 mm Continuous joints	Soft gouge > 5 mm thick OR Joints open > 5 mm Continuous joints		
	Rating		25	20	12	6	0		
5	Ground water	Inflow per 10m tunnel length	None		< 25 litres/min	25 - 125 litres/min	> 125 litres/min		
		OR Ratio joint water pressure / major principal stress	0		OR 0.0 - 0.2	OR 0.2 - 0.5	OR > 0.5		
		OR General conditions	Completely dry		OR Moist only (interstitial water)	OR Water under moderate pressure	OR Severe water problems		
	Rating		10		7	4	0		

B. RATING ADJUSTMENT FOR JOINT ORIENTATIONS

Strike and dip orientations of joints		Very favourable	Farourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 20
Class No	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK MASS CUSSES

Class No	I	II	III	IV	V
Average stand-up time	10 years for 5 m span	6 months for 4 m span	1 week for 3 m span	5 hours for 1.5 m span	10 minutes for 0.5 m span
Cohesion of the rock mass	> 300 kPa	200 - 300 kPa	150 - 200 kPa	100 - 150 kPa	< 100 kPa
Friction angle of the rock mass	> 45°	40° - 45°	35° - 40°	30° - 35°	< 30°

FIGURE 36. Geomechanics Classification of Jointed Rock Masses (after Bieniawski 1976)

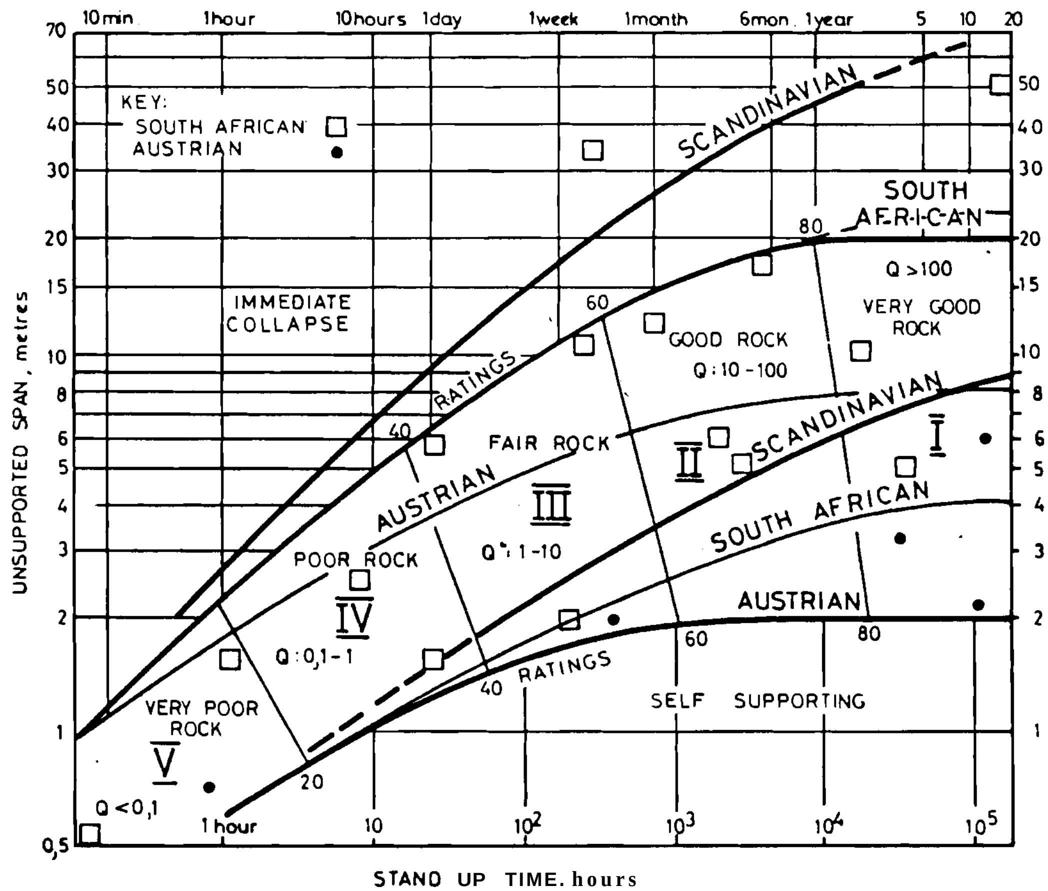
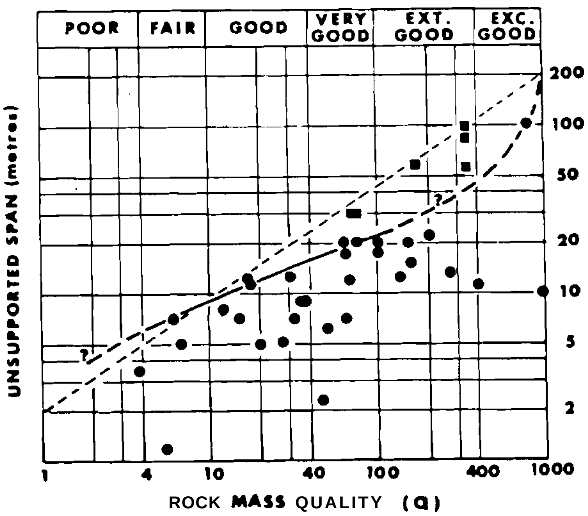
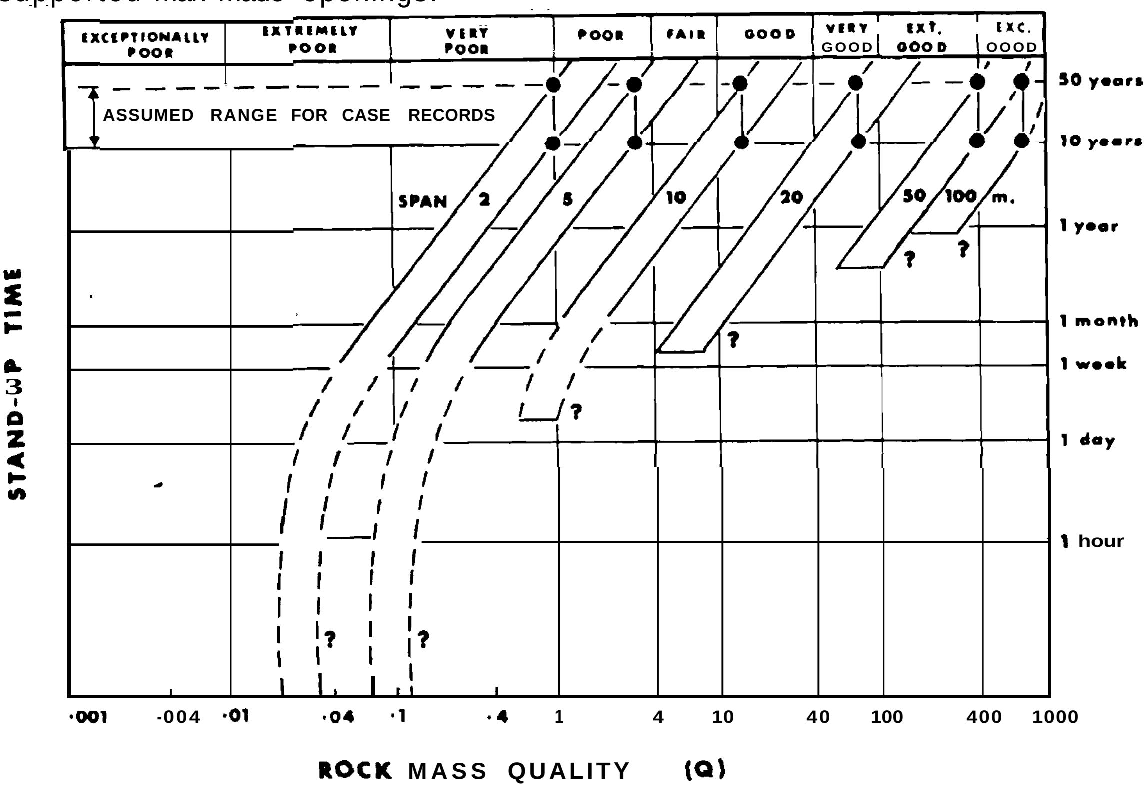


FIGURE 37. Unsupported Span of an Underground Opening as a Function of Standup Time in Relation to Various Geomechanics Classification Systems (after Bieniawski 1976)

pairs into three factors that involve crude measures of relative block size, interblock shear strength, and active stress. The product of these factors provides a measure of rock quality, Q. Figure 38 illustrates the relationships between unsupported span width of a cavern, standup time, and rock quality. A quantitative comparison of Barton's relationship with that of Bieniawski is difficult, except to observe that the predictions by the latter author's procedure appear to be more conservative.



Unsupported Span as Function of Rock Mass Quality. Circles represent man-made openings and squares represent natural openings from Carlsbad, NM. Curved envelope is an estimate of the maximum design span width for permanently unsupported man-made openings.



Standup Time of Unsupported Openings as a Function of Rock Mass Quality. Envelopes represent a preliminary attempt to predict reduction in standup time when span is increased beyond maximum design span.

FIGURE 38. Unsupported Span of an Underground Opening and Standup Time as Functions of Rock Mass Quality (after Barton 1976)

Walia and McCreath (1977) have employed Barton's rock mass quality rating system to define cavern spans and heights for CAES. For Q > 100, which typifies massive igneous and metamorphic rocks with relatively few joints and minor water inflow, they suggest cavern heights of 30 m and spans of 15 to 21 m. For less favorable joint properties and water inflow characteristics, as may be exhibited by limestones and dolomites (15 < Q < 100), they propose cavern heights to 20 m and spans of 10 to 15 m.

3.5.2 Influence of Horizontal Stress on Cavern Design

The influence of a relatively high horizontal stress on cavern stability is discussed by Anttikoski and Saraste (1977) in a case history for a region near Helsinki. Three oil storage caverns, each with a height of 28 m, a width of 14 m, and a capacity of about 83,000 m^3, were constructed in 1972-73 at a depth (floor) of approximately 48 to 55 m in migmatitic bedrock. The bedrock is composed of alternating layers of gneiss-amphibolite and granite with a dip of about 75°. Horizontal fissures are present, as well as a set of inclined intersecting joints with dips of 45 to 50°. The caverns were oriented nearly parallel to the strike of the bedrock (which paralleled the strike of the inclined joints), and excavated downward in one or two benches from a pilot heading.

During excavation of the final bench, and particularly near the intersection with a cross-cut tunnel, horizontal fissures appeared in the pillars, along with some vertically-oriented fractures in the roof and localized minor rock bursts. In situ stress measurements indicated a maximum horizontal stress of about 15 MPa in the bedrock, with an orientation that was almost perpendicular to the longitudinal axis of the cavern system. Through a combination of field observation and finite-element analysis, it was finally concluded that perhaps 60 to 90% of a pillar between caverns was in a state of tension, while the cavern roofs were heavily stressed in compression.

In spite of the high horizontal stress, the structural rock stability of the cavern system was not compromised, even after an operational period of some five years. However, the fracturing in the pillars did noticeably enhance the leakage of water into the caverns. The rock mass around the caverns was grouted, which reduced the water inflow by about 50%. Grouting was more effective in reducing water leakage from the pillar regions (tension zones), than from the cavern roof regions (compression zones). During use of the cavern system for oil storage, oil levels between adjacent caverns tend to adjust to a common elevation, indicating communication along open horizontal fissures in the pillar.

