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Workshop on Rock Mechanics Issues in Repository Design and Performance Assessment

Held at
Holiday Inn Crowne Plaza
Rockville, Maryland
September 19–20, 1994

Prepared by
Center for Nuclear Waste Regulatory Analyses
Southwest Research Institute

Prepared for
U.S. Nuclear Regulatory Commission

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ABSTRACT

The Center for Nuclear Waste Regulatory Analyses organized and hosted a workshop on "Rock Mechanics Issues in Repository Design and Performance Assessment" on behalf its sponsor the U.S. Nuclear Regulatory Commission (NRC). This workshop was held on September 19–20, 1994 at the Holiday Inn Crowne Plaza, Rockville, Maryland. The objectives of the workshop were to stimulate exchange of technical information among parties actively investigating rock mechanics issues relevant to the proposed high-level waste repository at Yucca Mountain and identify/confirm rock mechanics issues important to repository design and performance assessment. The workshop contained three technical sessions and two panel discussions. The participants included technical and research staffs representing the NRC and the Department of Energy and their contractors, as well as researchers from the academic, commercial, and international technical communities. These proceedings include most of the technical papers presented in the technical sessions and the transcripts for the two panel discussions.

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INTEGRATING ROCK MECHANICS ISSUES WITH REPOSITORY DESIGN THROUGH DESIGN PROCESS PRINCIPLES AND METHODOLOGY

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ABSTRACT: A good designer needs not only knowledge *for* designing (technical know-how that is used to generate alternative design solutions) but also must have knowledge *about* designing (appropriate principles and systematic methodology to follow). Concepts such as "design for manufacture" or "concurrent engineering" are widely used in the industry. In the field of rock engineering, only limited attention has been paid to the design process because design of structures in rock masses presents unique challenges to the designers as a result of the uncertainties inherent in characterization of geologic media. However, a stage has now been reached where we are be able to sufficiently characterize rock masses for engineering purposes and identify the rock mechanics issues involved but are still lacking engineering design principles and methodology to maximize our design performance.

This paper discusses the principles and methodology of the engineering design process directed to integrating site characterization activities with design, construction and performance of an underground repository. Using the latest information from the Yucca Mountain Project on geology, rock mechanics and starter tunnel design, the current lack of integration is pointed out and it is shown how rock mechanics issues can be effectively interwoven with repository design through a systematic design process methodology leading to improved repository performance. In essence, the design process is seen as the use of design principles within an integrating design methodology, leading to innovative problem solving. In particular, a new concept of "Design for Constructibility and Performance" is introduced. This is discussed with respect to ten rock mechanics issues identified for repository design and performance.

1 HLW DISPOSAL TODAY: *WHAT* INTEGRATION?

Nothing can illustrate better the current lack of integration of rock mechanics and geotechnical issues (including engineering geology and rock engineering) with repository design than the fact that during the recent 5th International High-Level Radioactive Waste Management Conference held in May in Las Vegas, the two sessions on "Rock Mechanics Issues" and "Repository Design" were held on the same day, at the same time, in rooms far apart! As a result, and reflecting the current situation at the Yucca Mountain Project, there was hardly any interaction between the two groups, poor attendance was evident at both meetings (because of split audiences) and little feedback was derived to benefit either program of activities. My paper (Bieniawski, 1994) was at the design session and while it generated considerable discussion, none of the questions from the audience was related to any rock mechanics issues!

How is this lack of integration possible? Or, is the whole process of site characterization and repository design so fragmented, so uncoordinated and so out of line with the current world-wide technology that it is becoming an embarrassment? And how is it possible that this country, which has the most distinguished rock mechanics and underground excavation design experts in the world, has reached a stage that - in the words of *The New York Times* (March 4, 1994) and *Nucleonics Week* (June 24, 1993) - "DOE has build nothing, is 12 years behind schedule, and it has spent $1.2 billion so far on HLW". Moreover, the prestigious journal of the National Academy of Sciences *Issues in Science and Technology* (summer issue 1994) in an article "Time to Rethink Nuclear Waste" recommended that DOE should be removed by Congress from any part in the nuclear waste disposal. Most recently, the new OCRWM Director Daniel Dreyfus stated (ASCE News, June 1994): "it's time to develop a game plan after a decade of inaction". A decade of inaction! This is clearly a crisis situation!

So, what can and should be done about this problem?

Fortunately, the Workshop is a proof that integration of rock mechanics issues with repository design is considered vital for the whole HLW disposal program. The organizers must be complimented for providing this forum for exchange of technical information and for identifying the issues important to the realization of the project. However, it remains to be seen whether any recommended action will actually take place because, as pointed out by the Nuclear Waste Technical Review Board (NWTRB, 1993), an overall DOE strategy is still missing for the project management.

This paper builds on over 12 years of my involvement in the HLW program and has three aims: (1) to point out the precarious status quo that has developed in the area of rock mechanics-repository design interaction, (2) to present design process principles and methodology for rock engineering as a means for integrating geotechnical issues with repository design and construction, and (3) to use the case history of the north ramp tunnel at Yucca Mountain as an example of the problems encountered and how they may be overcome by a comprehensive design methodology.

2 STATUS QUO: CHANGING OBJECTIVES, STARTLING COSTS, LACK OF INTEGRATION

In considering rock mechanics issues in repository design and performance assessment, one must first take a broader look into the overall objectives of the HLW program, its characteristics and status quo. Only then, can the role of rock mechanics be effectively identified for repository design, including the mode of determination and utilization of thermal and seismic loads.

2.1 Program Objectives and Role of Rock Mechanics

The National Academy of Sciences (NAS, 1991) pointed out that the US program of HLW disposal is unique in its rigid schedule, in its insistence on defining in advance the technical requirements for every part of the multibarrier system, and in its major emphasis on the geological component of the barrier. In essence, nuclear waste management is a tightly regulated activity, hedged with laws and regulations, which call for detailed predictions of rock behavior for tens of thousands of years, longer than recorded human history.

NAS stated further that the US program as conceived and implemented over the past decade, *is unlikely to succeed because it is poorly matched to the technical task at hand.* This is because the program assumes that the properties and future behavior of a geologic repository can be determined and specified with a high degree of certainty. In reality, the inherent variability of the geologic environment will necessitate frequent changes in the specifications.

Another independent body, established by Congress in 1987, is the Nuclear Waste Technical Review Board (NWTRB). Its purpose is to evaluate the technical and scientific validity of activities undertaken by DOE and, in the process, comment on the appropriateness of NRC regulations and EPA standards. In its reports (NWTRB, 1993), this Board also expressed many concerns and made far-reaching recommendations which could significantly improve the US nuclear waste disposal program. It pointed out that the standards should not impose restrictions that would *foreclose at the outset a candidate site subsequently shown to be suitable based on sound scientific considerations.* In fact, NWTRB also stated: *"The Board has seen no evidence to indicate that a waste package lifetime of 10,000 years, and perhaps more, would be unattainable in the United States program."* In this case, the required redundancy can be directed to the design of the repository, the main role of which will be to maintain retrievability of the waste for 85 years (including the time for repository construction, waste placement and decommissioning). Furthermore, NWTRB (1993) identified an urgent need to develop a comprehensive methodology for repository design and performance assessment of the overall engineering barriers concept.

Currently, about 22,000 tons of waste sit at 60 utilities in 40 states and will double by the year 2010 when a permanent repository is to be ready to receive the nuclear waste. Additional waste from US defense facilities, also intended for the repository at Yucca Mountain, should be up to 8,800 tons by then.

2.2 Progress and Costs

The ten-year task of characterizing the Yucca Mountain site at a cost of $6 billion has only really started last year, due to severe delays when the state of Nevada denied air-quality permits for dry drilling at the site.

However, as reported in *NuclearFuel* (May 23, 1994), DOE spent some $4.76-million constructing what some have called the most expensive tunnel ever built. The 198ft long, 30ft wide, 33ft high starter chamber of the north ramp at Yucca Mountain, started on April 2, 1993, cost about $24,000 per foot, or an extraordinary $2,000 an inch! This cost ($2.83-million for excavation and $1.93-million for rock bolts and shotcrete) is about four times the cost of a similar project in the private sector and would take about two months to complete, instead of six months. Critics said that the high cost of the starter tunnel was due to it being excavated by drilling and blasting rather than by machine boring, due to a major problem of poor interface between the contractors, and due to it being a government project on a cost-plus fee, riddled with regulations and special requirements.

August 8, 1994 was the target date to begin the TBM excavation of the 25ft diameter and 5 mile long main loop forming a part of the exploratory studies facility (ESF). The loop is to be completed in two years, with all the other excavations being done a year later (mid-1997), all totaling 13.6 miles of underground openings. The north ramp of the ESF is to be completed in 12 months.

One of the matters affecting rock mechanics issues is that the old ESF requirements for the in situ tests were compiled at a time when DOE envisioned using drilling and blasting, rather than TBM, to construct the test and disposal facilities. Thus, the need for some in situ tests may be questioned as was already pointed out by the NWTRB (1993), at whose urging DOE redesigned the ESF to use ramps (instead of shafts) and a TBM. Moreover, the method of excavation of the test alcoves is still unsatisfactory because the Design Package 2C specifies drill and blast construction in its 90% review document.

In another development, DOE is considering doubling waste retrievability requirements to 100 years and shifting some of the test work to the post-licensing, confirmatory period. The management and operations (M&O) contractor, TRW, announced that its repository design team is designing 100-year rock bolts for the ESF and repository.

Overall, the program is in a state of flux because future funding, costs and planning strategies are uncertain until Congress decides program budget for FY 1995 (OCRWM requested $532.2 million). Currently, the program costs over $1 million per day (FY 1994 budget: $381 million) and employs 2,790 people spread among a dozen major and some two-dozen minor contractors, several national laboratories, various government agencies and others (NWTRB, 1993).

Finally, it must be recognized that the SCP Conceptual Design Report of 1987 is out of date with the emergence of these leading concepts:

- long-lived high-capacity, robust waste package (MPC);
- MPC (multi-purpose container) sealed at reactor site for final disposal;
- in-drift instead of borehole emplacement of waste packages;
- thermal-loading strategies considerably different from the previous 57 kW/acre and 10 year old spent fuel: now "extended dry" is the lead concept of heat management, with age of waste being 30 years.
- repository capacity considerably greater than the original 77,000 tons of waste;
- ramp instead of shaft access to ESF;
- tunnels with considerably smaller dimensions; and
- TBM excavation instead drilling and blasting.

In addition, it is important not to consider the 10,000 years waste isolation criterion as dictating the rock mechanics and repository design issues. This criterion could soon be substantially changed for the Yucca Mountain Project because the new EPA regulations stipulate that the 10,000 years requirement, while now otherwise applicable, does not apply to the Yucca Mountain Project for whom, as directed by Congress, a separate standard for spent fuel and HLW disposal will be developed under the guidance of the National Academy of Sciences.

2.3 Possible Issues for Rock Mechanics - Repository Design Interaction

The new OCRWM program scenario includes:

• development and initial procurement of the multi-purpose container (MPC) which could be used for both transport and storage of spent fuel.

Implication for rock mechanics issues in design: different input data requirements for excavation design due to change from borehole emplacement to drift emplacement.

• downsizing of the underground and surface based testing programs.

Implication for rock mechanics issues in design: rock mechanics tests and design requirements must be revised and prioritized.

• phased-licensing approach relying on post-emplacement testing.

Implication: a new program of confirmatory rock mechanics and design verification tests will need to be initiated.

• extending the period of waste retrievability to 100 years which, including repository construction, waste emplacement and closure, may be ~ 150 years.

Implication: maintaining long-term tunnel stability will require new approaches to stability analyses and to design of rock bolts and other support.

According to the OCRWM, the objectives of the underground exploration and testing program at Yucca Mountain are:

- provide a complementary suite of investigations meeting site-characterization and site-suitability criteria.
- provide geological information.
- examine the in situ effects of imposed conditions (excavation, thermal loading) on natural geologic structure.
- assess site suitability and provide critical design support.
- provide underground access to specific geologic features and for in situ tests.