This case history illustrates a rather severe consequence of high horizontal in situ stresses. The in situ stress ratio was about an order of magnitude greater than normal for a depth of 700 to 1200 m. The primary result was enhancement of water leakage through the tension induced horizontal fissures.

3.5.3 Design Based on in Situ Stress and Joint Orientation

The general design procedures used by Norwegian engineering geologists, as summarized by Selnier-Olsen and Broch (1977), provide some interesting guidelines for the design of stable caverns in hard rock. With regard to cavern orientation, they recommend that the longitudinal axis be oriented:

- along the line that bisects the maximum intersection angle between the directions of the two dominating joint sets (including bedding planes and foliation partings), but not parallel to the direction of the third or fourth minor set (see Figure 39)
- at an angle of at least 25° from steeply dipping joint planes that are either smooth or filled with clay
- at an angle of 15 to 30° to the horizontal projection of the major in situ stress

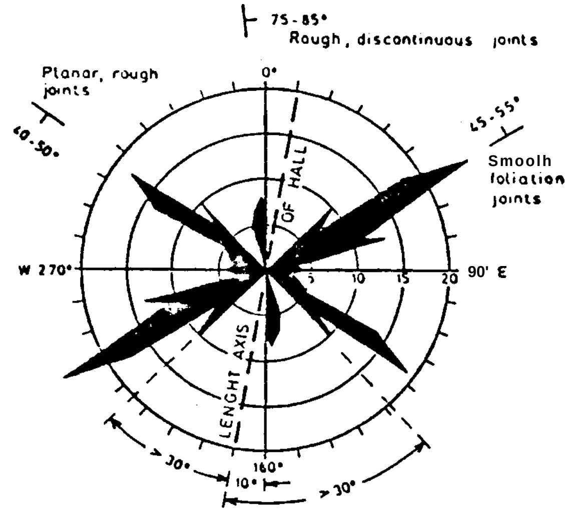


FIGURE 39. Joint Rosette Illustrating the Orientation of the Longitudinal Axis of an Underground Cavern for Minimization of Stability Problems (after Selmer-Olsen and Broch 1977)

- at a maximum angle, and at least 35°, to the strike of the planes of foliation or bedding when the major in situ stress direction parallels the strike.

From the viewpoint of the cross-sectional shape of a cavern, Selmer-Olsen and Broch (1977) suggest three guidelines based on Norwegian experience and rock mass conditions (generally igneous and metamorphic):

1. for the caverns at shallow and intermediate depths, unsupported roof spans of up to 10 m may be used if the bedding thickness between partings is at least 1 m or more and orientation with respect to joint plane directions is properly considered
2. for the conditions of 1) above and a bedding thickness of less than 0.5 m, the roof is normally profiled with a high arch
3. for greater depths, small curvature radii in the periphery of the cavern should be avoided.

Figure 40 illustrates preferred cavern shapes for various in situ stress directions and relative stress levels. Selmer-olsen and Broch also note that steeply dipping discontinuities in rock masses particularly influence the stability of cavern walls, while horizontally situated discontinuities are of concern to roof stability.

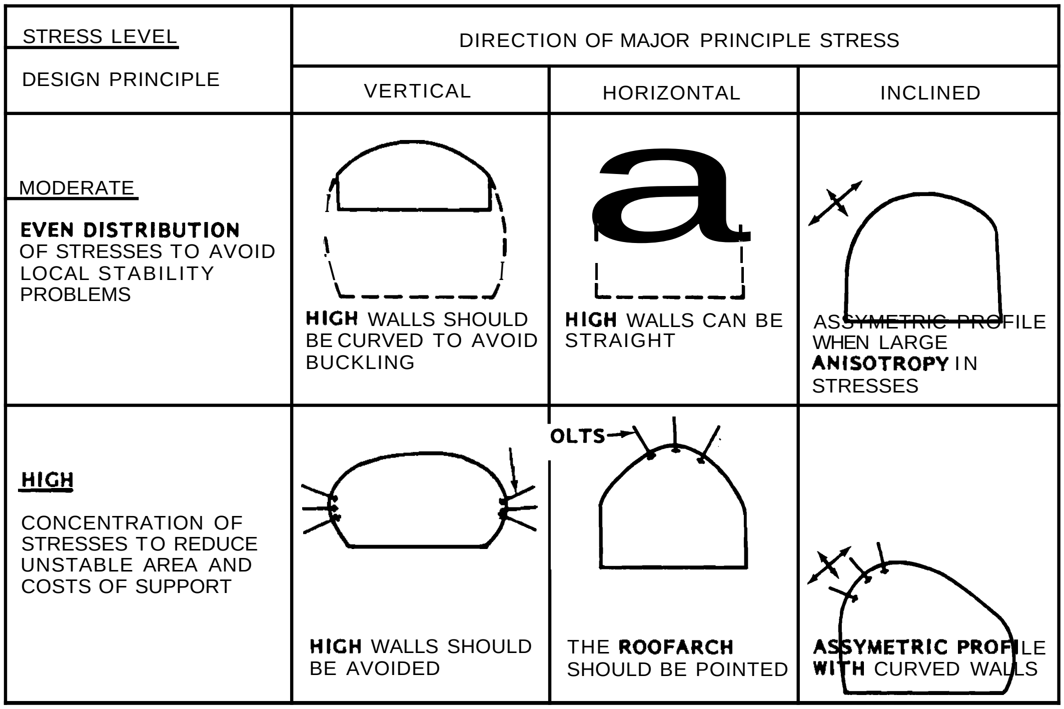


FIGURE 40. Design Principles for Qualitative Determination of Cavern Shape for Varying Stress Levels and Varying Directions of the Maximum In Situ Stress When the Direction of this Stress is Oriented Perpendicular to the Longitudinal Axis of the Cavern (after Selmer-Olsen and Broch 1977).

4.0 SITE QUALIFICATION

The depth at which a CAES cavern is excavated depends upon operating pressure, water table depth, lithology, and hydrostatic compensation requirements. The cavern walls must remain stable under the tensile hoop stresses imposed by the conipressed air (Howells 1977). Because of turbine machinery limitations and pressure losses during the generation cycle, a maximum air pressure of about 7.0 to 7.5 MPa is required. This operating pressure is reached at approximate depths of 715 to 765 m (Brandshaug and Fossum 1980; Gnirk and Fossum 1980). Because leakage to the surface in water-saturated rock is less than in dry rock, no lowering of the ground water table should be allowed (Electric Power Research Institute 1975).

Site qualification must begin with regional evaluation and selection of areas for detailed field study.

Field mapping and seismic surveying are to be followed by drilling, coring, and laboratory analysis to define geologic conditions in the overburden and host rock. Rock mechanical properties and near region ground water characteristics must be determined. It is important to recognize zones of weathering, leaching, jointing, fracturing, and faulting. The frequency, distribution, extent, and orientation of these zones of weakness will strongly influence site qualification.

The high costs associated with development of a CAES cavern demand comprehensive geotechnical evaluation of the host rock. Ground conditions must be determined precisely enough so that unexpected conditions encountered during excavation will be of minor consequence (Hansen and Lachel 1980). Ground conditions are a major influence on excavation cost. For example, shafts cost less to sink in tight, dry ground than in highly pervious ground below the water table, all other factors being equal. The appropriate mining method also depends on the ground conditions.

Engineering design requires that parameters of ground strength, deformability, and permeability be determined, in addition to the loads that will be applied during mining. The degree of accuracy required in determining each parameter depends on the individual case. For example, the strength of massive rock in which openings of small cross-sectional area are to be driven at shallow depths needs to be resolved only within several MPa. In such a case, the rock strength is unlikely to be of great economic importance unless the rock is very weak. Large openings at great depths require much more accurate appraisals of rock strength and deformation characteristics. Ground water conditions are important at any depth (Hansen and Lachel 1980).

The ground condition evaluation must include determination of the engineering properties of the host formation. This involves description of the structural geology as it relates to rock strength, i.e., the frequency, extent, and orientations of discontinuities such as faults, shear zones, joints, and bedding planes.

Geotechnical parameters also must be established for use in physical and mathematical mining model studies. The goal of geotechnical evaluation is to predict ground behavior during excavation and during CAES operations.

Gross features, such as major folds, are defined first; then a series of increasingly detailed investigations is conducted. Specifically, the geology study should start with techniques such as remote sensing, topographic map interpretation, and geologic mapping. After the gross features have been delineated, the studies are turned toward defining the more detailed elements of jointing, rock strength, ground water hydrology, and other characteristics. Finally, all of the information is integrated in an assessment of the rock behavior in relation to the types of openings, excavation methods, and support requirements that are appropriate for the rock conditions (Hansen and Lachel 1980).

4.1 INITIAL SCREENING

Initial screening will be based upon geological and hydrological information drawn from surface mapping, water and exploratory well drilling, geophysical mapping, mining experience, and other sources. The candidate sites must have acceptable depth, structural soundness, horizontal continuity, and hydrostatic containment. Proposed sites showing evidence of active faults or regionally active seismicity, and recent volcanic provinces would be eliminated during the initial screening. Sites characterized by complex structural geology and high horizontal in situ stress would also likely be discounted at this time.

Remote sensing techniques may have supplied data about faulting in the region of interest. These include aerial photography, side-looking airborne radar (SLAR), false-color infrared (IR) photography, thermal IR imaging, and multispectral scanning from satellites or high altitude aircraft (Hansen and Lachel 1980). These imaging techniques are used principally in locating lineaments and determining their lengths and orientations. Lineaments located by remote sensing must always be verified on the ground because the reasons for their occurrence are not necessarily associated with rock structure. If the lineaments prove to be faults or other structural features, they must be considered in the stability assessment, along with structural features found in later studies. Black and white aerial stereo photography is undoubtedly the single most useful type of imagery; it should be secured for every mining project. Sophisticated imagery from such techniques as SLAR, thermal IR, and multispectral scanning must be programmed and analyzed by trained personnel; otherwise it should not be obtained.

The most comprehensive source of remote sensing imagery is the federal government. The Department of Agriculture (Forest Service, Soil Conservation Service, and the Bureau of Land Management), and the Department of Defense (Air Force and Army Corps of Engineers) are potential sources of imagery in the United States. Some commercial firms also obtain imagery tailored to individual needs, or have photos of a particular area already in their files. A regional planning agency is usually aware of the availability of such imagery (Hansen and Lachel 1980).

4.2 ROCK CHARACTERIZATION

Merritt and Baecher (1981) have prepared an excellent overview of site characterization in rock engineering. They describe site characterization as an evolutionary process typically beginning with aerial photographic analysis and geologic mapping during pre-feasibility studies and progressing through exploratory borings, permeability measurements, laboratory testing, and perhaps large-scale in situ rock mechanics tests during the feasibility and design phases of the project. The purpose is to develop an operational concept of the geology of a site, to obtain field data for engineering analysis, and to identify geologic features that could adversely affect construction or facility performance. The site characterization process must continue throughout the construction phase because no exploration program can completely characterize all particulars of the geology.

The successful design and construction of air storage chambers is dependent on knowledge of subsurface conditions. To avoid construction problems or to solve them economically, a detailed exploration program should include analysis of the following geologic conditions (Selmer-Olsen and Broch 1977):

- lithology and mineralogy; mechanical and chemical properties; uniformity, areal extent and depth of rock mass; and alteration characteristics
- structure - faults, shear zones, fractures, joints, stratification, foliation; seismic risk; and in situ stress conditions and stability
- hydrology - surface water drainage, water table, zone of saturation, locations of aquifers, and hydrothermal sources
- depth of weathered zone
- features of engineering geology affecting excavation.

A multiphased core-drilling and geophysical data-gathering program is necessary to allow reasonable confidence limits on the suitability of a specific site for CAES applications.

Major limitations exist in the predictive tools that will enable the geologist and engineer to assess the local hydrology and in situ stresses in a rock mass and to design economic excavation methods. Direct rect geologic investigations are restricted to accessible rock exposures or to the borehole. Current methods for coring and core analysis are inadequate to obtain a representative sample of the structural characteristics of a rock mass reservoir and other rock properties (Einstein et al 1977). High-pressure logging and telemetry devices need to be developed to furnish information on lithology, porosity, permeability, fracture systenis, fluid content, and movement in rock masses. Instrumentation for monitoring rock stability and containment in the storage cavern over a long period and under adverse conditions also should be developed (U.S. National Committee 1978).

4.3 DRILLING AND IN SITU TESTING

The drilling program will delineate the host rock laterally and vertically, provide core samples to enable determinations of joint orientation, permeability, and mineralogy, and will enable measurement of hydrostatic heads. Horizontal in situ stress measurements should be made at the projected depth of the CAES cavern. Core specimens supplied to a rock mechanics laboratory will be tested for strength and permeability. The host formation will be evaluated for excavation stability and cost.

The drilling program will evaluate the stratigraphy of overlying formations, including the unconsolidated uppermost zone if present. Cores taken near the surface will be analyzed for rock suitability to surface reservoir construction. Simultaneously, the ground water regime will be evaluated.

The most common methods of measuring an in situ modulus include seismic or sonic velocity tests with the calculation of a dynamic modulus, borehole measurements using a dilatometer, and larger scale plate-jack tests with loading plates of varying sizes. In assessing these methods, the dynamic modulus of elasticity is almost always higher than corresponding static values and therefore must be reduced by an amount believed to vary with rock type, rock quality, and perhaps with a factor depending upon the velocity of the wave propagation of seismic versus sonic or ultrasonic tests. Common reduction factors of dynamic to static moduli may vary from 4 to 8 or even greater. Only in the most massive homogeneous material may the dynamic and static moduli be considered approximately equal (Merritt and Baecher 1981).

The borehole dilatometer can rapidly determine an in situ deformation modulus, but is severely limited by its small test area with respect to the geologic features that control the rock modulus. Its use is justified by many rock mechanics engineers on the basis of the large number of values obtained. The level of confidence in the results is great only if the rock mass is relatively homogeneous.

The plate jack test is considered by practicing rock engineers to provide the most reliable values of in situ moduli of deformation principally because of the larger area of the loaded surface. It also most closely simulates the direction of loading in a pressure tunnel, and measures the influence of the de-stressed zone around the tunnel perimeter. The loading surface can be of any size but plates of at least 1 m^2 are often preferred. In the case of pressure tunnels, for example, the rock should be loaded to operating stresses and cycled several times. If the rock is excessively jointed or weathered or is otherwise a low strength material, long-term tests should be run to most closely simulate actual operating conditions.

Measurements of rock displacement are best made using extensometers placed at varying depths behind the bearing plate. Surface measurements alone are not adequate because the modulus generally increases with depth behind the surficial stress-relieved zone and surficial values are often too low and therefore not appropriate for design purposes.

The hydrofracturing method of stress determination is uniquely applicable to any reasonably practical depth, particularly in excess of 500 m, and is used widely for proposed deep installations such as radioactive containments and deep pumped storage plants. Comparisons of these tests and overcoring methods have shown satisfactory correlations (Haimson 1980, 1981).

4.4 GEOPHYSICAL METHODS

The geophysical methods generally most useful in assessing ground conditions are seismic refraction and reflection, electrical resistivity, magnetic intensity, and gravity metering. The degree of resolution of each method depends on the adjacent rock units possessing highly contrasting values of the properties being measured. In other words, the greater the contrast, the greater the resolution (Hansen and Lachel 1980).

The accuracies of the geophysical investigations depend also on the validity of the basic assumptions made in the mathematical model. For example, the seismic refraction method requires that the seismic velocities of the various rock and soil units increase with depth. No refraction occurs if a low-velocity refractor underlies a high-velocity refractor. If refraction does occur in such a case, the calculated depth to the next high-velocity refractor below the lowest velocity layer will be in error, unless the effects of the low-velocity layer have been taken into account.

The ultimate results of the geophysical surveys are interpretations of the relationships between the geology and the properties measured. The art of geophysical interpretation lies in relating the field measurements to the site geology. Geophysical instruments do not measure the geology directly, so the interpretations must be verified by other means such as drill holes, test pits, and trenches.

Geophysical surveying, which is relatively inexpensive, is a good way to extend the interpretations made from the drilling and mapping program. Such surveys can guide exploration by locating anomalies to be checked by drilling, resulting in a greater geological understanding with less cost in time and money. Table 3 lists the most common geophysical methods, with their uses, limitations, and costs (Hansen and Lachel 1980).

Down-hole geophysical logging is an important supplement to visual and laboratory examination of core. Among the most effective logs for distinguishing changes in lithology are gamma ray-neutron, spontaneous potential-resistivity, three-dimensional velocity, and computer processed logs for elastic properties, interval and integrated travel time, and porosity.

4.5 JOINT SURVEYS

Knowledge of jointing is critical to the technical feasibility of CAES implementation because joint systems influence both cavern stability and air leakage.

Surveys of minor discontinuities and joints have been emphasized in recent years (Merritt and Baecher 1981). Attempts to place joint surveys on a firm sampling theory foundation have met with some success and stochastic descriptions of joint patterns are being studied.

The most commonly measured geometric properties of jointing are spacing, trace length, and orientation. The distribution of spacings between joints along a sampling line is usually well modeled by an exponential density function. This distribution corresponds to random and independent location of joints. Because the exponential density is defined by one parameter, a simple relationship exists between RQD and average joint spacing for hard unweathered rocks,

$$RQD = 100\Sigma^{-(0.1)\lambda}\,[(0.1)\lambda+1]$$

where

λ = average spacing in meters.

TABLE 3. COMMON GEOPHYSICAL EXPLORATION METHODS (HANSEN AND LACHEL 1980)

Method	Instrument Cost[a] (1978 US $)	Principal Use[b]	Limitations	Application
Seismic Refraction	500 to 30,000	Locate top of rock	Assumes increasing velocity with depth	General
Seismic Reflection	3,000 to 30,000	Determine rock structure	Difficult to interpret; poor resolution above 46 m in depth	Limited
Electrical Resistivity	900 to 5,500	Locate top of rock and groundwater	Affected by stray currents and buried metal	General
Magnetic Intensity	900 to 3,500	Determine geologic structure	Difficult to interpret	Limited
Gravity Metering	11,500 to 14,000	Determine geologic structure	Difficult to interpret	Minimal

(a) The costs cited do not include the cost of airborne equipment or of special analysis equipment such as plotters, computers, etc.
(b) All geophysical techniques should be correlated with drilling or other means of direct examination.

The exponential assumption is inappropriate for certain classes of joints, e.g., bedding plane joints.

Reported distributions of joint trace length are less consistent than those for spacing, perhaps because biases are implicit in sampling plans and in data grouping before analysis. Log normal distributions are the most frequently reported.

Measurement of the shear resistance of joints or other discontinuities has also been a subject of renewed interest, but basic problems of mechanics need to be resolved before inferences of parameter values for analysis can be placed on a rigorous basis (Merritt and Baecher 1981). The influence of ground water is important because flow occurs primarily along joint planes.

4.6 PERMEABILITY MEASUREMENT

Permeabilities measured in the laboratory have been of little use in establishing rock mass permeabilities because of the importance of fracturing and the difficulty or impossibility of obtaining representatively large specimens for laboratory testing. This means that most testing for permeability is done in situ. The advantages of in situ testing are that it lessens sample disturbance, tests a representative volume of rock mass, and may be more economical than laboratory testing. The disadvantages are that boundary conditions are usually complex, and that strong variations in stresses, strains, and heads may exist across the rock mass affected by the test (Merritt and Baecher 1981).

In situ tests currently in use are based on one of four principles: 1) injection flow rates are measured in a boring or other opening at constant internal pressure (injection tests); 2) recovery of pressure or head is measured in a boring after addition or removal of a known volume of fluid (pulse test); 3) flow of dyes or tracers is measured between two or more borings while holding pressure or head gradient constant (tracer test); 4) transmissivity distributions in space are inferred from the distribution of observed potentials (inverse problem) (Merritt and Baecher 1981).

Injection tests pressurize an interval of boring isolated by impermeable packers. The test was originally proposed for grout-take predictions and provides only poor accuracy or reliability for measurements of permeability. Flows into a boring are often nonlinearly related to applied pressures, possibly due to turbulence and opening of fra'ctures, and often change with time. Nonradial flows at the packers and other uncontrolled boundary conditions make the measurements difficult to interpret. Results are generally erratic; variations of 10- to 100-fold in measured water inflows are not uncommon for fractured rock. Of course, this erraticism may be due to actual variations of the rock mass (e.g., fracturing), and may be used as boring water loss is used to identify geologic structure.

Well pumping tests, in which drawdowns are held constant, possibly have less effect than injection tests on fracture apertures. However, they are less easily directed at individual zones within a boring, and require more time to perform. Clusters of borings installed with piezometers about a pumping well allow larger volumes of rock to be tested, as well as to test for anisotropy, but are correspondingly more expensive.

Pulse tests have become increasingly popular in recent years, but share most of the advantages and limitations of injection tests. They require less complicated instrumentation than injection tests, particularly in low permeability formations. On the other hand, interpretation of their results is complicated by the influence of equipment compliance on pressure decay (Merritt and Baecher 1981).

Tracer tests are less common than injection or pulse tests, in part because of the difficulty and expense of performing them. They require two or more borings and instrumentation for measuring tracer concentrations in well fluids. A significant advantage of tracer tests is that they allow measurement of dispersion parameters and transit times. A disadvantage, particularly in fractured media, is that the actual flow between borings may not be known, and may be extremely tortuous (Merritt and Baecher 1981).

Back calculation of spatial distributions of transmissivity is made possible by the development of numerical modeling capabilities, notably finite element methods. Having measurements of transmissivity and storage at each node position, and estimates of boundary conditions, predictions of heads at each node can be made. By trial and error adjustment, model parameters can be changed until predicted and observed head patterns are similar. More recent work has focused on statistical methods for estimating parameters from heads. Typically these involve linear or quadratic programming. One difficulty of the inverse approach is that transmissivities cannot be calculated from heads alone. At least one value of transmissivity must be known along each streamline of flow to determine transmissivity all along the line. This means that transmissivity must be known along one curve (or surface) within the flow domain.

A second problem is that small errors in measured heads are unimportant for predicting flows, but very important for back calculating transmissivities. A small error in head, given the small gradients in ground water flow, induces large errors in gradients that are coefficients in the equation to be solved for transmissivity.

4.7 OVERBURDEN THICKNESS LIMITATION

Proposed CAES sites should be assessed for unfavorable geologic and hydrologic conditions. Investigations should include data from water well drilling and boreholes.