To meet these objectives, a suite of 42 tests were defined by the OCRWM in 1992, with special emphasis on the unsaturated zone testing. However, plans for thermal testing in the underground core area have not been adequately developed (NWTRB, 1993). Yet, a test plan is necessary before the design of the core test area can be finalized and, since the ESF will be a part of the repository, a repository conceptual design is also required featuring both high and low thermal loading options. NWTRB(1993) stated *"integration is lacking among those working on plans for underground in situ thermal testing, repository design and waste package development"*. This lack of coordinated planning of the in situ test program in the core area seems quite unexplainable bearing in mind the vast experience available from design investigations associated with large underground projects, in this country and around the world (USNCTT, 1984; Bieniawski, 1976).

2.4 Priorities for Action

With respect to rock mechanics issues involved in repository design and performance assessment, the preceding discussion clearly indicates that:

• the Yucca Mountain Project lacks an overall strategy for exploration and testing which could integrate testing priorities with design and excavation approach for the ESF.

• the current design of the core test area is highly complex, as shown in Figure 1, while using drilling and blasting for test alcoves excavation, as proposed by the M&O in Design Package 2C, is inappropriate. The core test area should be redesigned (simplified) for excavation by a TBM).

• the design of the exploratory facility should be compatible with potential repository designs because if the Yucca Mountain site is found suitable, the exploratory facility is to be integrated in the repository design.

• there is a wealth of technical expertise and industrial experience from which the Yucca Mountain Project could benefit more effectively. Discussions with DOE staff and contractors have revealed that many of them are unaware of the classic rock engineering and underground excavation design projects, such as those discussed by Hoek (1980), USNCTT (1984), and by Bieniawski (1992).

• the considerable experience ("painful" at times) gained from rock mechanics testing at the Basalt Waste Isolation Project (BWIP) as well as the DOE-initiated License Application Review of BWIP (LAR-1989) - performed by a group of international experts* and well documented - have not been utilized in any way in the planning of the ESF activities.

• the changing of administrations, as well as of the main DOE contractor (M&O) has led to accusations of "a decade of inactivity" (although institutional, i.e. DOE's, responsibility should be continuous) disregarding what was achieved at BWIP and earlier at YMP. Also, many good ideas were lost involving design and construction efficiencies developed by the pervious contractor (NWTRB, 1993).

It is believed that the above problems can be solved more effectively and the priorities identified more easily if use is made of a comprehensive and integrating design methodology, which incorporates design principles, on the basis of which the whole design and construction process could be evaluated and properly integrated system-wide. Such a design methodology is described in the next section.

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* Bartlett(chair), Bieniawski, Blake, Brekke, Cook, Domenico, Hoek, Lerman, Pigford, Poeter, Salamon, and Smith.

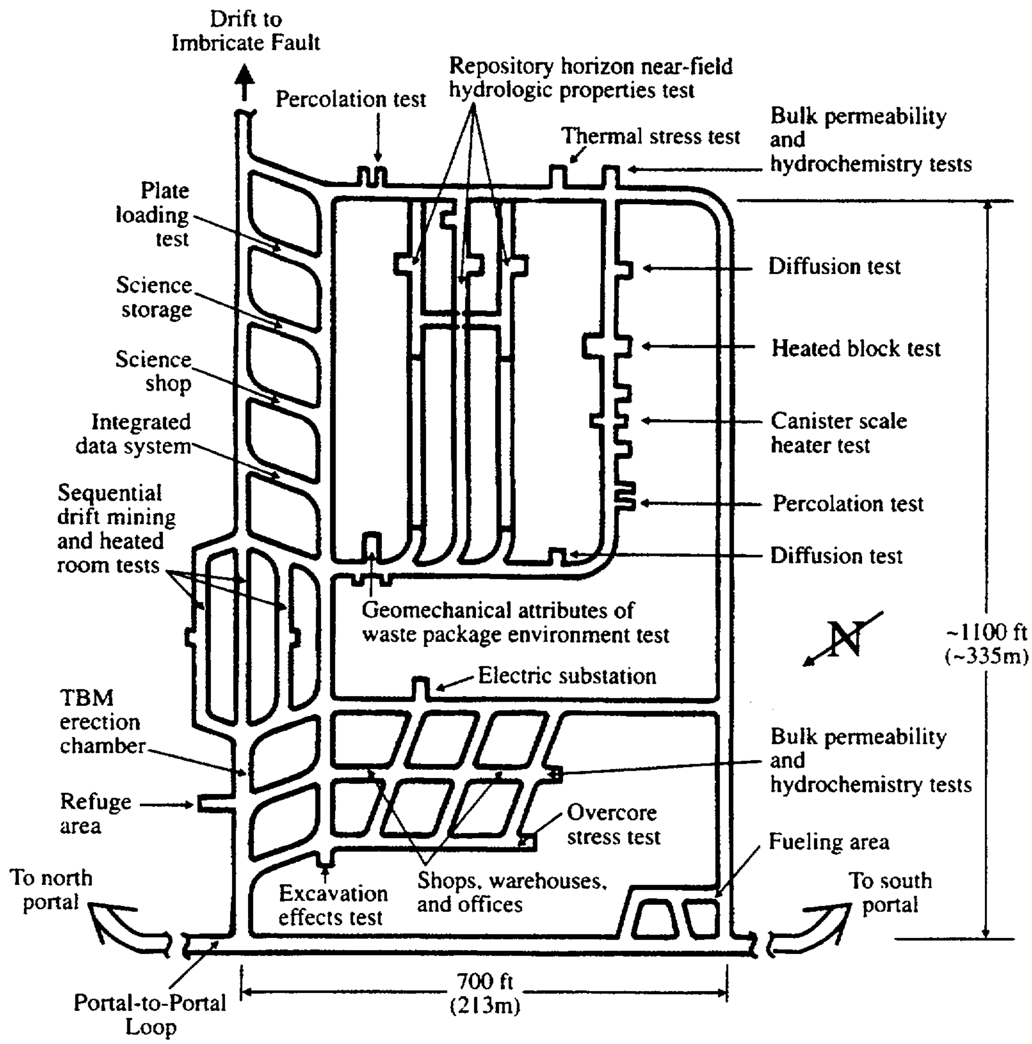

Figure 1 Currently proposed core test area, in plan view (NWRTB, 1993).

3. SYSTEMS DESIGN METHODOLOGY FOR ROCK ENGINEERING

A good engineering designer needs not only knowledge *for* designing (technical knowledge that is used to generate alternative design solutions) but also must have knowledge *about* designing (appropriate principles and systematic methodology to follow). Concepts such as "design for manufacture" or "concurrent engineering" are widely used in the manufacturing industry.

In the field of rock engineering, only limited attention has been paid to the design process because design of structures in rock masses presents unique challenges to the designers as a result of the uncertainties inherent in site characterization. Nevertheless, the importance of the design process in tunneling has been widely recognized, particularly abroad (Duddeck, 1989).

Recently, Gupta et al. (1991) described the design control process requirements, which are a measure of whether (i) applicable regulatory requirements and the design basis are correctly translated into the design, (ii) design interfaces are identified in the design, and iii) the adequacy of the design is verified. Foster (1991) elaborated on design reviews from a regulative perspective and listed the generic design review criteria as well as requirements for systems engineering. The latter advocates a top-down design approach, meaning that high-level functions have to be decomposed into discrete, implementable system elements. This would lead to vertical and horizontal traceability between functions, constraints and design solutions. These design objectives should be integrated into an effective design methodology.

Further discussions of the design control overview was provided by Cikanek and Petrie (1992), Blejwas (1992) and by Bullock and Grenia (1992). However, evidence that design review was actually performed, as discussed in these papers, is very difficult to trace. In fact, NWTRB (1993) complained that "at the recent 90% design review, of 41 review team members, all were employees of the DOE or under contract to the DOE and few had experience on projects using TBMs".

Nevertheless, the need for a logical design procedure for the Yucca Mountain Project was recognized by both the Nuclear Regulatory Commission (NRC) and the Department of Energy (DOE), resulting in their own methodologies. Nataraja et al. (1992) aimed to develop a design methodology to satisfy the regulatory aspects, taking the effects associated with thermal loads into account. In essence, the NRC provided a flowchart to incorporate thermal effects on the host rock, surrounding strata, and the ground water system. In addition, the DOE was challenged by the NRC staff to develop and use "a defensible methodology to demonstrate the acceptability of the underground facility design".

A drift design methodology for the YMP was developed by Hardy and Bauer (1991) for the Department of Energy. They presented a methodology to assess drift stability by determining whether the rock mass strength sufficiently exceeds the applied stresses. This methodology was structured into two parts, (1) a preliminary drift design, focusing on functional requirements, local geologic conditions, and evaluation of unsupported drifts, and (2) design of a compatible ground support system for drill-and-blast drifts. At the time this methodology was developed (in 1989), it was an important step forward but by now that approach is out of date, for reasons discussed in Section 4. Yet, a follow-up report was prepared (Lin et al., 1993) and the same old approach is still advocated by the DOE and the A/E in the Design Package 2C. Thus, DOE has not come up with a defensible methodology, as recommended by the NRC staff.

However, a comprehensive, well-integrated, design methodology for a HLW repository is available since three years ago, as discussed below, and is a systematic decision-making process directed at an improved design of underground structures in rock, including a nuclear waste repository. Known as the Systems Design Methodology for Rock Engineering (SDM), it was developed under an NSF project specifically for excavations in geologic media recognizing that, unlike other engineering materials, rock mass formations have unique and complex characteristics.

3.1 Design Principles for Rock Engineering

A comprehensive design methodology is not just a sequence of flow charts for step-by-step design. To be comprehensive, a design methodology must incorporate design principles which can be used to evaluate designs and to select the optimum one fulfilling the perceived objectives. To obtain a better performance, one requires correct design principles and methodologies to guide decision-making in design. After all, there are unacceptable design solutions as well as good design solutions, so there must be a fundamental set of principles determining a good design practice. Without them, design would be a mysterious creative process; with them it can be a rational and systematic activity. A design methodology must indeed recommend an order of design stages but these must be so structured as to assist in effective decision making and promote design innovation in accordance with the appropriate design principles.

Recently, Suh (1990) of MIT developed an axiomatic approach to mechanical engineering design, identifying design axioms which constitute the basic principles for analysis and decision-making, and help the creative process of the design activity. In essence, Suh proposed just two principles of design, each pertinent to its own domain. In the *functional* domain, we ask: "what do we want to achieve"? In the *physical* domain, we must provide the actual design solution by answering: "how do we want to achieve it"? The significance of this approach is that it provides two specific principles for distinguishing between a poor design and a good design. Suh's work paved the way for proposing further design principles, specifically for rock engineering, and incorporating them all in a specific design methodology.

There are six principles of design recommended for rock engineering (Bieniawski, 1993), as the principles for developing, evaluating and optimizing alternative designs. Thus, while Suh's two principles are considered necessary, they are not sufficient for rock engineering design because the behavior of rock masses is governed by the geologic environment which imposes special constraints not found in other branches of engineering. These principles are:

#1 Independence Principle: There exists a minimum set of independent functional requirements that completely characterize the design objectives for a specific need (Suh, 1990).

#2 Minimum Uncertainty Principle: The best design is one which poses the least uncertainty concerning geologic conditions; the geotechnical data obtained from site characterization must be traceable to the design objectives and performance requirements as well as to the design solution (Bieniawski, 1993).

#3 Simplicity Principle: The complexity of any design solution can be minimized by creating the fewest number of design components forming a part of the design solution and corresponding to the appropriate functional requirement (Suh, 1990). The data from site characterization should be traceable to the design solution.

#4 State-of-the-art Principle: The best design maximizes technology transfer of the state-of-the-art research and the best industrial practice (Bieniawski, 1993).

#5 Optimization Principle: The best design is the optimal design which is evolved from quantitative evaluation of alternative designs as well as on quality assurance and cost effectiveness considerations (Bieniawski, 1993).

#6 Constructibility Principle: The best design facilitates the most efficient construction of a rock engineering structure by enabling the most appropriate construction method and sequence, together with a fair construction contract (Bieniawski, 1993).

Some additional explanations are helpful when working with the above design principles, as follows:

- <u>Independence Principle.</u> Proper problem definition is the most important in design and Design Principle #1 is directed to that purpose. Since the designer can arbitrarily define the functional requirements to meet a certain perceived need, an acceptable set of functional requirements is not necessarily unique. Moreover, corresponding to a set of functional requirements there can be many design solutions. This then provides ample scope for creativity and produces design winners and losers.