Shafts can be sunk in areas where the overburden in incompetent or heavily water-bearing, but only at increased grouting, lining, and freezing costs. These measures have a practical depth limit of about 50 m (Walia and McCreath 1977). Highly specialized and costly measures are required to sink shafts to a depth below 50 m in areas where ground water and overburden conditions are unfavorable. The additional cost may be as much as 1% of the total cost of the CAES facility.

Specific shaft construction studies should be undertaken for each CAES site. If an absolute limit on shaft depth can be determined, this information can be used in screening all potential CAES sites.

5.0 OPERATIONAL ISSUES

During the operation of a CAES cavern, the rock is subjected to cyclic variations in applied pressure and temperature (Gnirk and Port-Keller 1978). These conditions induce thermomechanical stress perturbations in the surrounding mass which may degrade the rock strength and lead to progressive general and local instabilities. Such instabilities, in turn, create the potential for other geologic and hydrologic phenomena to occur. Cavern operating parameters and their likely impacts are discussed in this section. Techniques for monitoring the effects of reservoir operation are also described.

5.1 INJECTION PARAMETERS

Cavern injection parameters include temperature, pressure, humidity, cycle duration and migration of the air/compensating water interface. The inlet air temperature will fluctuate between 30 and 80°C. Compensating water may fluctuate between 0 and 30°C, depending upon surface temperature, conduit/rock temperature, and compressed air temperature.

The maximum allowable reservoir pressure will be determined by the height of the water compensation column. For pure water the pressure is 9.81 kPa/m of depth. Maximum charging pressure should not exceed hydrostatic pressure by much more than the pressure differential required to displace aquifer water. Maximum charging pressure probably should not exceed 12 kPa/m of depth, i.e., about 20% over hydrostatic pressure.

Humidity of the injected air at temperature and pressure has not been specified. Humidity will be a function of ambient air humidity and the intercooling, aftercooling system used in the compressor train. The interaction between reservoir air' and the compensating water column will tend to increase the humidity. The injection/withdrawal cycle duration may reach 24 hours on weekdays and exceed 24 hours on weekends. During typical weekday operation, up to 12 hours of electrical generation may be provided. The air/compensating water interface may migrate between the floors and the ceilings of the cavern array. Because of water compensation, pressure fluctuations within the cavern will be relatively small.

The system design will prevent rapid cavern depressurization caused by the champagne effect. A detailed discussion of this design is beyond the scope of this document.

5.2 POTENTIAL IMPACTS WITHIN THE HOST ROCK

The cavern injection and operating parameters will have a variety of impacts depending on the nature of the host rock. These potential impacts are detailed in the following subsections.

5.2.1 Applied Stress

Laboratory studies of the effects of applied stress on permeability and porosity provide useful information that may be used to predict the hydrologic characteristics of rock at depth, as well as behavior of certain structures (Port-Keller and Gnirk 1981). Because of petroleum industry needs, many studies have concentrated on porous intact sedimentary rocks. Less extensive work has been accomplished on pressure effects in intact metamorphic and igneous rocks. Finally, studies of pressure effects on secondary hydrologic properties (due to fractures, etc.) are only beginning to clarify stress-flow behavior.

The effect of applied stress on hydraulic conductivity of rocks and rock masses has been approached by attempting to define first the stress/fissure opening relationship and subsequently the fissure opening/conductivity relationship. The effects of different modes of applied stress, rates of loading, and loading history of samples make permeability measurements under stress highly variable. Naturally, variation is also particularly significant between intact and nonintact rock. Attempts to numerically model such stress-flow behavior have taken various approaches.

Witherspoon and Gale (1977) have extensively reviewed studies of mechanical and hydrologic properties of fractured rocks and their relationships. Some significant general findings and representative laboratory data are discussed below. Figure 41 illustrates the hydraulic

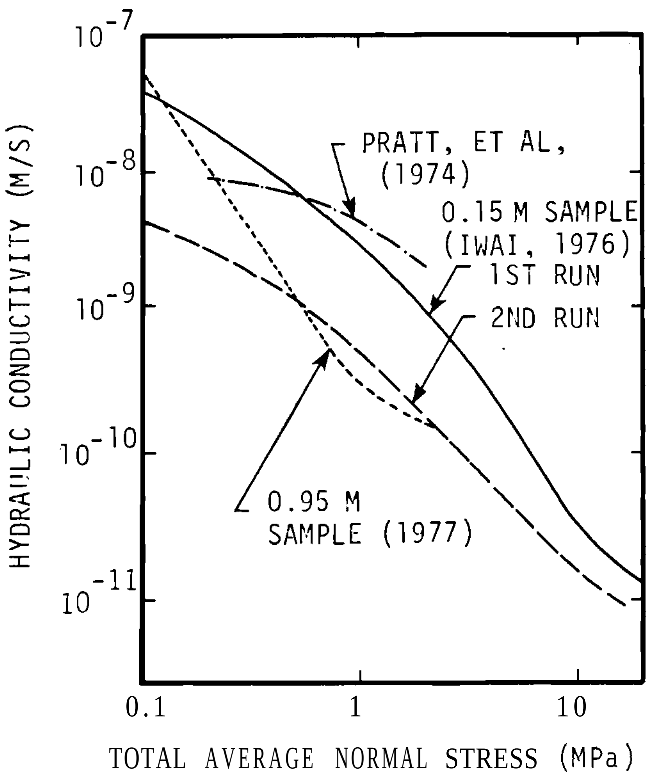


FIGURE 41. Hydraulic Conductivity of Granite Core Specimens as a Function of Normal Stress for Constant-Head Tests (Witherspoon and Gale 1977)

conductivity of a single joint as a function of stress applied normal to the joint plane. Figure 42 plots rock mass permeability as a function of normal stress for several values of joint spacing. Some experimental data points are also included. Figures 43 and 44 show a general decrease in permeability with increased normal stress.

The applicability of calculated stress/flow functions to actual rock mass behavior is highly questionable. Theoretical calculations usually assume planar joints of consistent width and regular spacing. The occurrence and spatial dimensions of real joints are not homogeneous with respect to aperture width, frequency, length, and orientation. Thus, actual stress-flow behavior will probably be highly variable.

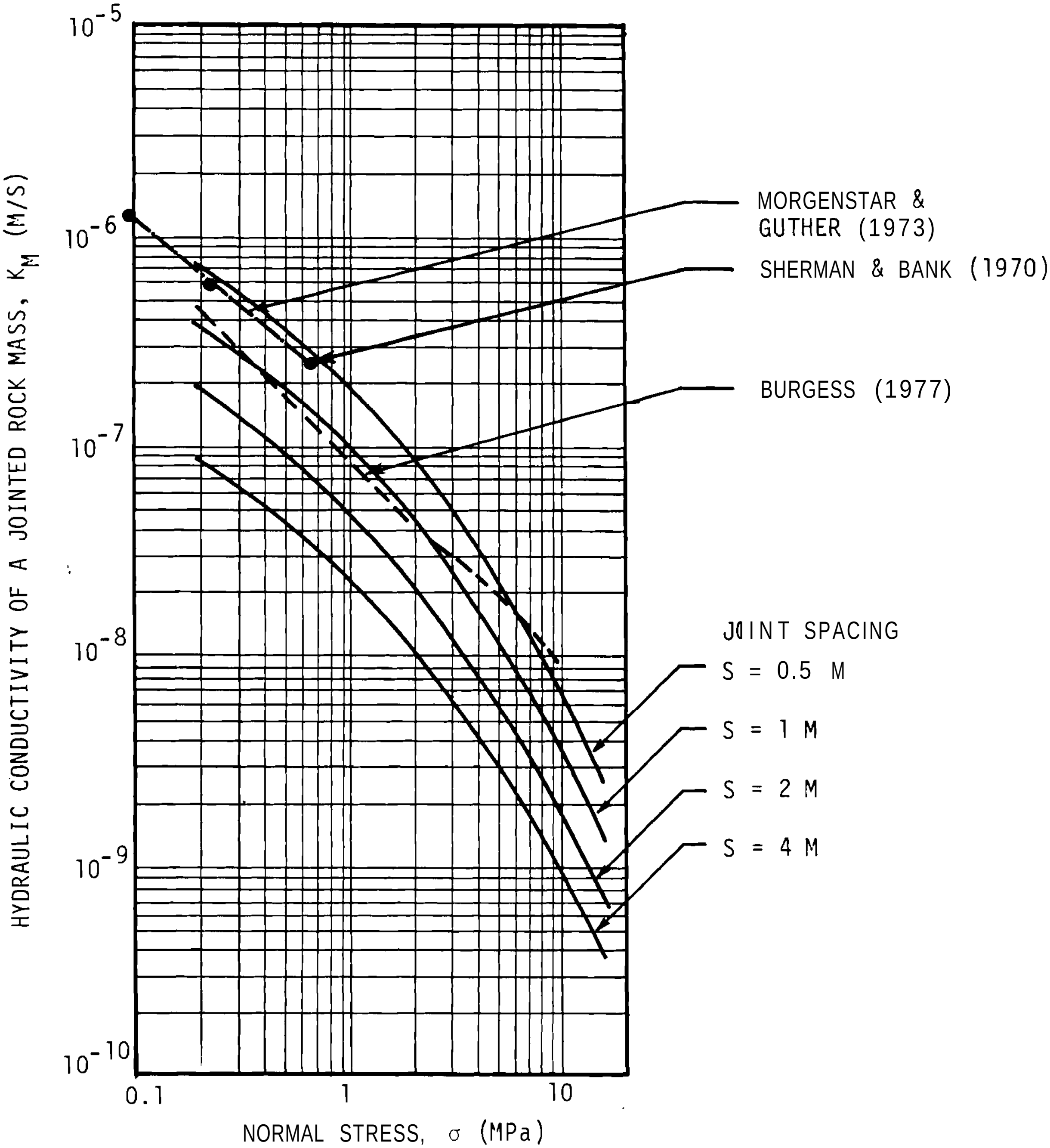


FIGURE 42. **Hydraulic Conductivity of a Jointed Crystalline Rock Mass as a Function of Normal Stress for Various Joint Spacings (Port-Keller and Gnirk 1981)**

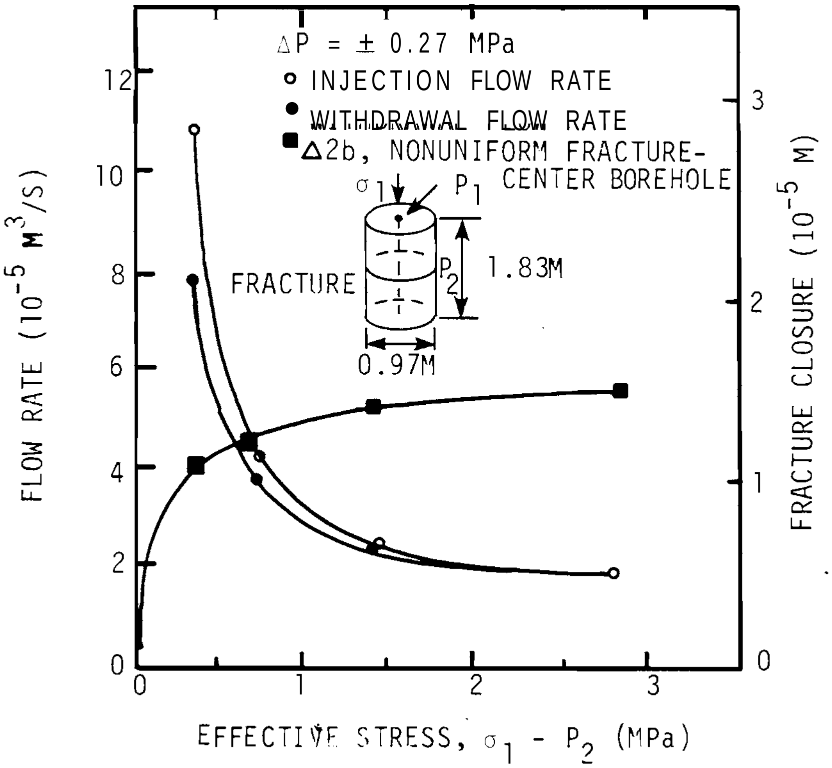


FIGURE 43. Flow Rate and Fraction Aperture Closure as a Function of Effective Stress for a Horizontal Saw-Cut Fracture in a Granite Cylinder (Port-Keller and Gnirk 1981)