- <u>Minimum Uncertainty Principle.</u> This principle is proposed for rock engineering because rock masses cannot be fully characterized for engineering design in a manner that steel or concrete can. Rock masses are complex geologic structures governed by large scale geologic discontinuities and are difficult to test as a full scale prototype. Accordingly, extrapolation of data from small-scale laboratory samples to large-scale in situ features will always involve a degree of uncertainty. In fact, asking: "When are the needs for site characterization met?" and "How much information is enough?" have no consensus of answers in the rock engineering community.

- <u>Simplicity Principle.</u> Once Principles #1 and #2 are satisfied, it should be remembered that the output of the design process is in the form of drawings, specifications, and other relevant knowledge required to create the physical entity. Thus, the best design solution should be as simple as possible, so the design output can be conveyed with minimal effort. This is the essence of this Principle #3. Its motto is: "the simpler, the better."

- State-of-the-Art Principle. In spite of extensive research performed in the field of rock engineering since the First International Congress on Rock Mechanics held in Lisbon in 1966, innovation in rock engineering design has not proceeded as rapidly as in other engineering fields. This is mainly due to the designers reacting cautiously to change and being reluctant to introduce new products and approaches until they have been proven elsewhere, or simply not being aware of the latest developments. It is submitted that great strides have been made in rock mechanics research and in the tunneling industry worldwide which must find a way to rock engineering practice by innovative designs featuring state-of-the-art technology.

- Optimization Principle. A good design is one achieved by the use of optimization techniques resulting in an innovative design product. In essence, optimal design is viewed as applying the optimization techniques to engineering design (Siddall, 1982). Optimization is crucial in design because most engineering problems do not have a unique solution. Reconsideration of the solution may be necessary in an attempt to approach a feasible compromise between the often conflicting requirements and resources.

- Constructibility Principle. In rock engineering design, we can envision three domains: functional domain, physical domain and construction domain. Each of these domains is defined by its own multiparameters or multivariables. During the design stage, the functional requirements (FRs) must be satisfied by choosing a proper set of design components (DCs), whereas during the construction phase, the DCs must be satisfied by selecting an optimum set of construction procedures (CPs). Effective design for constructibility requires the optimization of the relationships among the functional, design and construction domains; so, there is a relationship between the functional requirements and the construction procedures.

The Constructibility Principle is a unique innovation in rock engineering. It is already recognized in mechanical engineering that the concept of "design for manufacturability" has led to highly streamlined production processes. The construction principle advocated here deserves a special place in the field of rock engineering, particularly that in so many tunneling and mining projects, design and construction are performed by different companies, often lacking integration of the overall effort. Because a selected construction procedure (CP) cannot be fully tested in advance, events such as a tunnel boring machine being immobilized in a drift are not unusual. By establishing the functional path FR-DC-CP, the chosen design component can be effectively constructed.

3.2 Design Methodology for Rock Engineering

It is believed that a design methodology for rock engineering can benefit from a structured process featuring a number of design stages but one that would not constitute a 'straight jacket'; rather it should be a flexible framework adaptable to the problem at hand. One should thus visualize design methodology as a checklist (not unlike the one used by pilots before taking off) or a road-map guiding the designer to fulfillment of the problem objectives by evolving the best design option. It is thus a sequence of steps or activities within which a design can unfold logically but it must incorporate specific design principles. They serve to evaluate design alternatives and to select the optimum design solution fulfilling the design and performance objectives. The design methodology serves as a useful reference of where we are, where we ought to be, and what the next step should be within the overall work plan. Accordingly, an effective design methodology for geologic media can include elements of a systematic design process and it can also incorporate the use of engineering heuristics ("rules of thumb").

Figure 2 presents the details of the advocated Systems Design Methodology for rock engineering. This methodology should be seen as a systematic decision-making process aimed to satisfy the perceived needs, identified by independent functional requirements. Effective design solutions are represented by design components which meet the corresponding functional requirements and facilitate selection of efficient construction procedures. The design principles ensure that a good design is produced and offer a basis for developing, comparing, evaluating, and optimizing designs. Team-work and integration are emphasized in all these activities.

An important consideration for using the Systems Design Methodology for HLW management at Yucca Mountain is that the design process emphasizes the need to start with the early identification of clear design objectives and performance criteria, proceeding through site characterization (which must be justified by or traced to these objectives), going through conceptual design options, which use the determined site characterization data, evaluating design alternatives, and finally moving on to construction, in a most efficient manner, not unlike the concept of "design for manufacture" in other engineering fields. Note that a number of design options should always be considered, documented and recommended for final selection. Unlike what is found in the Design Package 2C, alternative design options should be discussed and the selected design fully motivated.

Finally, an overall systems design methodology for the whole process of high-level waste disposal in the USA is yet to be undertaken along the lines discussed above. In principle, just as an engineering *structure* (e.g. a repository) requires a design methodology, so does a *process* - such as HLW disposal - needs a comprehensive and integrating systems approach, a design methodology which serves as a management plan to ensure that all the design and performance objectives are fully achieved.

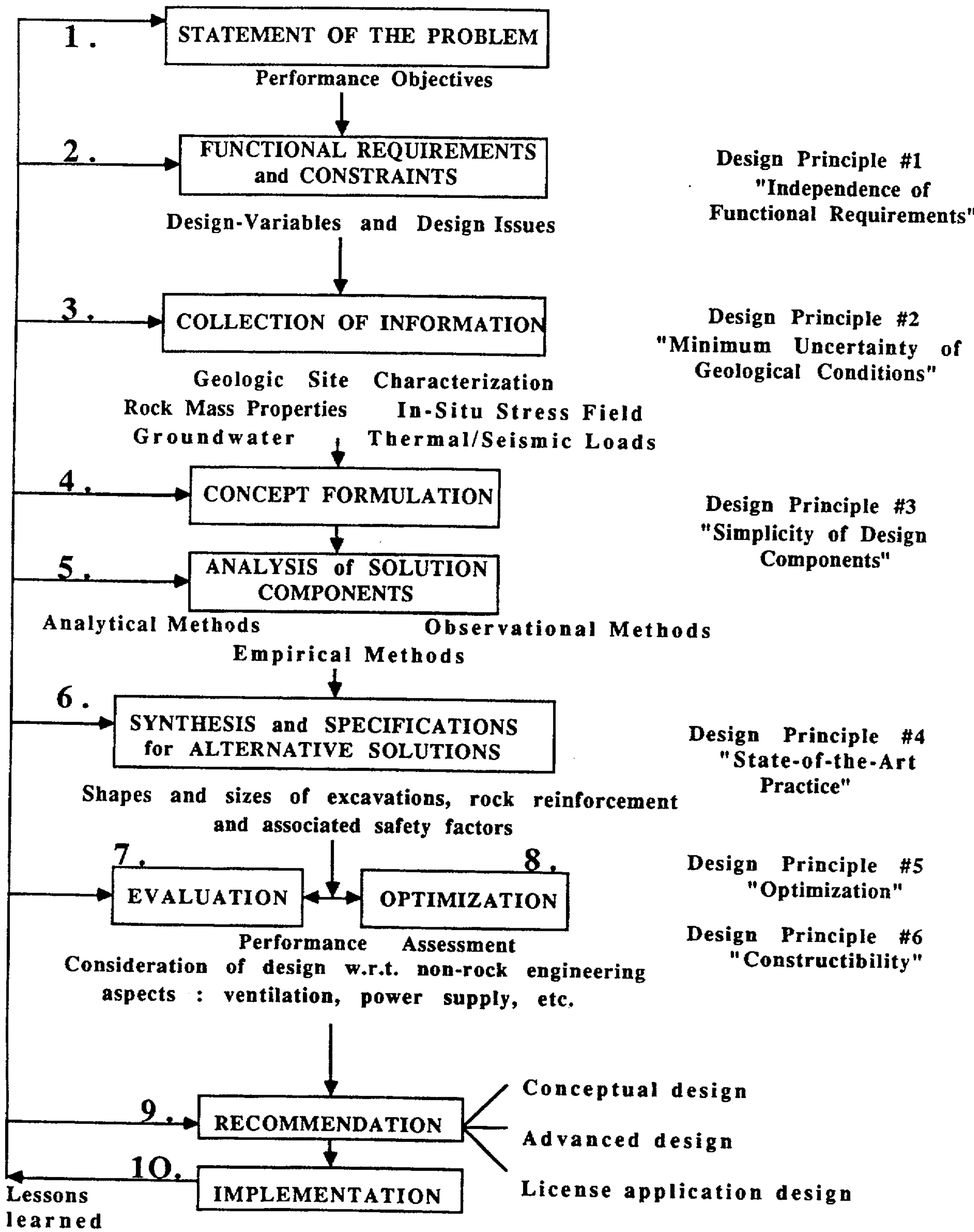

Figure 2 Systems Design Methodology for rock engineering, including the use of design principles (Bieniawski, 1993).

4 CASE STUDY: THE STARTER TUNNEL AT YUCCA MOUNTAIN

The design and construction of the starter chamber as well as the north rump at Yucca Mountain offers a good insight into the current design and construction problems and provides an example for the potential of using the Systems Design Methodology for rock engineering, discussed above.

Three aspects are discussed in this section: (i) DOE's drift design methodology, (ii) Design Package 2C with respect to integration of rock mechanics issues in ESF design, and (iii) identification of rock mechanics issues most important for repository design and performance assessment.

4.1 DOE Drift Design Methodology: An Out-of-Date Approach

The DOE A/E uses the drift design methodology (Hardy and Bauer, 1991) developed in 1989 for Sandia National Laboratories, when drill-and blast excavation and other repository design concepts were applicable. At the time, it was a positive and appropriate development in the absence of any design methodology by the DOE and it served a useful purpose of emphasizing a need for a design process, building up on previous research, and dealing with rock mass strength criteria.

But much has changed since then, following the pioneering work of Suh (1990) and its extension to rock engineering (Bieniawski, 1992). These two publications, together with notable developments in design methodologies in Germany and Japan, brought about two major contributions: (i) the need for **design principles in rock engineering** to assess alternative designs, and (ii) the need for a concept of **design for constructibility** in rock tunneling similar to the all important "design for manufacture" or "concurrent engineering" used in manufacturing industries.

It is difficult to understand why for some five years the DOE and the A/E have ignored all these developments and today still rely on the old document (Hardy and Bauer, 1991), even ignoring its own revision (Lin et al., 1993). This certainly does not invite a feeling of confidence in the DOE, by the design and tunneling community, with respect to the state-of-the-art awareness, let alone a display of innovation.

My involvement in reviewing the original document (Hardy and Bauer, 1991), which I considered a worthwhile work in 1989, is not in conflict with my current attitude of its unacceptability in 1994, as well as the shortcomings of the follow up report (Lin et al., 1993). The reasons for the shortcomings of the drift design methodology are many and I sympathize with the authors who had to work under pressure. But the fact that recent developments in design methodologies were ignored, that rock mass classifications and analyses were used wrongly, that erroneous information was recommended for incorporation into the RIB, and the acceptance by the A/E of the 1989 study, without justification , for repository design - must be pointed out.

Space limitation does not permit to deal here with all the shortcomings of the DOE/AE methodology but it simply is just not workable without any specific design principles. Moreover, it does not pay any attention to the changed circumstances at Yucca Mountain involving TBM excavation. This is demonstrated in the next section, showing that Design Package 2C has many inconsistencies because it is lacking an appropriate design methodology.

4.2 Design Package 2C and Integration of Rock Mechanics Data in ESF Design

In any rock engineering project, both in mining and in tunneling, integration of input data and design activities involves three basic loops: Loop 1 - from the project objectives to geotechnical data collection and interpretation; Loop 2 - from geotechnical data to design specifications; and Loop 3 - from design specifications to construction procedures.

However, in Loop 1, three further loops are involved: subloop (i) from the project objectives to geologic characterization; subloop (ii) from geological data to rock mass quality quantification (engineering rock mass classifications); and subloop (iii) from the project objectives to rock mechanics testing. All these loops require effective integration of activities, involving a complexity of interactions depicted in Figure 3. In addition, these activities must be performed on an efficient time schedule so that the whole process can provide the designers with timely input data, before the design has to be finalized. Thus, design validation should not be only an after-the-fact "verification" of what could be a vastly overdesigned excavation because the input data were not available on time.