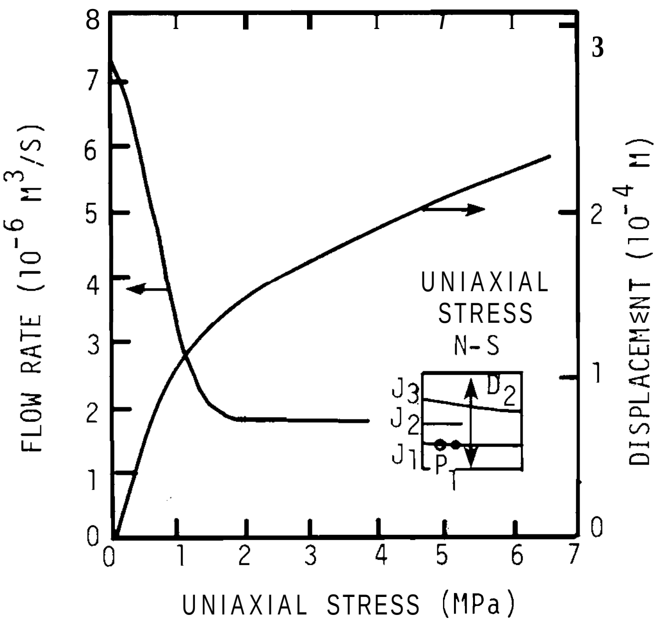


FIGURE 44. Effect of Stress on Displacement and Fluid Flow in a Vertical Fracture J_1 in Granite (Port-Keller and Gnirk 1981)

Tests on larger laboratory and field samples have shown that volumetric flow rates decrease with increased applied stress. However, these tests have also shown that such flow rates tend to reach a minimum and then remain fairly constant. Thus, the joints appear to never actually close completely. Figures 43 (laboratory) and 44 (field) illustrate this phenomenon. Other investigators (Shehata in Sharp and Maini 1972 and Jones in Nelson and Handin 1977) also indicate that rock mass fractures never completely close to fluid flow.

The effect of confining pressure on permeability of intact rock has also been studied. Figure 15 plots hydraulic conductivity as a function of total average normal stress.

5.2.2 Dilatancy Effects and/or Cyclic Mechanical Loading Effects

The subject of dilatancy effects and/or cyclic mechanical loading effects on the permeability of both intact and nonintact rock is not yet well understood (Port-Keller and Gnirk 1981). Depending on several factors--notably, percentage of fracture strength to which a specimen is loaded, number and rate of applied loading cycles, and rock type--permeability may increase or decrease.

Zoback and Byerlee (1975) have measured the permeability of Westerly granite deformed under constant confining pressure and constant pore pressure at 75 to 95% of intact failure strength. Figures 45 and 46 give data for samples that were previously loaded to high differential stress over 20 times. The figures present volumetric strain (compression is positive) as a function of differential stress (difference between axial stress and confining pressure). As a sample was initially stressed, permeability slightly decreased; however, with further increase in differential stress, the samples became dilatant and permeability increased. As differential stress was removed from samples, permeability remained quite high until almost all load was removed (Zoback and Byerlee 1975).

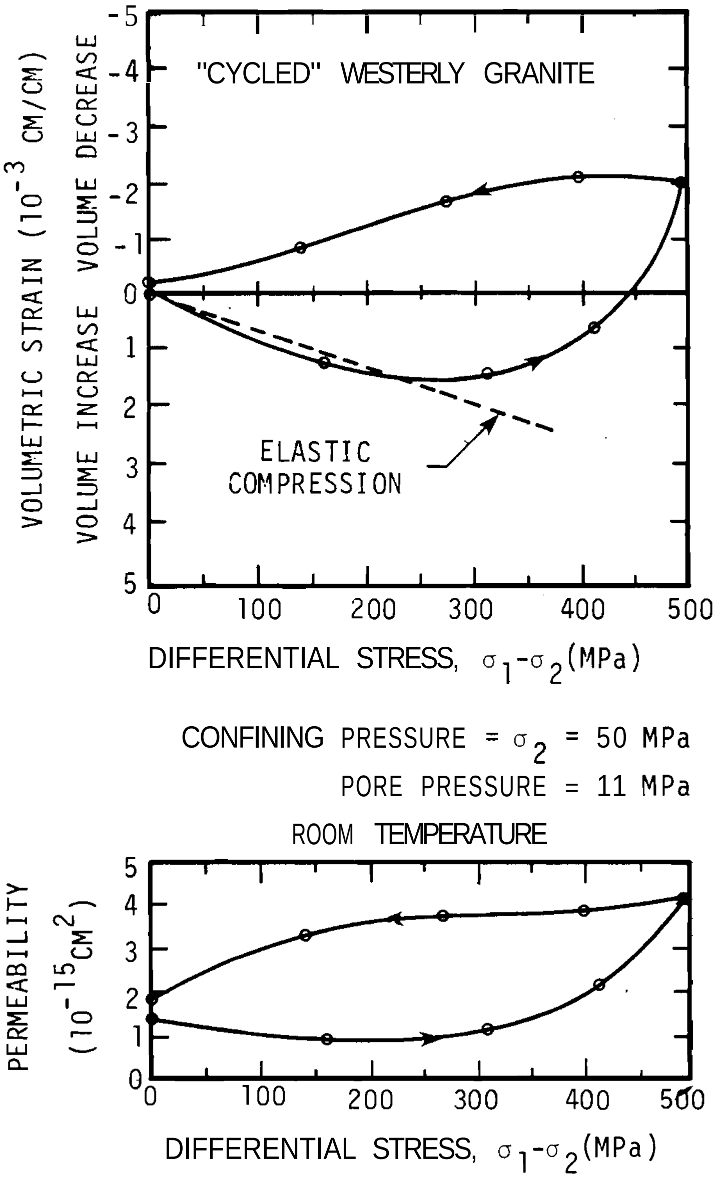


FIGURE 45. Volumetric Strain and Permeability as a Function of Differential Stress for Westerly Granite at a Confining Pressure of 50 MPa (Zoback and Byerlee 1975)

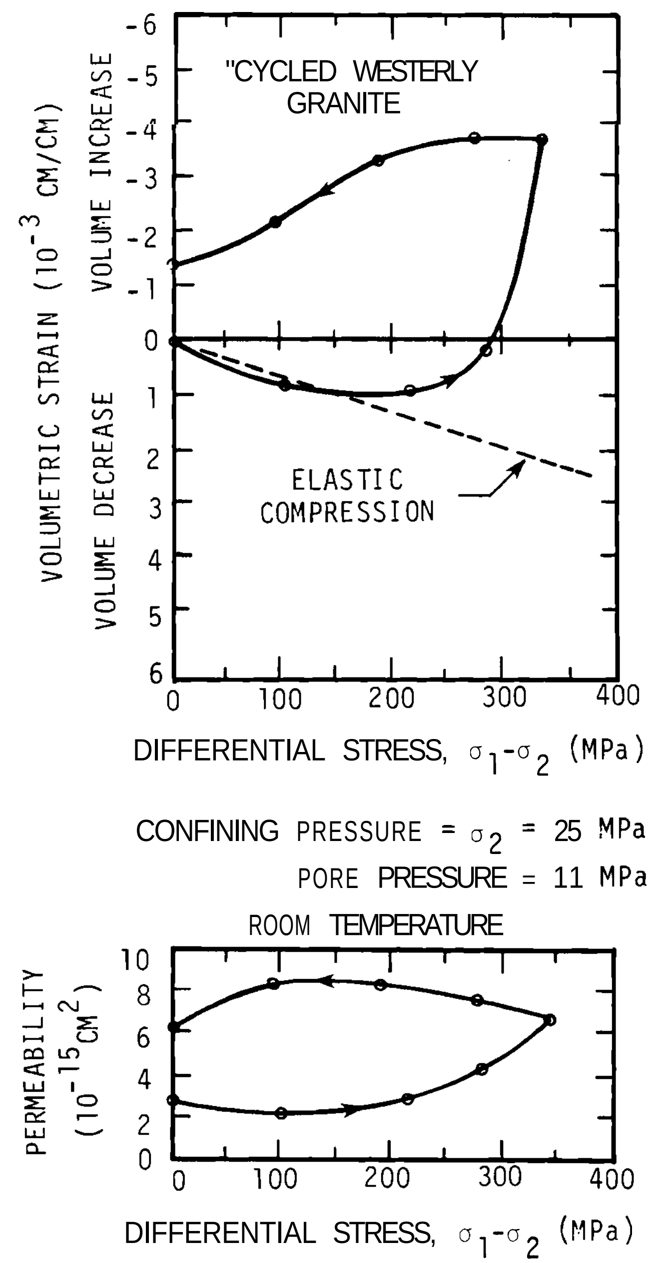


FIGURE 46. Volumetric Strain and Permeability as a Function of Differential Stress for Westerly Granite at a Confining Pressure of 25 MPa (Zoback and Byerlee 1975)

Brace (1977) repeated measurements similar to Zoback and Byerlee and reports similar results (see Figure 47). His samples had been previously stress-cycled, and the fracture stress was assumed to be 520 MPa.

Finally, the effect of shear deformation on permeability of rock masses is also not clearly understood. Increasing shear stress and deformation appear to cause both increasing (Sharp and Maini 1972) and decreasing (Jouanna 1972) permeability. Behavior may depend upon dilatancy, which occurs predominantly after peak strength is reached (Witherspoon and Gale 1977). Sharp and Maini (1972) indicate that an investigation by Maini (1971) shows a marked increase in permeability with shear stress; however, they also state that the large amount of dilation probably occurred as a result of low normal stress and that such a large effect might not be observed in an actual rock mass.

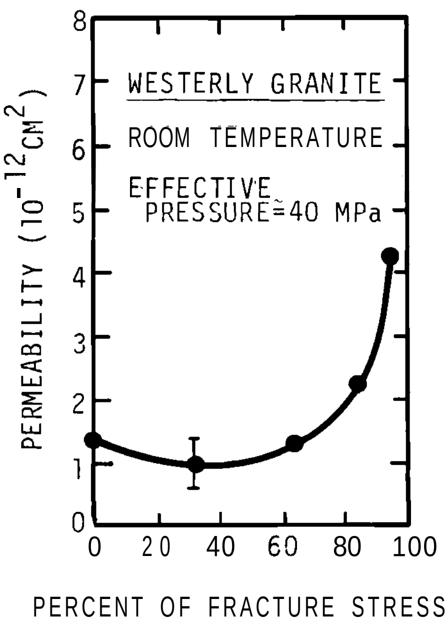


FIGURE 47. Effect of Stress on the Permeability of Westerly Granite (Brace 1977)

It is clear that dilatancy effects and the effects of cyclic mechanical loading are not thoroughly understood, particularly under the special loading conditions that would be encountered in the CAES cavern situations. Cavern design should avoid maximum rock stress conditions during cycling to prevent the onset of dilatency.

5.2.3 Temperature

The effect of temperature change on the hydrologic properties of rocks and rock masses may also be a significant factor in CAES concepts, but it is not a thoroughly understood phenomenon (Port-Keller and Gnirk 1981). Studies of temperature effects have originated within the petroleum industry.

Local differential thermal stress can result from temperature gradients, mineral inhomogeneities, anisotropy of expansion coefficients for individual minerals, and differential thermal expansion between matrix and granular constituents. If differential stresses at grain boundaries are sufficient, microcracking may cause dislocation or fragmentation of grains. However, preliminary laboratory ventilation experiments on quartzose sandstones using dry air at elevated temperatures and pressures indicated no significant changes in the microstructural fabric of these rocks (Pincus 1979).

5.2.4 Air Slaking and Rock Spalling

Air slaking or spalling of rock surfaces in either compensated or uncompensated storage caverns may occur as a result of humidity variations (General Electric 1976; Lee and Klym 1977). Hence, CAES chambers should not be constructed in rock masses with closely spaced discontinuities such as joints, fractures, faults, thin bedding, cleavage, and foliation. These structurally weak zones would be subject to decay or alteration under variable moisture, pressure, and temperature conditions. The cyclic wetting and drying of clays in shear and weathered zones will cause swelling and disintegration.

The slake durability (resistance of rocks to wetting and drying) must be studied as a part of the engineering geologic research to be conducted for each potential site. Because pre-construction geological surveys are inadequate for this purpose, it will not be possible to completely assess and eliminate all zones of poor durability prior to excavation. Also, rocks exposed during excavation are subjected to a new geological environment (Weinstein et al 1978). Zones deemed susceptible to slaking and spalling can be treated by grouting, shotcreting, rock bolting, and wire mesh reinforcement in various combinations.

5.2.5 Air Leakage Experiments

Bawden and Roegiers (1980) have performed experiments to study two-phase countercurrent flow through simulated rock fractures with varying aperture and orientation relative to the pressurized cavity. Various entrance geometries were simulated to examine their importance in bubble or slug initiation. Sensitive pressure transducers were used to detect small variations in the pressure field during bubble propagation. Deformations and volumes of bubbles were photographed.

Gas escape virtually never occurs if there is visible water flow into the cavern. Gas escape generally occurs shortly after water inflow ceases. Gas escape occurs initially as lobes or tongues of air penetrating water-filled apertures. As would be expected, gas escape through larger apertures is much faster than through fine apertures. With large apertures only two or three lobes occur. Each lobe is wide with a moderately large radius of curvature at the nose. For finer apertures the initial gas escape forms a dendritic pattern of thin branching lobes with much smaller nose curvatures. Following the initial lobe or tonguing escape, the lobes expand and the aperture eventually becomes completely dewatered. Gas escaped from a square cavern at pressures about 20% below those for a horizontal elliptical cavern. The escape always initiated at the square corners. Escape is also related to aperture dip angle with steep angles favoring bouyant escape through water. Fracture roughness, contact areas and fracture interconnectivity are significant, although unstudied, parameters.

5.3 CAES RESERVOIR MONITORING

Reservoir monitoring will detect local rock instability, general rock instability, air leakage instability and behavior of the compensating water column. Instabilities due to rock strain and failure should be detectable by monitoring compression and shear waves originating within cavern walls, by monitoring tilting and subsidence, and by separating and examining solids from withdrawn compressed air. Air leakage rates can be determined by measuring the height of the horizontal water-air interface as a function of time. A simple recording float device can be used. Temperature and pressure within the cavern can be continuously monitored at various locations. (Currently available instruments may be inadequate for long duration monitoring.)

Microseismic disturbances may divulge and locate roof falls or slabbing from the walls. Two sensors at different locations are needed to determine the source. Tiltmeters and precise level-recording instruments will reveal minute degrees of subsidence (Thoms 1978).

After a cavern has been completed, a shut-in pressure test may be used to prove air containment stability (Golder Associates 1979). Pressure would be monitored for 24 hours. If depressurization were detected attempts could be made to correlate it with microseisms originating within the cavern walls, which could signify rock failure. Longer term pressure testing may be necessary to achieve meaningful results.

The two most important devices used to monitor deformations above a roof are extensometers, which measure deformations parallel to the axis of the borehole, and inclinometers, which sense deforniations normal to the direction of the borehole (Golder Associates 1979).

Methods available for monitoring the cavern walls, roof, floor, and compensating water column may not be adequate for all operational requirements. Continuous monitoring at depths near 800 m, at temperatures up to 80°C and over three decades is a strong challenge.

5.4 CHAMPAGNE EFFECT

The champagne effect is a two-phase flow instability that could occur in a hydraulically compensated compressed air reservoir (Giramonti and Smith 1981). This effect results from the functional relationship between air solubility in water and system pressure. As air-saturated water rises in the compensating water shaft, the solubility decreases causing the exsolution of air as bubbles. This two-phase medium is less dense than the single phase medium, causing a pressure imbalance between the water shaft and the air storage cavern.

A thorough understanding of the dynamics is needed to enable invention of countermeasures. Many organizations have conducted preliminary analytical and experimental modeling of this effect. Although these efforts have not produced definitive designs to prevent the champagne effect, a number of potentially promising control schemes have been identified. These methods attempt to control the physical mechanisms governing this hydraulic instability. These methods can be classified as those that attempt to 1) restrict the rate at which cavern air is dissolved, 2) reduce the buoyancy effects due to bubbles in the shaft, 3) minimize or balance the pressure differential which tends to accelerate the fluid's rate of rise, 4) introduce geometric constraints which tend to reduce acceleration of the fluid, and 5) combine any of the above.

Potential solutions to the champagne effect problem involve geotechnical evaluation. They include contouring the cavern to minimize the air/water interface area; oversizing the cavern to prevent blowout; and constructing a U-bend water seal between the reservoir floor and the water shaft to balance the buoyant head and overcome the inertia of the column.

5.5 DISCUSSION

The specification of CAES caverns in hard rock immediately identifies as candidates igneous (granitic), possibly metamorphic, and certain sedimentary (limestones, marbles, dolomites) rocks. At the pressures and temperatures mentioned above, these rocks behave in a brittle manner in which the modes of inelastic behavior are microcracking and frictional sliding on fissure surfaces. This type of behavior is dominant for near surface crustal conditions as opposed to at-depth conditions where the pressure-temperature regime is conducive to ductile rock behavior involving dislocation processes. Thus, consideration must be given to the influence of joints or planes of weakness, permeability, and ground water presence.

The general instability of the cavern must be evaluated by use of the Mohr-Coulomb condition of rock failure with temperature-dependent properties. The strength of the rock will be progressively reduced with the number of loading cycles.

The hydraulic conductivity of the rock, a function of stress and temperature, will be perturbed by the initial excavation, and subsequently perturbed by the cyclic pressure and temperature loadings. Failure of the intact rock and/or joints will also perturb the hydraulic conductivity.

The local rock instability of the cavern periphery is related to the spalling and microfracturing characteristics of the rock under cyclic pressure/temperature loading and air/water interaction. The limit of acceptable rock disintegration must be established with respect to degradation rates over the cavern lifetime and from the viewpoint of possible particulate transport to the turbine system during compressed air withdrawal.

The initial shearing deformation caused by mining is likely to exceed the peak strength of the rock. All subsequent stress changes must fall within the elastic range of the rock. When compressed air is introduced into the storage cavern, some of the original stable compressive stresses will be restored. However, the internal pressure created by the compressed air represents only a fraction of the unbalanced overburden pressure due to the deformation of rock by mining. The situation is further complicated if the original stress field is not isotropic (Howells 1977).

Substantial temperature rises can be permitted, but the extent to which compressed air entering the cavern at high temperature can be cooled is limited by the ability of the rock walls to withstand compressive stresses. A drop in air temperature cools the surrounding rock, creating tensile stresses. Temperature transport by the pressure compensating water column will also affect the cavern temperature regime. Fatigue will influence the permissible temperature range in the cavern.

Information and tools for determining the fatigue process in jointed rock are limited. Documentation of fatigue for varying operational scenarios would be very useful. This should be done in conjunction with early CAES developments to establish future constraints on operational parameters (Weinstein et al 1978).

5.6 SUMMARY OF CAES OPERATING ENVIRONMENT

The CAES operating environment of primary interest includes the following (Fossum August 1979, Port-Keller and Gnirk 1981):

- Initial vertical and horizontal in situ stresses and hydraulic pressures corresponding to cavern depths of 500 to 1,000 m. (Vertical stress gradients range from 0.0282 MPa/m for metamorphic rocks to 0.0247 MPa/m for sedimentary rocks with an intermediate value of 0.0269 MPa/m for igneous rocks). In general, for these depths, the maximum horizontal in situ stress may be substantially greater than the vertical in situ stress, by a factor of up to 1.5. The water pressure may be taken as 0.01 MPa/m.
- Inlet air temperatures from 30 to 80°C.
- Compensating water temperatures from 0 to 30°C.
- Initial rock temperatures from 20 to 60°C. (The geothermal gradient varies from region to region with an average value of approximately 29°C/Km).
- A useful life of approximately 10,000 cycles, or 30 years.

6.0 GUIDELINES AND STABILITY CRITERIA

The guidelines and stability criteria, currently available to the CAES cavern designer, are presented in Table 4. Categories include general geological environment, hydrology, host rock characteristics, structural characteristics, other geological characteristics, design parameters, and operating parameters.

In evaluating a particular site, the minimum acceptable design should be compared with the known geology before recommending preliminary exploration. Factors that should be considered in this comparison include hydrostatic pressure; surface water availability; host rock depth, thickness and competence; host rock structure and horizontal stress; and nature of the geological province.

TABLE 4. GUIDELINES AND STABILITY CRITERIA

Category	Requirement	Qualification
General Geological Environment	Hard rock formation with adequate depth, geometry, lithology, structural integrity, ground water saturation and absence of negative environmental features, e.g., high lateral stress field.	Analysis of existing geological information. Geophysical surveying, exploratory drilling, core analysis, fracture pattern mapping geophysical logging, in situ permeability and stress measurements.
Hydrology		
a. Hydrostatic pressure	Hydrostatic pressure within the host rock equals the pressure of stored air.	Depth of reservoir beneath a stable water table would typically be at least 700 m. This corresponds to 6.86 MPa at a hydrostatic gradient of 9.8 kPa/m.
b. Ground water chemistry	Ground water in contact with host rock is essentially in chemical equilibrium with the host over the range of CAES injection/withdrawal air temperatures and at CAES pressure. The ground water chemistry is not perturbed by commingling with the compensating water column.	Surface compensating water and ground water will be commingled and analyzed for precipitation and change in pH.
c. Surface water availability .	Surface water supply must be ample to provide compensating column and makeup water. In addition, a site for a surface reservoir must be identifiable in the immediate vicinity of the candidate cavern site.	Surface lakes, rivers or shallow aquifers will be evaluated from available hydrological information on volumes, level fluctuations, runoff, pumping rates, etc. Topography, overburden and near-surface bedrock will be evaluated for suitability as a reservoir containment.