In turn, the designers have to bear in mind the practical tasks of tunnel constructors who have to execute the proposed designs. Moreover, in the context of nuclear waste disposal, quality assurance will dictate that high standards, good record-keeping, work traceability and accountability are maintained throughout. In fact, as listed by Cikanek and Petrie (1992), the following documentation should be collected, controlled, stored, maintained as the QA Records in accordance with the YMP procedure:

1. All design inputs and relevant correspondence.
2. Analyses.
3. Drawings, including as-built versions.
4. Specifications.
5. Evidence of internal and external reviews.
6. Approved changes to design inputs, analyses, drawings and specifications.
7. Evidence of design verification.
8. Records confirming interface control.

I do not believe that the above is being done and would like to challenge the responsible DOE office to demonstrate where this information is kept, for public access and review, with particular reference to the design of (i) starter tunnel, and (ii) the north ramp. Also a question: when is a post-construction report prepared, what must it contain, and where may one inspect it?

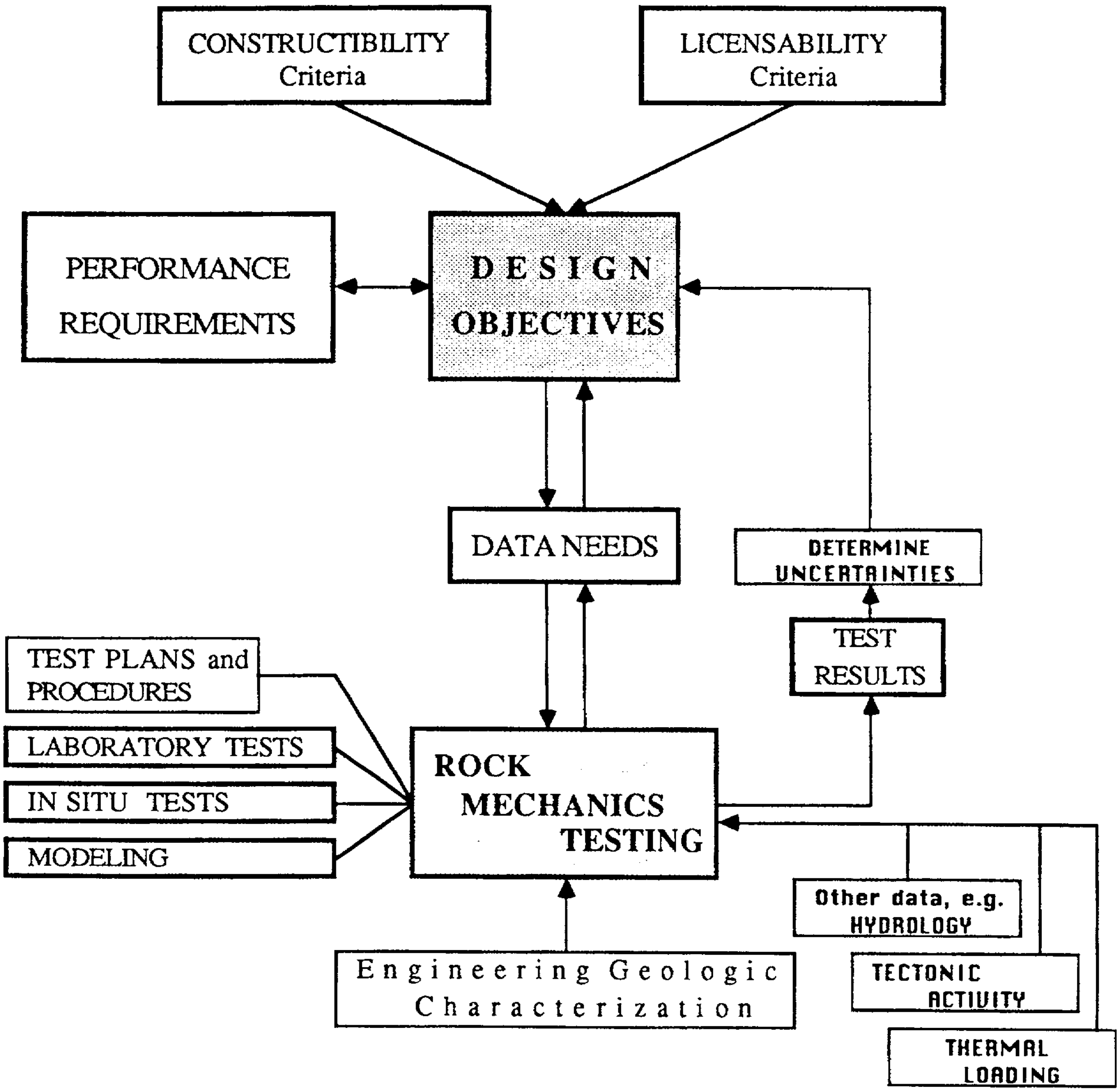

Figure 3 Integration of geotechnical activities with design objectives and performance requirements for repository design.

The reasons for these statements are that as a part of an independent research project, sponsored by the NSF, and also as an educational experience for my civil engineering students taking an environmental engineering class design project, I have researched this matter, visited the DOE library in Las Vegas, visited the Yucca Mountain site, and held discussions with many professionals during the HLW conference in Las Vegas in May. I was unable to trace the above information; it was certainly not in the DOE library (called "Research and Study Center"), where it should be.

However, the library had the Design Package 1A (starter tunnel) available for review. It is a highly disjointed document and it does not answer the question as to why the rock support design is quite different from its as-built version.

I was also able to acquire Design Package 2C (USDOE, 1994), which is a much better document, dealing with the design of the north ramp. Although this document was a "draft material" for 90% review, in DOE's own words: *"at this point, project engineers have completed their design but they leave a leeway to make further changes"*. I am not sure what changes were decided upon in June when this package was finalized.

While the limited space here only permits a brief review of some highlights of this document, I believe that it provides the ample proof for the lack of integration of geologic - rock mechanics - design - construction issues. It also contains errors, in terms of rock mass classifications and design methodology, such that my students and I had to wonder what was the meaning of "Quality Assurance" Q-1 and Q-5, referenced as document no. - note this: BAB000000-01717-00005!!! My concerns are illustrated below.

• <u>Rock mass classifications.</u> The document invented its own rock mass classes designations for the RMR and the Q systems, adding a new class and canceling another, and called this a "standard procedure". To add to confusion, it also reversed the order of classes, making Class 1 the worst and Class 5 the best, instead as is customary for Class 1 (as in "first-class" expression) being the best. Furthermore, it employed a drill-and-blast analysis of tunnel stability and called it "conservative" rather than undertaking a quantitative analysis for TBM construction, guidelines for which are available in the literature (*SME Mining Engineering Handbook,* 1992, chapter 10.5).

The RMR values were used to estimate the stand up time for *horseshoe shape* tunnels, while the Q ratings were used to estimate rock support categories for *drill-and-blast* tunnels (justified as "conservative" for TBM excavation). For this purpose, for TSw2 unit, the average values were Q = 1.53 and RMR = 55. This gave a stand up time for the 7.62 m diameter tunnel as 400 hours (16.6 days) an support as untensioned grouted rock bolts, 3.14 m long with 1 m spacing and 25 to 75 mm of shotcrete (no wire mesh). However, both Q and RMR values are probably wrong because for Q calculations an error was made when selecting the Excavation Support Ratio ESR), which adjusts the effective span. For major tunnels, a value of ESR=1.0 is used but for underground nuclear power stations, ESR=0.8 is required which increases the effective span. The DOE A/E used SRF=1.0 without justification. Also, the RMR ratings were erroneously lowered by averaging the actual values with ones calculated from the RMR vs Q correlation, which has not been established as valid for tuff. Unfortunately, without consulting Hardy and Bauer (1991), the reader/reviewer/constructor using Package 2C would not know these aspects, nor which newly created rock class is actually the design (average) condition.

In fact, the design condition is category 4, not 3 (see Table 1) because the RMR "best rock" class was not used and a new class "extremely poor rock" was created. For that condition (category 4), the stand up time is 2000 hours (83 days) and not 16 days which barely meets the TBM "window" for support installation in 11 days.

• Design Analyses. Design Package 2C is highly significant because the A/E actually explains the design approach, methodology and provides discussion of the "key issues" concerning the design and construction of the ESF in general and the north rump in particular. Thus, the DOE A/E makes an important statement: *"the drift design methodology (Hardy and Bauer, 1991) is considered by the A/E to be technically appropriate for the ESF design... which will be incorporated into a geologic repository".* This methodology is listed in Package 2C as the only "criterion" for design analysis (Sandia's update report (Lin et al, 1993) is not used). Bearing in mind that this DOE design methodology is now 5-years out of date, as discussed in the previous section, one must worry not only about the integration of issues but also how innovative and up to date are the A/E's design efforts.

The A/E lists five specific ground support categories, no justification given, specifying steel ribs spacing to an accuracy of 1 millimeter (1/32 inch), typically at 1.22m centers! Interestingly enough, these categories reverse the category designations established by the drift design methodology of Hardy and Bauer (1991). Clearly this leads to considerable confusion and inconsistencies in Package 2C, as shown in Table 1.

Table 1

Rock Mass Quality Categories for ESF Design

Standard RMR Designations	DOE/M&O Design Package 2C Drift Methodology	DOE/M&O Design Package 2C A/E
1 Very Good	-	1 Best
2 Good	5 Good	2
3 Fair	4 Fair	3 Average
4 Poor	3 Poor	4
5 Very Poor	2 Very Poor	5 Worst
-	1 Extremely Poor	-

Based on these, and erroneous, rock mass classification procedures, support selection is made which is subsequently confirmed by two support charts and by numerical modeling. Both these charts are inappropriate and outdated. The support chart by Hoek (1980) was developed by that author as a "preliminary and crude estimate" primarily for deep mines and excavations where high in situ stresses and elastic behavior of rock apply. The Schmidt chart was developed for BWIP long ago as a preliminary attempt for a site specific rock classification and was not accepted there because it is based on just one parameter, the RQD, which is difficult to determine reliably without three-directional drilling and borehole core orientation devices.

Numerical design analyses were used in Package 2C *"to analyze ground support recommendations from empirical rock mass classifications"* using a Mohr-Coulomb plasticity model". They provided no less than 315 figures (!) of plots based on FLAC and UDEC software and on input rock properties data from NRG drill core testing. No reference was made to the rock mass strength analyses, used in the drift design methodology which involved the Hoek-Brown and the Yudhbir-Bieniawski failure criteria.

These criteria are listed in Table 2 with the corrected equations and input RMR values, based on Alber (1993).

For the numerical analyses, in situ stresses were specified based on the information found in the RIB while, for thermal stresses, it was concluded that they would not be significant for the north ramp up to 100 years of repository operations. For seismic loads, quasi-static and dynamic analysis were performed with 0.4g being used as the maximum horizontal and vertical components of acceleration, and (for dynamic analyses) a P-wave was introduced with a frequency of 10 Hz and duration of 0.25 seconds. However, no reference was made to the work of Krinitzsky (1993) who pointed out that such probabilistic seismic hazard analysis is defective because for earthquakes with M>5 on the Richter scale, which are most important, the geometry of earthquake rupture changes from an unrestricted propagation in all directions within a plane (good for generalizations) to a directional propagation in one or two directions.

In any case, based on these numerical analyses, it was concluded that "for the average rock mass quality, the selected ground support components performed well" (i.e. 3 m long grouted rock bolts on a 1.5 m square pattern, above the spring line, plus welded wire mesh, 150mm x 150mm, without shotctrete. One wonders whether 315 pages of graphs were needed to arrive at this conclusion!