Category	Requirement	Qualification
Host Rock Characteristics		
a. Lithology	Candidate rock types include granite, granodiorite, diorite, gabbro, peridotite massive basalt, welded tuff, quartzite, marble, massive gneiss, dolomite and dense limestone. Homogeneous lithology is preferred.	Existing geological information. Surface mapping. Geophysical surveying. Exploratory drilling and core analysis.
b. Solubility	If a limestone or dolomite host rock is selected, its solubility in ground water and compensating water must not compromise the cavern over its design life.	Natural or artificial saturation of ground water and compensating water with carbonate and maintenance of a high pH may suffice to lower solution rates.
c. Thermal stability	The reservoir must be stable within the the temperature range imposed by CAES, i.e., 4 to 80°C.	Laboratory experiments will indicate host rock behavior.
d. Permeability	Host rock hydraulic conductivity must be less than 10^{-8} m/sec for water.	Host rock drill core samples will be tested at various locations and in various orientations. Special in situ evaluations of secondary permeability will be made with pump tests.
e. Rock strength	Unconfined compressive strength is to exceed 25 MPa over the cycling life.	Laboratory examination of replicate specimens from several orientations will qualify the host rock. An initial strength of 50 MPa or more is desirable.
f. Rock competence	Host rock must be competent enough to sustain mined out caverns supported by residual pillars and walls with minimum rock improvement measures.	Rock competence can be judged by measurement of "rock quality", numerical modeling and experimental. determinations of elastic, plastic and fatigue behaviors.

Category	Requirement	Qualification
g. Rock response	Host rock must not undergo slaking, spalling, geochemical alteration, thermo-mechanical fatigue or mechanical dislocation sufficient to cause local or general air leakage instability.	Numerical laboratory and field studies will be carried out to assure cavern stability and air containment. Economical rock improvement measures may be necessary in some rock environments or in areas of the underground system, such as intersections and manifolds. With adequate hydrologic conditions, slaking and spalling will not cause air leakage.
Structural Characteristics		
a. Joints and fractures	Planar openings must be widely spaced and relatively tight with discontinuities rare. Heavily fractured, jointed or faulted rocks will be excluded from consideration. Zones of weathering or mineralization are undesirable.	Exploratory drilling will produce oriented cores to measure joint attitudes, spacings and formation contacts.
b. Faulting	No historically active normal, reverse or horizontal displacement fault will be identifiable within the immediate host rock formation or within near associated formations.	Available geologic field information and seismic data will probably suffice. Vertical or inclined boreholes may identify a fault plane and its attitude.
c. Proximity to geologic contacts	The underground reservoir should not transect significant geologic contacts such as angular unconformities, other erosion surfaces, or igneous/country rock contacts. The nearest major contact should be not closer than 100 m. However, a change in the geology should not eliminate a site if the engineering and hydrogeologic requirements are satisfactorily met.	Regional mapping and subsurface mapping by drilling and geophysical methods will delineate both flat-lying and steeply-dipping contacts.

Category	Requirement	Qualification
d. Horizontal in situ stress	In situ stress is not to exceed vertical stress by more than a factor of 1.5. The limit is 0.037 MPa/m of depth for sedimentary rock, 0.040 MPa/m for igneous rock, and 0.042 MPa/m for metamorphic rock.	Vertical stress gradients range from 0.0282 MPa/m for metamorphic rocks to 0.0247 MPa/m for sedimentary rocks. Igenous rocks show an intermediate vertical stress gradient at 0.0269 MPa/m. In situ stresses will be measured by more than one device if. possible. Hydraulic fracturing may be the only practical method.
e. Complex geology	Areas with significant tectonic deformation or other crustal activity exhibited by tight folding, faulting, seismic activity, volcanism, excessive subsurface dissolution or subsidence will require extremely careful characterization and shall be avoided if possible.	Routine literature searching and geological surveying will reveal such areas via their complex outcrops water table disturbances, seismic events or gaseous emanations. Detailed exploration of prime sites will be required to assure cost effective and predictable design.
f. Orientation of caverns' longitudinal axes with respect to structural discontinuities and in situ stress state	■ along the line that bisects the maximum intersection angle between the two dominating sets of structural discontinuities (joints, bedding planes, foliation partings), but not parallel to the direction of a third or fourth minor set; at an angle of at least 25" from steeply dipping structural discontinuities that exhibit smooth surfaces or are filled with clay; at an angle of 15 to 30° to the direction of the maximum in situ stress in the horizontal plane (or parallel if required by other conditions arising from the orientation of the structural discontinuities);	Field and subsurface mapping and in situ stress measurements.

Category	Requirement	Qualification
f. (continued)	▪ at a maximum angle, and at least 35° to the strike of dipping foliation or bedding planes when the direction of the maximum horizontal in situ stress parallels the strike.	f. (continued)
Other Geological Characteristics		
a. Incompetent overburden	Sites beneath more than 50 m of incompetent water-bearing overburden should be eliminated.	Sinking of shafts through thicknesses greater than 50 m may be excessively costly. Shallow drilling will evaluate overburden competency.
b. Seismicity and volcanism	Areas with historical moderate to strong seismicity or volcanism will be disqualified.	Active faults and volcanic emanations, pyroclastic ejections or lava flows within a geologic province will eliminate sites within a radius of potential influence.
c. Mineability	The host rock shall be mineable by standard procedures with reasonable economy.	Mining experience in this rock type is to be investigated. Similar mining ventures will enable accurate cost estimating.
Design Parameters		
a. Cavern design	Temperature, pressure and humidity cycles are to be accommodated by the cavern design.	Experiments involving exposure of host rock specimens to cyclic physical CAES conditions will investigate geochemical reactions and thermo-physical fatigue.

Category	Requirement	Qualification
b. Cavern volume	Cavern volume is to exceed the maximum amount required for a generation period to allow for manifold and passage space not usable for storage. This extra amount would typically be about 15 to 20% over the nominal design volume.	Volume will be computed from surface plant requirements.
c. Cavern layout and configuration	Cavern layout and configuration should accommodate excavation/construction factors and result in an excavated volume at minimum cost. Configuration should account for appropriate shaft location, length of haulageways, turning radii, ventilation requirements, utility locations, and cost.	The method of excavation/construction is influenced by a combination of economics, geotechnical considerations equipment and contractor availability. The underground layout and cavern shape are also influenced by cost factors. In conventional heading and benching operations, excavation of headings is more costly than excavation of benches Thus, from this standpoint, cavern height to width ratios should be as large as possible. Also, a single cavern may be more economical than multiple caverns with the same volume, but a single cavern may extend the schedule by minimizing the number of working faces. However, additional consideration must be given to the length of haulageways, ventilation requirements, utility locations, etc. leading to an optimal design situation.
d. Cavern geometry, size, number	Cavern shape, size and number will be determined by host rock stability and economics of mining and rock improvement measures.	Field experience, in situ measurement of horizontal stresses and numerical modeling will contribute to underground reservoir design.

Category	Requirement	Qualification
e. Depth to cavern roof	750 to 850 m, depending on the turbo-machinery[a]	At 750 and 850 m, pressures of compensating water columns will be 7.36 and 8.34 MPa, respectively. The hydrostatic head of the compensating water column will determine the operating pressure.
f. Excavation methods	Excavation is to be performed to minimize cost and preserve formation integrity.	Field experience will determine excavation methods, e.g., smooth blasting, boring, etc.
g. Air loss rate	Air loss is not to exceed one percent during the daily storage period.	Initial air injection experiments will qualify the cavern or disclose global air leakage instability. Cavern boundaries may require specific load sealing of larger scale fissures or other permeable zones.
h. Compensated vs. uncompensated	The reservoir will be water-compensated.	Economics of excavation requires minimum design volume.
i. Champagne effect	Uncontrolled transients resulting from rapid evolution of dissolved air in the compensating water column must be prevented. A U-bend below the cavern bottom whose depth equals 13% of the reservoir depth is one proposed solution.	Numerical and experimental studies are evaluating the magnitude of this effect. Design countermeasures can be incorporated if necessary. The critical parameter is the concentration of air in the compensating water. Daily operations are not likely to be affected. Programmed operations after long air-charged shutdowns can prevent the transient behavior.

[a] Because of current machinery limitations and pressure losses during withdrawal, a maximum pressure of 7.61 MPa is desirable. This would require a cavern depth of 777 m.

Category	Requirement	Qualification
j. Rock improvement measures	Limited rock improvement measures may include grouting, shotcreting, rock bolting and rock netting.	Rock improvement costs may seriously impact CAES economics if rock quality requires medium to heavy support.
k. Cavern life	Cavern operating life is to be 30 years or longer, i.e., about 10,000 cycles.	Numerical and laboratory tests of the host rock and cavern geometry should simulate 10,000 cycles to verify this life.
1. Location of generation facility	Cavern is to be located within feasible distance of the surface plant.	Calculations of two-way pipe friction, heat loss, plumbing costs will affect the maximum horizontal distance. Actuation time delay may be important.
m. Charging pressure	Maximum charging pressure will be 12.0 kPa per meter of depth. This number is determined by the difference between cavern and compensating reservoir elevations with allowance for system pressure drop, inertial displacement of water, and control losses.	The least lithologic vertical pressure gradient will be 22.63 kPa per meter of depth. The hydrostatic head will be about 10 kPa per meter of depth.
Operating Parameters		
a. Operating pressures	Operating pressures will be nearly constant for a particular cavern, i.e., about 7.35 to 8.33 MPa for respective cavern depths of 750 and 850 m.	Design of surface plant turbines will require nearly constant pressure. Maximum storage pressure will be about 10.0 kPa per meter of depth, which is equal to the hydrostatic pressure for slightly saline water.

Category	Requirement	Qualification
b. Operating humidity	Operating humidity will be determined by the initial humidity of the compressed air and the secondary added humidity caused by evaporation of the compensating water. This will be near saturation.	No specific lesser humidity requirement has been identified.
c. Inlet air temperature	Inlet air can vary in temperature from 30 to 80°C.	Temperature can be continuously and redundantly measured at the cavern crowns.
d. Compensating water temperature	Compensating water may fluctuate between 0 and 30°C.	Compensating water temperature will be determined by surface temperature, conduit/rock temperature and CAES temperature. Temperature should be continuously monitored at several levels.
e. Injection-withdrawal cycle	Weekdays - up to 24 hours of injection and withdrawal. Weekend cycles may be somewhat longer. Will provide up to 12 hours of electricity generation per weekday for a typical peaking cycle.	Determined by electrical load.
f. Compressive vs. tensile	Cavern design should ensure compressive stresses tangent to the cavern boundaries.	Stressmeters and strainmeters should be utilized at key locations.
g. Despressurization	Ordinarily the reservoir can not be subject to rapid depressurization.	Rapid depressurization could endanger cavern integrity. In a compensated cavern air removal is accompanied by water entry. In the event of water exclusion, depressurization should not exceed 1 MPa per hour.

Category	Requirement	Qualification
h. Accidental overcharging or over decharging	Cavern storage volumes must be sufficient to meet the plant's energy storage requirement while at the same time providing both air and water buffer volumes. Buffer air and water volumes must be sufficiently large to keep air and water velocities to less than 1 m/s.	These volumes should be large enough to keep water and air velocities to a minimum to 1) prevent erosion of the cavern floor and subsequent plugging of connecting water shafts, and 2) prevent the water from being churned such that water particles become suspended in the air and carried to the turbine. Positive water level instrumentation will be provided.