Table 2

Rock Mass Strength Criteria for Yucca Mountain Tuff
based on Rock Mass Classification (after Alber, 1993)

Hoek - Brown Criterion	$\frac{\sigma_1}{\sigma_3} = \frac{\sigma_3}{\sigma_c} + \sqrt{m\frac{\sigma_3}{\sigma_c} + s}$ For tuff: $\sigma_c = 160$ MPa
Hoek - Brown Parameters	$m = m_i e^{\frac{RMR-100}{28}}$ $s = e^{\frac{RMR-100}{9}}$ For tuff: $m_i = 14$
Yudhbir - Bieniawski Criterion	$\frac{\sigma_1}{\sigma_c} = A + B(\frac{\sigma_3}{\sigma_c})^{0.75}$
Yudhbir - Bieniawski Parameters	$A = e^{(0.0765\,RMR-7.65)}$ For tuff: B = 3
Rock mass classifications for tuff	RMR = 69 (range 50 - 82) Q = 12 (range 1.04 - 64)
Parameter values for Hoek - Brown and Yudhbir - Bieniawski criteria	Hoek - Brown: m = 1.98 s = 0.03 Yudhbir- Bieniawski: A = 0.09 B = 3
Design Criterion	$\sigma_{1S} = 21 + 9\sigma_3^{0.75}$

In summary, Design Package 2C, from the point of view of activities integration: geology - rock mechanics - design - construction, leads to these observations:

• while an attempt is being made to integrate construction of the north ramp and site characterization (special facilities for mapping on the TBM), there is considerable fragmentation of activities between USGS, Bureau of Reclamation (geologic mapping), Sandia Labs, Agapito Associates (geotechnical mapping and rock mass classifications), M&O design construction and monitoring teams (Morrison Knudsen, CTS-TBM, Kiewit, Raytheon, and others). Will all these teams be able to interact effectively in an underground environment?

• integration of rock mechanics activities with either geological aspects or with the design aspects leaves much to be desired based on the evidence contained in Design Packages 1A (starter tunnel) and 2C (north ramp).

• good geotechnical data collection and representation is evident from Package 2C but actual rock mass classification procedures are suspect and interpretations are out of date.

• the "phased approach" by the A/E is reasonable for support design and installation but there is little evidence of any integrating methodology for "incorporating the ESF design into a geologic repository and for accommodating the Site Characterization Program and Performance Confirmation", as is claimed in Package 2C.

• the rock mechanics analyses involving empirical as well as numerical methods seem to have been performed independently of each other and only compiled in the same document for the purpose of Design Package 2C. In fact, 315 pages of numerical data were presented which did not change any empirical findings, some of which are erroneous. No explanations were given as to why the tunnel diameter is 25 ft and why no TBM effects were analyzed, except saying "this adds further conservatism". In this respect, it might be noted that recent work at Penn State has shown that the benefit of TBM excavation on rock mass quality can be quantified as follows:

$$RMR_{TBM} = 0.8\,RMR_{D\&B} + 20.$$

• most of all, an outdated drift design methodology is used without regard for the major developments in the last five years in rock engineering design principles, methodologies and rock mass strength criteria. By not involving any specific design principles, the DOE A/E methodology does not provide design alternatives which can be evaluated and optimized.

4.3 "Top Ten List" of Rock Mechanics Issues for Repository Design

1. Review of the specific design needs (to be provided by DOE/M&O) concerning new thermal loading strategies, tunnel dimensions and layout, retrievability period, repository capacity and design alternatives (e.g. multilevel repository).

2. Effects of different thermal loading options on rock mechanics data needs.

3. Consequences of in-drift emplacement instead of borehole emplacement.

4. Concepts for rock mass strength in design using the criteria of Hoek-Brown, Yudhbir-Bieniawski and Mohr-Coulomb and incorporating TBM effects; in situ confirmation tests.

5. In situ stresses, techniques to be used, assumptions to be made (e.g. is the intermediate principal stress neglected) and design values.

6. Seismic loading; current probabilistic seismic hazard analyses may be defective for the most important earthquakes, i.e. those with M>5 on the Richter scale (Krinitzsky, 1993).

7. Redesign (simplify the layout) of the core test area, including the possibility of a multilevel repository, reducing the number of underground offices, store rooms, shops and warehouses, and using full-face TBM excavation. Test alcoves should not be excavated by drilling and blasting.

8. Prioritized program of in situ tests in the ESF core test area, complete with specifications for the configurations of the test alcoves.

9. Machine excavation effects on rock mass classifications and geologic data collection.

10. Incorporation of all the above into the Systems Design Methodology using the six design principles for rock engineering, with emphasis on "design for constructibility".

5 CONCLUSIONS

Little integration is evident at the Yucca Mountain Project between geotechnical and design activities. Design Package 2C has been analyzed and found deficient in this respect. Ten rock mechanics issues for repository design have been identified and an integrating design methodology, featuring specific design principles for rock engineering, was described. However, to resolve the issues of rock mechanics - repository design integration, the main initiative must come from the design engineers whose duty it is to specify and utilize the required geotechnical data.

Acknowledgment

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A PROBABILISTIC APPROACH TO ROCK MECHANICAL PROPERTY CHARACTERIZATION FOR NUCLEAR WASTE REPOSITORY DESIGN

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ABSTRACT: A probabilistic approach is proposed for the characterization of host rock mechanical properties at the Yucca Mountain site. This approach helps define the probability distribution of rock properties by utilizing extreme value statistics and Monte Carlo simulation. We analyze mechanical property data of tuff obtained by the NNWSI Project (Price, 1983) to assess the utility of the methodology. The analysis indicates that laboratory measured strength and deformation data of Calico Hills and Bullfrog tuffs follow an extremal probability distribution (the third type asymptotic distribution of the smallest values). Monte Carlo simulation is carried out to estimate rock mass deformation moduli using a one-dimensional tuff model proposed by Zimmerman and Finley (1986). We suggest that the results of these analyses be incorporated into the repository design.

1 INTRODUCTION

The U.S. repository program is founded on the philosophy that the repository performance relies primarily on the natural barrier even though the use of engineered barriers is considered to augment the chance to meet the performance requirements. The requirements are specified in 10 CFR 60 (NRC 1987), 40 CFR 191 (EPA, 1986) and 10 CFR 960 (DOE, 1987) and they drive the repository design and site data collection activities. These activities are, therefore, planned and executed to support the PA computations. The foregoing federal regulations, in particular the EPA's 40 CFR 191, require that assessments of long-term repository performance incorporate uncertainties as a probability distribution.

Rock mechanics analyses supporting the PA computations employ deterministic methods predominantly, and the site data collection activities are designed to support the deterministic methods. Recently, attempts have been made to analyze the long-term stability of emplacement rooms using a probabilistic approach (Kemeny and Cook, 1990). Yet, most numerical and analytical models currently used for design activities are basically deterministic, and employ single values to represent rock mass properties. They do not incorporate statistical variabilities of rock properties into their computational schemes. Therefore, they do not predict the range of situations to which the repository is exposed since they can only consider specific conditions and not the expected range of variable conditions.

We contend that probabilistic approaches can be formulated and utilized for some key design activities. As a first step toward this goal we propose an approach which is based on extreme value statistics and Monte Carlo simulation for host rock characterization. We analyze a set of published site data from the NNWSI program primarily to describe our proposed approach.

1.1 Background

The properties of host rock including mechanical, thermal and hydrologic properties along with in situ stresses at the repository horizon are key engineering parameters affecting the performance of the repository during pre- and post-closure periods. The performance of emplacement holes and rooms, the access ramps and shafts, and other structures is directly affected by these host rock properties. These properties must be characterized comprehensively in order to arrive at a defensible design of repository structures. Properties of the host rock which is a natural material as opposed to an engineered material, are random variables instead of narrowly definable constants. Their probability distributions have to be known in order to assess the failure or survival probability of repository structures which will be constructed within the host rock in the given geologic setting, and they have to be incorporated into design.

It should be noted that the probability distribution function can not be obtained by simply fitting test data to an arbitrary form. It has been shown that the same data can be fitted almost equally well to several models (Baba, Nakagawa, and Naruoka, 1980). But, the test data must fit the selected model well and the appropriate model must be selected based on a theoretical mechanism. One such example is the Weibull distribution (Weibull, 1939) which was developed based on the weakest-link theory. The use of the Weibull distribution has been widely accepted for the prediction of material failure including rock. Another example is the Gumbel distribution which is based on extreme value statistics (Gumbel, 1958). This theory has been applied to rock (Yegulalp and Mahtab, 1983; Yegulalp and Kim, 1992). These studies reveal that mechanical properties of intact rocks obtained from laboratory tests follow an extremal probability distribution, the third type asymptotic distribution of the smallest values. Recently, attempts have been made to assess the variability of a candidate repository host rock to arrive at a more defensible design methodology. Site data obtained at the Basalt Waste Isolation Project were analyzed using extreme value statistics (Kim and Gao, 1994). They analyzed the deformation modulus obtained in laboratory and field (borehole jacking) tests and found that the data follow the third type asymptotic distribution of the smallest values. Then, they employed Monte Carlo simulation to estimate rock mass properties. In their simulation, they used the distribution functions of the component parameters including laboratory and field measured properties as input to the mathematical relationships obtained previously by other investigators. The simulation results yielded the distribution functions which could be used to estimate variabilities of rock mass mechanical properties.

Rock engineers have been using deterministic methods to guide their activities in site characterization and design of conventional underground structures. Deterministic methods do not cover all possible situations over the design life of the rock engineering structures. In the case of nuclear waste repository which requires long term performance, a risk based performance assessment methodology must be formulated and used to arrive at a defensible design. We present a methodology which is a step toward this goal. In this paper, we discuss a probabilistic method and the preliminary results of analyses which used Calico Hills tuff and Bullfrog tuff data previously obtained by the NNWSI project (Price, 1983) at the Nevada Test site.

2 PROBABILITY DISTRIBUTION OF ROCK MECHANICAL PROPERTY

The determination of rock mass property is considered an expensive proposition because rock mass properties are a complicated function of intact rock and joint properties as well as other geological factors. Even in intact rocks which appear very homogeneous, pronounced variabilities are commonly observed when they are tested. Expert rock engineers frequently forego any attempt to assess variabilities of rock mass properties and resort to subjective expert judgments. These variabilities are considered inherent characteristics, and they are attributed partly to the geologic processes which the rock has experienced, and partly to measurement uncertainties. Properly executed tests suppress the measurement uncertainties and allow the determination of the characteristic variability.

The methodology we propose entails 1) characterizing the intact rock properties in the laboratory and field using the probabilistic method, and 2) estimating rock mass properties using Monte Carlo simulation. The latter requires a mathematical equation describing the relationship between the component rock parameters and rock mass properties of concern. We describe the proposed procedure in the following sections.

2.1 Probability Distribution of Laboratory Test Data

We have reviewed pertinent data in open literature to select a suitable data set of high quality for our analyses. We have found that the data obtained by the NNWSI project are most suitable for our purpose, which are from four stratigraphic units within Yucca Mountain. An initial examination reveals that the number of samples varies widely from one to as many as 100. The data include Young's modulus (E_o) and unconfined compressive strength (C_o) (Price, 1983). Each set of data is analyzed by using extreme value statistics and is found to follow the third type asymptotic distribution of the smallest values. The cumulative distribution function is expressed (Gumbel, 1958):

$$F(x) = 1 - e^{-\left(\frac{x-e}{v-e}\right)^k} \tag{1}$$

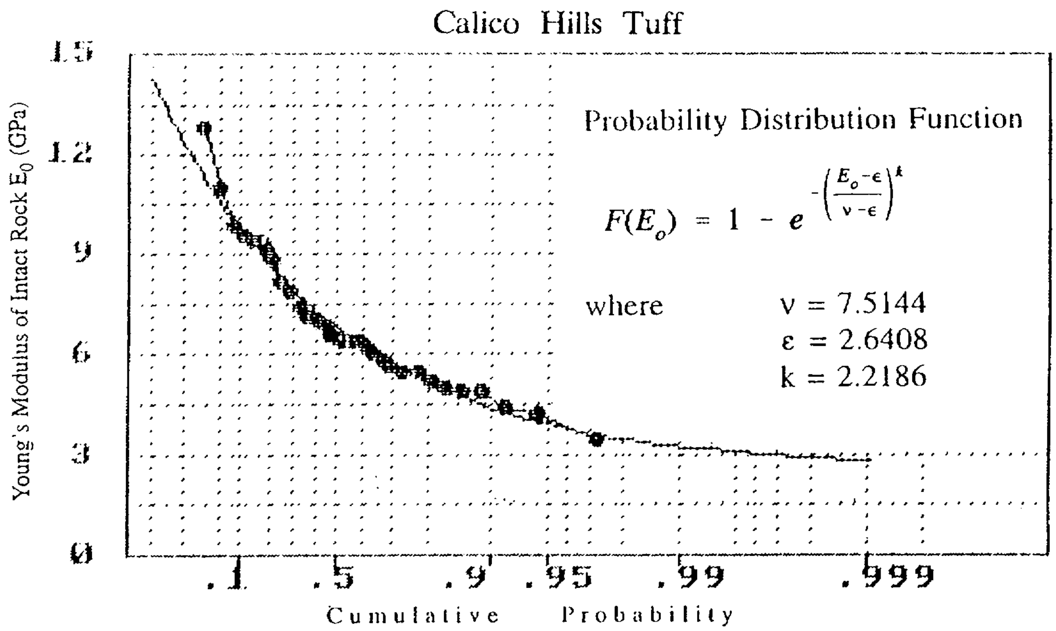

Figure 1. Observed and theoretical extremal distributions of Young's modulus (E_o) of Calico Hills Tuff obtained from laboratory core tests