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APPENDIX A

CASE HISTORIES OF CAVERN AIR STORAGE

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The Flygmotor CAES cavern system near Trollhatten, Sweden is used to supply compressed air for wind tunnel testing of aircraft engines (Bergman, Lundberg, and Windelhed 1979). The system was excavated in a biotite gneissic rock mass at a depth of about 90 m during 1949-51 and has a volume of approximately 12,000 m^3 of usable storage. The system is water-compensated and, due to the low (3.8 m) height of the individual caverns, essentially provides a constant air pressure source. The cavity is normally charged five to six times a day and has experienced approximately 30,000 unload-load cycles since construction.

Following cavity excavation, a tightness test was performed in September 1951. The cavern was pressurized to 830 kPa without water compensation and the pressure drop was monitored for 24 hours. This test was repeated twice; the results are listed below.

Test No.	Pressure Drop kPa/day	Air Leakage %/day
1	29.4	3.6
2	27.0	3.3
3	39.2	4.8

Although these air leakage rates are high and might well be unacceptable in most CAES applications, they were acceptable to Flygmotor.

An air leakage model was developed and used to simulate the air tightness test of the Flygmotor CAES cavern system (Brandshaug and Fossum 1981). Quantitatively, the results for leakage were somewhat greater than actually occurred during the tests. However, coefficients of lateral earth stress could not be obtained and had to be assumed based on typical measured values in hard rock at a depth of about 100 m. Also, two values of permeability were available. The empirical permeability/stress relationship was derived with a bias toward the greater values of permeability. Quantitatively, air leakage simulations made with the use of stress-dependent permeability relationships showed that the coefficient of lateral earth stress had only minor influence on the volume of air leakage. However, the location of leakage in the cavern is strongly dependent on the in situ stress ratio, with the higher value giving rise to a greater loss of air through the cavern walls.

Aberg (1977) and Lindblom et al (1977) have described Swedish experience in storing LPG and air in excavated reservoirs. Rock caverns must be tighter for LPG storage than for compressed air storage. Even so, a large number of excavated compressed air energy storage chambers could be used for only short times because of excessive leakage. However, those that had connecting drifts as well as concrete plugs tightened with water locks could be used to store air with pressures ranging from 0.7 to 1.2 MPa. The volumes of these chambers ranged from 1000 to 5000 m^3. The leakage from chambers constructed in this manner has been small, provided that the surrounding rock mass was not drained of water or appreciably affected by the mining procedures. For example, careful measurement of the leakage of a 5000-m^3 compressed air chamber in the Zinkgruvan mine showed leakage of 0.11% per day at pressures ranging from 0.7 to 0.75 MPa.

A recently constructed Swedish underground storage plant for LPG was equipped with a "water curtain", a special tightening arrangement using water-filled drillholes above the chamber roofs. All cracks and pores in the rock mass surrounding the cavern are filled with water at a pressure higher than the gas pressure of the storage chamber. The flow of water in the rock mass must be toward the storage chamber with a hydraulic gradient greater than 1. To prevent draining the surrounding rock of water during excavation, systems of drillholes and drifts are arranged above and around the chamber prior to blasting. This injection system must then be kept pressurized with water.

The gas tightness of this storage plant was tested by pressurizing it with air and measuring the pressure, temperature, and humidity as a function of time. With the use of these measurements, the leakage rates for a 24-hour period were calculated for air pressures of 360, 500, and 700 kPa. These leakage rates were determined to be 0.0059%, 0.0066%, and 0.0071% per day, respectively.

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APPENDIX B

POTOMAC ELECTRIC POWER COMPANY FEASIBILITY STUDY .

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POTOMAC ELECTRIC POWER COMPANY FEASIBILITY STUDY

The findings of a major study, sponsored and financed jointly by the U.S. Department of Energy and the Electric Power Research Institute (EPRI), into the technical, economic and environmental feasibility of underground energy storage were presented by Willett and Curtis (1980). The program, undertaken under the general direction of the Potomac Electric Power Company of Washington, D.C., was concerned with two energy storage concepts, underground pumped hydro (UPH) storage and compressed air energy storage (CAES). Both concepts require large caverns excavated in rock at depths ranging up to 1500 m (5000 ft) below ground level. Typically, the most economic pressures at which to store the air lie between 4.8 and 7.6 MPa (600 and 1100 psi). At 6.9 MPa (1000 psi) the volume of air required to provide 10,000 MW hours of storage (1000 MW for 10 hrs) amounts to approximately 690,000 m^3 (900,000 yd^3).

SITE INVESTIGATION

The site investigation program undertaken to assess the subsurface conditions was completed in two phases. The first phase included 10 shallow boreholes to depths less than 30 m (100 ft) and two additional boreholes completed to depths of 150 m (500 ft). The second phase comprised a single deep borehole to the proposed cavern depth with continuous core recovery and logging, determination of physical and mechanical properties of the rock, geophysical logging, permeability measurements, and in-situ stress determination using hydraulic fracturing (Willett and Curtis 1980).

The shallow drilling phase showed the Sykesville boulder gneiss to be overlain by 6 to 23 m (20 to 75 ft) of residual soii. The upper zone of this consists of medium-grained dense sandy silt overlying a highly deconiposed rock zone with gradual transition to the underlying

Sykesville. Rock cores (NX size) obtained from the two 152 m (500 ft) boreholes were often extracted in lengths of 3 m (10 ft) and showed RQDs of 75 to 100%. In most areas below the upper 30 m (100 ft) of weathering the hydraulic conductivity was measured to be 10^{-5} to 10^{-7} cm/sec. The boulder gneiss is intensely foliated with the plane of foliation dipping at between 60° and 70" to the northwest.

The primary objectives of the deep borehole were to confirm the existence of the Sykesville formation to a depth of 1523 m (5000 ft) and to determine the suitability of the Sykesville for the UPH or CAES facility. Because of the limited funds available for the exploratory program within the terms of reference of the overall study, it was decided to limit the final cored size of the drill hole to the minimum required for the use of geophysical logging and hydrofrac equipment. A borehole diameter not less than 75 mm (2.98 in) was therefore selected after extensive discussions with drilling companies and geophysical equipment suppliers. However, difficulties in maintaining vertical alignment resulted in termination of the hole at 998 m (3274 ft) measured along the borehole with a deviation of 53° from vertical. This corresponds to a vertical distance of 780 m (2556 ft) below the surface.

Geophysical logging was performed in the uncased section of the borehole with measurement of relative density, natural gamma radiation, temperature, hole diameter and three-dimensional sonic velocity. Elastic properties calculated from the geophysical logging shows a modulus of from 66 to 83 MPa (9.5 to 12 ksi) and Poisson's Ratio of from 0.28 to 0.31. The temperature at the base of the hole was 23°C (73°F) with a thermal gradient of 1.8°C/100 m (1°F/100 ft).

Hydraulic fracturing indicated that the minimum stress in the plane perpendicular to the borehole was 1.1 times the weight of the overburden (yz) and was oriented parallel to the strike. In the same plane the maximum stress, perpendicular to the strike, was measured as 1.8 yz. At the location of the hydraulic fracturing tests, the borehole is approximately perpendicular to the plane of foliation, so it is considered that the stresses measured are two of the principal stresses with the third located parallel to the borehole. Calculation of this minimum principal stress using the assumption that the vertical stress is equal to the overburden pressure leads to a negative value. This is obviously not the correct interpretation. A more reasonable assumption would be that the minimum principal stress is equal to or somewhat less than the intermediate stress.

Initial testing of rock core from the two 152-m (500-ft) boreholes showed highly anisotropic strength properties. Subsequently, the testing program was expanded to include several triaxial tests at various confining pressures and angles of confinement. The unconfined compressive strength ranged from a low of 57 MPa (8300 psi) with the foliation inclined at an angle of 45° with the maximum principal stress to a high of 100 MPa (14,600 psi) with an angle of foliation of 0 or 90°.

GEOTECHNICAL CONSIDERATIONS

For both the UPH and CAES schemes, the cavern arrangement consists of a series of large parallel caverns connected at opposite ends by water and air collector tunnels. The parallel caverns are oriented in the most favorable direction geotechnically, which has been determined to be in a northwest-southeast direction. These caverns will be parallel with the maximum horizontal stress. Thus, stress concentrations around the caverns will be due to the minimum horizontal stress and vertical stress which act in the plane of the cross section. Additionally, the caverns will be perpendicular to the strike (of the foliation), which is generally considered to be the optimal orientation for construction (Willett and Curtis 1980).

Because the large caverns are in the most favorable direction with respect to the in situ stresses as well as the foliation, the collector tunnels will consequently be in the least favorable direction with respect to these two factors. Because of this, and because the several intersections will be located along the collector tunnels, these tunnels have been designed to be smaller than the main parallel storage caverns.

At intersections there will be minimal confining stress in the walls together with large stress concentrations; therefore, stability will very likely be a problem. These areas will require considerable support. The use of a few relatively large caverns will reduce the number of intersections necessary.

The quantity of air leakage from the CAES caverns will be directly related to the rock mass permeability and will increase operational costs for each system. As an example, a 2% per day air leakage rate would result in an additional annual levelized compression power cost in excess of $1,000,000. Although relative to permeability the Sykesville should be considered a fractured medium on a global basis, calculations can be based on the equivalent permeability of a homogeneous medium. The permeability testing from the deep drilling program indicated an average permeability less than 10^{-12} cm^2 (hydraulic conductivity equals 10^{-7} cm/sec). Using these values of permeability, four models were used to estimate the air leakage rate. Leakage rates ranged from a low of 0.02% of the cavern volume per day, assuming one-dimensional flow, to a high of 1.5% of the cavern volumes per day assuming three-dimensional flow with each cavern acting independently. These rates are considered acceptable without major remedial action. However, the cost estimate included provision for grouting of any open joints that may be found during excavation.

Construction of the CAES scheme is estimated to require 4-1/2 years. The entire underground cavern system could be completed in 4 years. It is estimated that 1-3/4 years will be required for shaft sinking and 2-1/4 years for cavern excavation. The cost of live storage exclusive of shaft costs is estimated to be $\$53/m^3$ ($\$41/yd^3$). All costs are quoted in June 1979 U.S. dollars (Willett and Curtis 1980).

NUMERICAL MODELING

Brandshaug and Fossum (1981) modeled the Potomac Electric Power Company (PEPCO) caverns using both isotropic and anisotropic properties to confirm the architect-engineering rock mechanics design. The use of anisotropic elastic properties did not alter the state of stress around the caverns significantly compared to that involving isotropic elastic properties.

Application of a guide for required artificial support (that considers the elastic stress state only) suggested no support would be needed for the cavern walls; light support such as rock bolts, wire mesh, and shotcrete would be required for the cavern roof.

This requirement was confirmed by the results of the failure analyses, which showed limited failure along joints in the roof and the floor. The thermoelastic analysis revealed no tensile stresses.

The overall PEPCO analysis strongly indicates that cavern stability can be maintained with a minimum requirement of artificial support both before and during CAES operations.

In general, this program showed that adequate information is available to enable design of an underground opening with the size, shape, and depth required of a CAES cavern. This information is based on a combination of numerical modeling and precedent considerations. The design methodology can provide a guide to support requirements for caverns of different shapes. The design assumes excavation in a rock mass with relatively well-quantified rock properties and associated geotechnical characteristics.

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