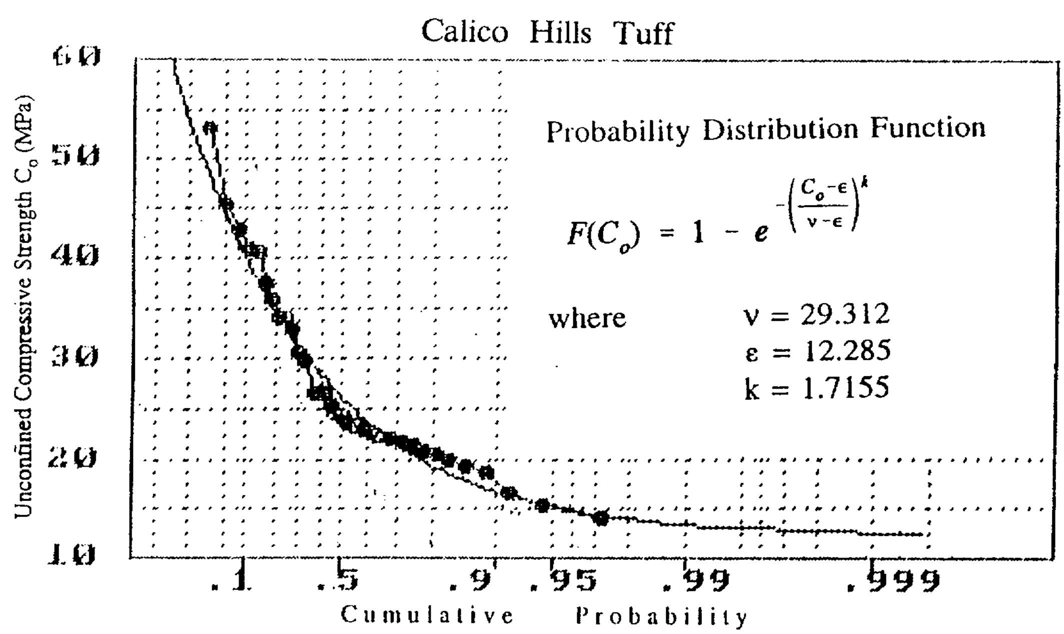

Figure 2. Observed and theoretical extremal distributions of unconfined compressive strength (C_o) of Calico Hills Tuff obtained from laboratory core tests

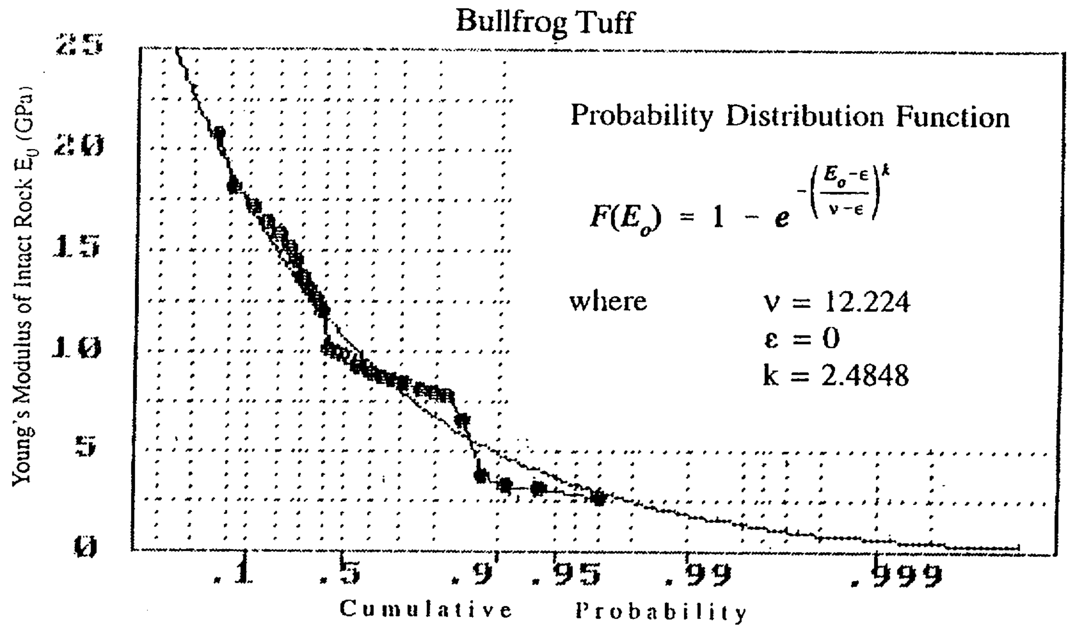

Figure 3. Observed and theoretical extremal distributions of Young's modulus (E_o) of Bullfrog Tuff obtained from laboratory core tests

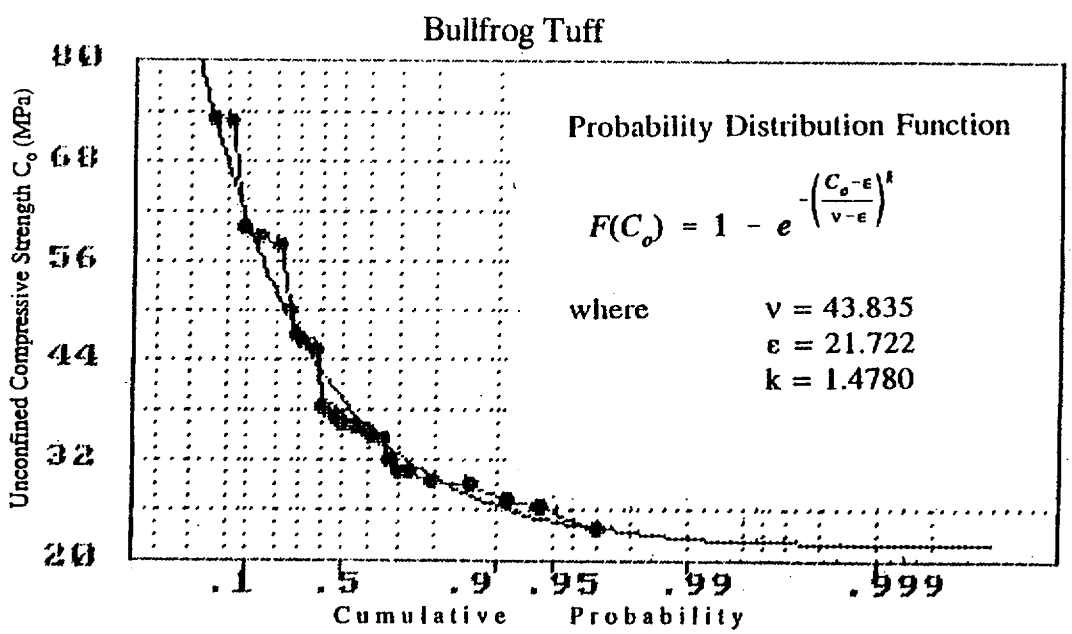

Figure 4. Observed and theoretical extremal distributions of unconfined compressive strength (C_o) of Bullfrog Tuff obtained from laboratory core tests

where k is the shape parameter of the distribution,

ε is the lower bound on X, and

v is the scale parameter of the distribution (also called the characteristic smallest value).

Data are regressed to extremal distributions which are plotted by using a extreme value statistics computer package (Yegulalp, 1988). Figures 1, 2, 3 and 4 display graphically the extremal distributions of the Young's modulus (E_0) and unconfined compressive strength (C_0) of Calico Hills tuff and Bullfrog tuff. The χ^2 goodness-of-fit test at 95% confidence level indicates that all of the data sets can be represented by this type distribution. This suggests that the third type distribution represents the inherent variability of mechanical properties of rocks.

2.2 Application of Monte Carlo Simulation

Having obtained the probability distributions of the mechanical properties of intact rock, we attempt to evaluate the probability distribution of rock mass properties using Monte Carlo simulation. The method is traced back to the end of last century, but the systematic use of the method started during World War II, when this approach was applied to problems related to the development of nuclear weapons (Metropolis and Ulam, 1949; Law and Kelton, 1991). This method has been widely used in science and in engineering fields, public services, and military works. It is generally defined as representing the solution of a problem by using random numbers to construct the sample space from which statistical estimates of interests can be obtained.

We apply Monte Carlo simulation to the one dimensional mathematical model describing rock mass mechanical properties obtained previously from the NNWSI Project (Zimmerman and Finley, 1986):

$$E_{dt} = \left[\frac{1 \quad + \quad n \, V_m}{(n\sigma_n E_0 V_m / (\sigma_n + \sigma_h)^2) + 1} \right] E_0 \tag{2}$$

where E_{dt} is the tangent modulus of deformation of rock mass (MPa),
E_0 is the Young's modulus of intact rock (MPa),
n is the number of joints per meter,
V_m is the unstressed joint aperture (m),
σ_h is the one-half closure stress for a joint (MPa), and
σ_n is the stress normal to the joint (MPa).

In this equation we treat the Young's modulus of intact rock (E_0) as a random variable, which follows the third type asymptotic distribution of the smallest values as given in Equation (1). We use an average value for the one-half closure stress because its probability distribution is not known. Regarding the joint spacing (J_s) (the reciprocal of J_s

is the number of joints per meter) and the joint aperture (V_m) we use the exponential distribution function proposed by Einstein and Baecher (1983). Regarding the stress normal to the joint (σ_n), we assume that the stress normal to the joint (σ_n) varies uniformly between the maximum and the minimum horizontal stresses σ_{Hmax} and σ_{Hmin}.

We carry out numerical simulations for four different cases of joint spacings. Figure 5 displays the histogram constructed using simulation results. These results are, then, analyzed by using extreme value statistics. Figure 6 presents the theoretical and simulated deformation modulus of rock mass. The fit is nearly perfect, similar to those observed in the analysis of laboratory test results. This suggests that the simulation results reflect the distributive characters of rock mass properties.

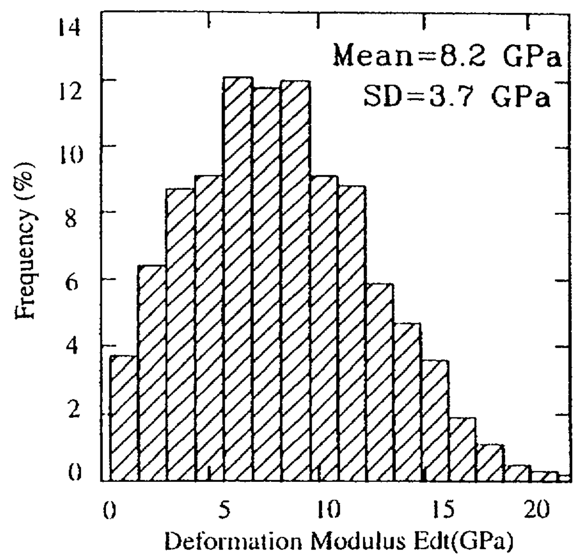

Figure 5. Histogram of simulated tangent deformation modulus of rock mass

3 PERFORMANCE ASSESSMENT FOR NUCLEAR WASTE REPOSITORY

It has been common practice that engineers rely on subjective expert judgments for the design of rock structures, which is guided by deterministic analytical or numerical methods. These methods do not incorporate explicitly the statistical nature of rock mass properties. The question regarding how to characterize variabilities of rock mass properties and their effects on the design of repository subsystems has emerged as a new challenge to rock engineers. Engineers have been attempting to cope with uncertainties by using empirically chosen safety factors. The computational methods currently used for repository design do not consider the variability of rock properties and assume the mean values of rock properties measured or estimated to be representative of the rock mass characteristics. This method can cause grossly misleading results depending on the distributive character of rock property variations.

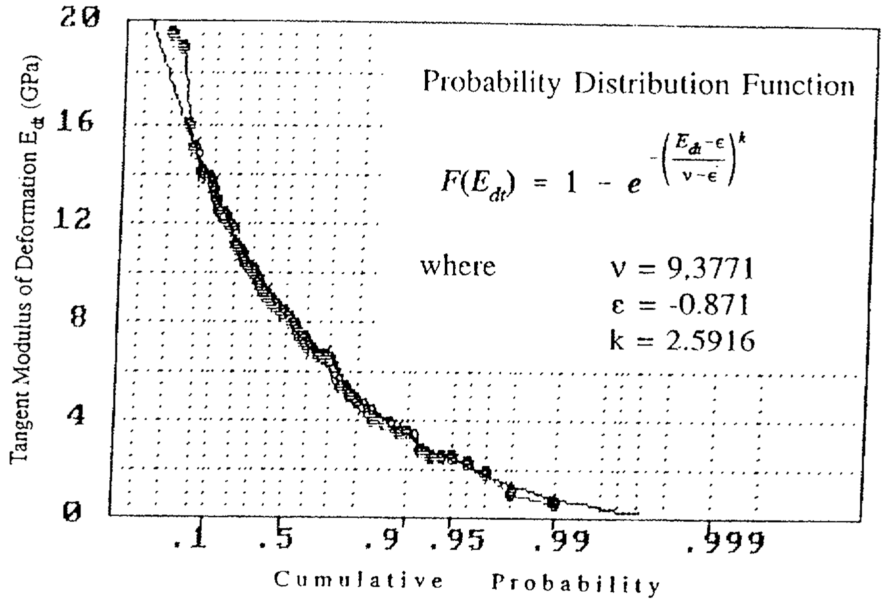

Figure 6. Simulated and theoretical extremal distribution of tangent deformation modulus

As a step toward improving the current situation, we propose a probabilistic approach for the characterization of the mechanical properties of rock mass. This approach accounts for the uncertainties due to the variability of the rock material properties and the patterns of the discontinuities in rock mass. In this approach all parameters describing the rock mass properties are considered random variables. Monte Carlo simulation is applied to obtain the probability distributions of mechanical properties of rock mass. Then, the simulation results are analyzed by using extreme value statistics. As demonstrated above mechanical properties of rocks follow the third type asymptotic distribution of the smallest values (Equation 1). A close examination of the third type distribution of the smallest values (Equation 1) reveals that it is the same as the well-known Weibull distribution if the term (ν - ε) is expressed as a single parameter. It is noted that the extremal distribution is derived under the most general conditions, regardless of its intended specific field of application.

4 DISCUSSIONS

A question regarding how many samples are needed to obtain the probability distribution has been addressed by Gumbel (1958). He asserts that at least twenty-five data points have to be sampled in order to minimize the sampling and estimation errors. Twenty-five tests can be conducted in laboratory and borehole scale tests, but not in large scale in situ tests. Monte Carlo simulation, as demonstrated above, can generate thousands of "samples" which can be used to arrive at an estimated probability distribution. It should be noted the analytical relationship representing the rock mass is requisite in this approach. Therefore, we suggest that large scale in situ tests be designed and executed to generate information to describe the rock mass properties in analytical forms. The characterization of component material properties should also be carried out to generate the distribution functions. Then, via Monte Carlo simulation we can estimate the distribution functions of rock mass properties which help assess performances of rock structures in repository conditions.

The parameter uncertainties estimated using this approach can be incorporated into the design and performance assessment of repository subsystems. As shown in Figure 7, the overlap between the strength and the stress distributions indicates the zone that contributes to the probability of failure P_f. The reliability (L) is defined as $(1-P_f)$, which has been developed to give the following results. The reliability (L) of a structural component is the probability of the strength being greater than the stress (Haugen, 1968):

$$L = \int_{-\infty}^{+\infty} f(S) \left[\int_{-\infty}^{S} f(R)\, dR \right] dS \tag{5}$$

where R and f(R) are rock mass strengths and their probability density function, and S and f(S) are rock stresses and their probability density function respectively.

When those two forms of functions, f(R) and f(S), are given, we can calculate the reliability (L). This concept, though well developed theoretically, has not been followed in the design of rock structures because of the difficulty of obtaining the density functions f(S) and f(R). We have presented in this paper that the probability distribution functions of rock material properties can be obtained from laboratory and field tests, and they can be used to estimate f(R) of rock mass by applying Monte Carlo simulation.

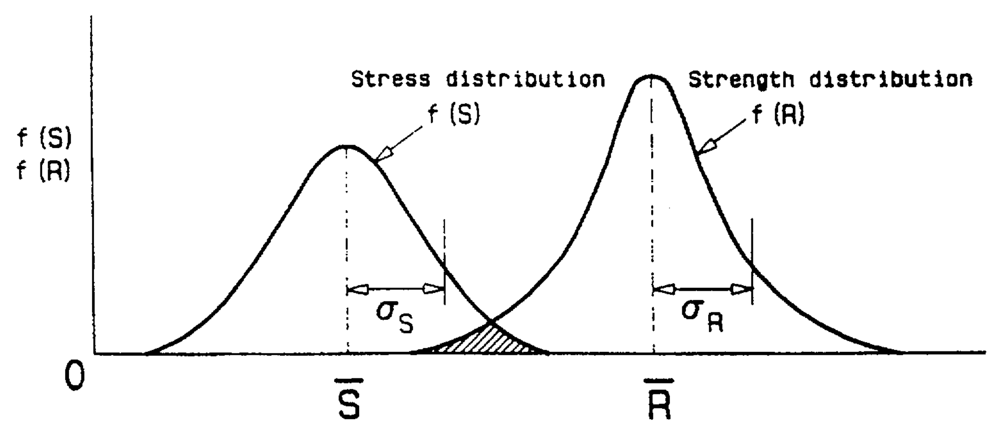

Figure 7. Stress (S) and strength (R) probability distributions (after Haugen, 1968), The overlap between the strength and stress distributions indicates the zone that contributes to the probability of failure.

5 SUMMARY

The difficulty of assessing the distributive characters of rock mass properties is not unique in the rock engineering field. Many naturally occurring phenomena show pronounced variabilities which can not be easily assessed. For instance, hydrologic properties vary typically a few orders of magnitude and their temporal and spatial variations are as difficult to assess as rock mass properties. Monte Carlo simulation has been successfully employed in post closure performance assessment. This approach has been widely accepted in other fields when clear analytical solutions are not available.

We propose that Monte Carlo simulation be used to estimate the distribution functions of rock mass properties. It provides general distributive characters of rock mass properties when used with appropriate empirical or analytical relationships. Their probability distributions can be found by sampling, by field mapping, or by Monte Carlo simulation. Results of these analyses, i.e. the distributive functions, can be and should be incorporated into the design methods. Then, the probability of the structural failures can be more quantitatively and rationally assessed than currently used methods which rely on the safety factor concept. The simple method of Monte Carlo simulation as used with readily available advanced computational schemes can produce more defensible designs and help assess the uncertainties quantitatively. We conclude this paper by highlighting the salient points in the findings of our study below.

(1) Laboratory mechanical properties of tuff specimens obtained at the NNWSI project from laboratory core tests have been found to follow the third type asymptotic distribution of the smallest values. This reveals the nature of the inherent variability of the mechanical properties of tuff specimens. The third type asymptotic distribution of the smallest values is proposed to be a general statistical representation of mechanical properties of rocks.

(2) The Monte Carlo simulation is proposed as a method to estimate the probability distribution of rock mass properties. As demonstrated above, the probability distribution of the deformation modulus of rock mass obtained from Monte Carlo simulation is also represented by the third type asymptotic distribution of the smallest values. This consistency indicates that the third type distribution reflect the inherent variation of the rock mass properties. However, the results of simulation depends entirely upon the functional relationships describing the rock mass properties, which have to be rigorously validated through the field tests. Once the relationships are validated, then, the results of simulation can be considered as a reasonable representation of the probability distribution of rock mass property.

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A NEW PEAK SHEAR STRENGTH CRITERION FOR ROCK JOINTS WHICH INCLUDES SPECTRAL PARAMETERS AS ROUGHNESS MEASURES

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ABSTRACT: Most of the natural rock joint surface profiles do not belong to the self similar fractal category. In general, roughness profiles of rock joints consist of non-stationary and stationary components. At the simplest level, only one parameter is sufficient to quantify non-stationary joint roughness. The average inclination angle I, along with the direction considered for the joint surface, is suggested to capture the non-stationary roughness. It is shown that even though the fractal dimension D is a useful parameter, it alone is insufficient to quantify the stationary roughness of non-self similar profiles.

A new strength criterion which takes the following general form is suggested for modelling the anisotropic peak shear strength of rock joints at low normal effective stresses:

$$\tau = \sigma \tan\left(\phi + a(SRP)^c \left[\log_{10}\left(\frac{\sigma_J}{\sigma} \right) \right]^d + I \right)$$

where σ, τ, σ_J, ϕ and SRP denote, respectively, the effective normal stress on the joint, peak shear strength, joint compressive strength, basic friction angle, and the stationary roughness parameter. The parameter combination $a K_s^b D^c$ is suggested to represent the term $a(SRP)^c$. The parameter K_s is a spectral coefficient. Joint roughness data should be used to estimate the roughness parameters I, K_s and D in different directions on the joint surface. Parameter D reflects the rate of change in length in response to a change in the scale of measurement r. Because K_s is a scale-dependent parameter, it can be used to model the scale effect. The coefficients a, b, c and d in the strength criterion should be determined by performing regression analysis on experimental shear strength data. In practice, to allow for modelling uncertainties, the new equation should be used along with a factor of safety of about 1.5.

1 INTRODUCTION

To predict the mechanical and hydraulic properties of rock masses, it is necessary to characterize (a) the joint geometry network, (b) the geomechanical and hydraulic properties of the joints, and (c) the geomechanical and hydraulic properties of intact rock. Due to the complex nature of natural joint patterns in rock, and the uncertainty involved in estimating geomechanical properties of intact rock and joints, the prediction of mass properties of rock at a large scale using the available stress analysis techniques, and a limited field, and laboratory information presents a great challenge for the geo-engineering profession.

Strength and deformation behavior of joints and flow properties of joints depend very much on the surface roughness of joints. These effects arise from the fact that the surfaces composing a joint are rough and mismatched at some scale. The shape, size, number, and strength of contacts between the surfaces control the mechanical properties. The separation between the surfaces or the "aperture" determines the hydraulic properties. A few methods have been suggested to characterize surface roughness of natural rock joints. The work done so far has been limited to characterization of surface roughness along linear profiles or in one dimension (1D). These investigations have led to controversial findings (McWilliams et al., 1993; Huang et al., 1992). In addition, since joint planes are two-dimensional, quantification of surface roughness on two dimensional planes is required. Usually, roughness on natural rock joint planes is anisotropic. However, anisotropic roughness quantification is not addressed in the literature. These clearly show our limited understanding on quantification of roughness of natural rock joints. A part of this paper presents findings of a research program initiated to characterize anisotropic roughness of natural rock joints.

The shear strength of rock joints should be studied under two major categories - they are the filled joints and the unfilled joints. In this paper, the investigation is restricted to the shear strength behavior of unfilled joints. The shear behavior of unfilled joints depends on the following factors: (a) rock type (b) level of normal effective stress on the plane of sliding, (c) degree of roughness, (d) size of joint (scale effect), (e) degree of weathering, (f) presence of moisture, and (g) water pressure. Only the effect of factors (b) and (c) on peak shear strength is focussed in this paper. The effect of roughness on the shear strength is more pronounced in the low normal effective stress range (0-0.4 times unconfined compressive strength). Therefore, in this paper, investigations are limited to the low normal effective stress range.

The main contributors for the development of peak shear strength criteria in the low normal effective stress range are Ladanyi and Archambault (1969) and Barton (1973). Movements along rock discontinuities in foundations, dams, tunnels and slopes can occur in various directions, depending on the external forces (such as external loads, water pressures, earthquake forces, etc.) acting on the structure and the kinematic constraints. Therefore, it is important to know the strength of rock discontinuities in different directions. The joint peak shear strength shows anisotropic properties due to roughness variation with the shearing direction in direct shear tests (Huang and Doong, 1990; Jing et al., 1992). Even along one particular direction, the shear strength of a natural joint can be different between the forward and the backward directions. To capture the above observations, either the existing shear strength criteria should be improved or a new peak shear strength criterion should be developed.

This paper presents findings of research initiated to develop an anisotropic peak shear strength criterion for rock joints. The emphasis was to develop a criterion which will be useful in practice. Therefore, the number of roughness parameters in the equations was kept to a minimum, as long as the parameters are sufficient to capture the essential behavior related to peak shear strength of rough joints.

2 BACKGROUND

The "Joint Roughness Coefficient", JRC (Barton, 1973), various statistical parameters and the fractal dimension, D (Mandelbrot, 1983) have been suggested as parameters to quantify roughness of rock joint profiles along linear profiles (or in 1D).

2.1 Joint Roughness Coefficient

Barton (1973) introduced JRC to quantify roughness in one dimension. Barton has suggested two methods to choose an appropriate value for JRC. In the first method, the roughness profile of the natural joint is visually compared with ten standard profiles published by Barton (1973). For these standard profiles JRC values have been assigned between 0 and 20 in steps of 2 starting from the smoothest to the roughest. This visual comparison method has been found to be subjective and unreliable by some investigators (McWilliams, 1993; Maerz and Franklin, 1990; Wakabayashi and Fukushige, 1992; among others). In the other method, tilt and pull tests should be performed on natural discontinuities to estimate JRC by back analyzing the test results using the empirical equation suggested for peak shear strength by Barton (1973) along with the estimations for normal stress and basic friction angle on the joint plane, and joint wall compressive strength. The practical merit of the second method has been questioned (McWilliams et al., 1993). It is important to note that the ten profiles mentioned above are stationary. The stationary profiles satisfy the following second-order stationary properties: (a) The mean surface is a constant with respect to spatial location; (b) The variance of the surface height around the mean surface is a constant with respect to spatial location; and (c) The covariance function of the surface height depends only on the lag distance irrespective of the spatial location. On the other hand, most of the natural rock joint profiles are non-stationary. JRC, even though it may be a useful index for stationary roughness, does not have the capability of capturing the non-stationarity.

2.2 Statistical Parameters

A recent paper (Kulatilake et al., 1994) has provided a literature review on quantification of roughness using conventional statistical parameters. The same paper has presented a new peak shear strength criterion for rock joints which includes two statistical parameters.

2.3 Fractal Dimension

Euclidian geometry is concerned with the mathematics of perfectly and precisely defined shapes. However, such perfection is rarely found in the real world, with respect to the shapes of natural objects. Fractal geometry introduced by Mandelbrot (1983) allows the description of irregular shapes which are more complex than the Euclidian geometric forms such as spheres, cylinders, planes or their derivatives. The concept of fractals can be understood by considering the existence of a family of mathematical functions that are continuous but nowhere differentiable. Linear profiles across a rough surface belong to this class of mathematical functions. The fractal dimension, D, has been used to characterize a feature having a fractal property. The fractal dimension is a fraction lying between the topological and Euclidian dimensions and describes the jaggedness or degree to which the fractal function fills up the Euclidian space. A linear profile across a rough surface may have a D between 1 (the topological dimension of a line) and 2 (the dimension of a Euclidian plane). Similarly, a rough surface may have a D between 2 and 3.

Some researchers (Brown and Scholz, 1985; Aviles et al., 1987; Lee et al., 1990; Power and Tullis, 1991; Huang et al., 1992; McWilliams et al., 1993) have investigated the possibility of using fractal dimension to provide a measure for roughness of rock fractures. Such work relies on the assumption that natural rock fracture surfaces may be represented by self-similar or self-affine fractal models. Self-affine fractal models may have a better chance to be applicable to geologic phenomena than self-similar fractal models. In essence, a self-similar fractal is a geometric feature that retains its statistical properties (statistical moments, to be more precise) through various magnifications (scales) of viewing. In contrast, self-affine fractals remain statistically similar only if they are scaled differently in different directions.

Four methods have been suggested to estimate D of a profile of a rough surface: the divider method, the box counting method, the spectral method, and the variogram method. In this paper, the background is provided only for the estimation of D by using the spectral method. A future paper will present the background for estimating D by using the other three methods.

2.3.1 Spectral method

Brown and Scholz (1985), Power and Tullis (1991), Huang et al. (1992) and McWilliams et al. (1993) used spectral analysis to estimate D for synthetic as well as natural rock joint profiles. They found that the estimated D depended upon frequency or the spatial scale used for the topography of the natural rock surfaces they investigated. Huang (1992) found a significant effect of non-stationarity on estimated D. This indicates the importance of removal of non-stationarity of the profile prior to performing spectral analysis. It is highly questionable whether D alone is sufficient to quantify natural rock joint roughness. This paper shows that D alone is insufficient to characterize rock joint roughness through spectral analysis. Incorporating two spectral/fractal parameters to quantify stationary roughness and another parameter to quantify non-stationary roughness, a new peak shear strength criterion is developed for rock joints in this paper.

3 EXPERIMENTAL STUDY

Silicone rubber was used to obtain a pair of perfectly mated silicone rubber casts to represent the topographic features of a natural rock joint of about 100 mm diameter. These silicone rubber casts were then used to prepare a series of model material joints (a mixture of plaster of paris, sand and water) having the same cross section and topographic features of the natural rock joint. This procedure was repeated for another two rock joints. Table 1 provides the rock types, core diameters, model mixtures, and curing conditions used in preparing model joints for three groups named A, B and C. For example, Figure 1 provides the surfaces of the natural joint, silicone rubber cast and the model joint obtained for group A.

For each group, since all the model material joints were made from the same silicone rubber cast, it was sufficient to measure the surface roughness of only one model material joint surface. A stylus profilograph was used to perform the roughness measurements. Roughness profiles were obtained in six directions (at every 30 degrees) on the model joint surface (Fig. 2a). In each direction, roughness profiles were obtained along nine parallel lines which are spaced at equal distance (Fig. 2b). Along each line the surface height was recorded at every 0.2 mm up to an accuracy of 0.01 mm.

Each pair of perfectly mated model material casts was used to prepare a model material shear sample to perform a direct shear test in a large Wykeham Farrance shear machine having a square

Table 1. Main features of the model material joint preparation for the three groups

Group	Rock type	Core diameter	Model mixture used plaster: sand: water by weight	Cured conditions
A	Slate	10 cm	1 : 1.5 : 0.83	tem.=20~25°C humd.=50~60% days = 3
B	Sandstone	10 cm	1 : 1.5 : 0.83	tem.=20~25°C humd.=50~60% days = 3
C	Slate	10 cm	1 : 0.33 : 0.73	tem.=20~22°C humd.=60~65% days = 10

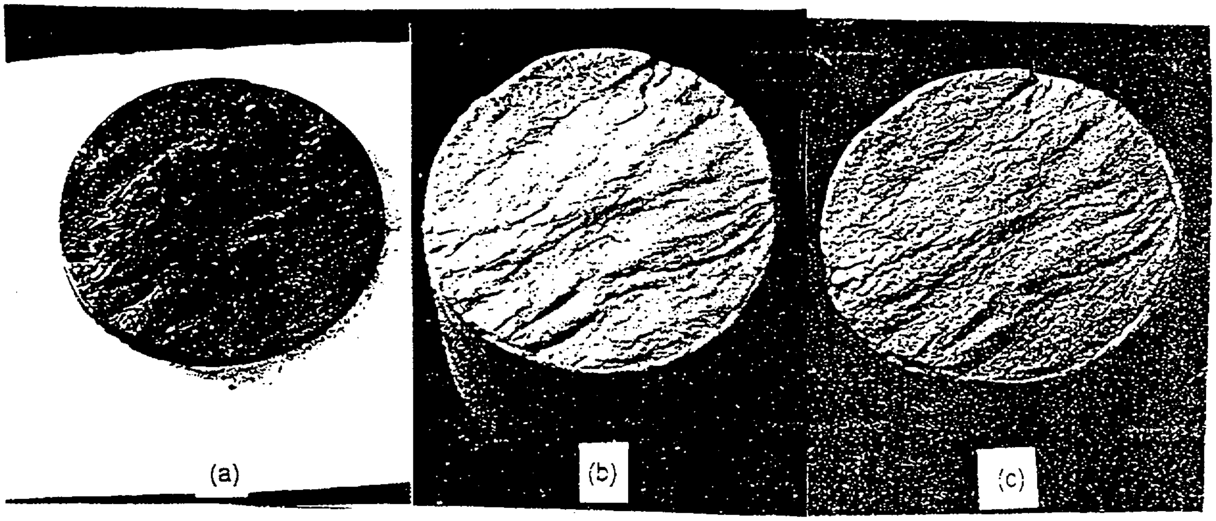

Fig. 1. Photographs illustrating the surface of (a) natural rock joint (b) silicone rubber cast, and (c) model joint for group A.

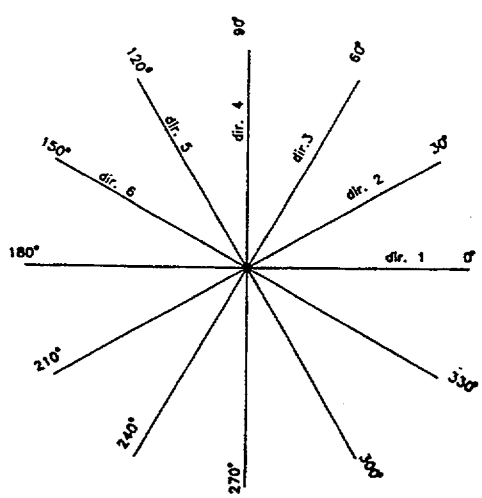

Fig. 2a Directions along which joint roughness measurements and joint shear testing were performed.

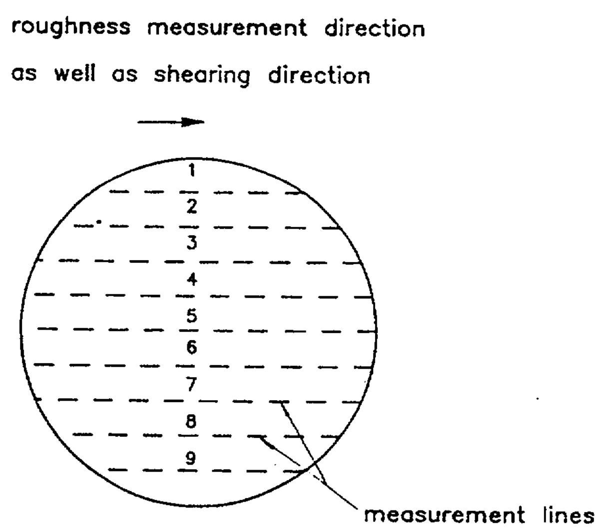

Fig. 2b Lines along which roughness measurements were performed in each direction.

Table 2. Measured shear strengths of the model joints in different directions for Group A

Shearing direction (degrees)	Shear strengths (kg/cm^2) at following σ (kg/cm^2) values				
	σ=0.5	σ=1.0	σ=3.2	σ=5.0	σ=10
0	.94	1.21	2.65	4.68	7.33
180	1.54	2.24	3.99	5.59	8.3
30	1.09	1.47	3.25	4.75	7.66
210	1.33	1.50	4.32	6.14	8.62
60	.76	1.38	2.72	4.65	6.97
240	.94	1.38	3.63	5.04	7.42
90	.85	1.31	3.8	5.06	7.82
270	.72	1.15	3.02	4.22	7.48
120	.99	1.46	3.46	4.85	8.01
300	.73	1.11	3.16	4.68	6.92
150	.99	2.2	3.95	5.41	8.46
330	.82	1.55	2.69	3.99	7.53

Basic friction angle of model material $\phi = 24°$

Joint compressive strength of model material $\sigma_j = 40\,kg/cm^2$

cross section shear box of size 12 inches. Each shear sample was subjected to a selected normal stress in the range 0.5 - 10 kg/cm^2 and was sheared either in the forward or backward direction along one of the six directions used for roughness measurements. Using this testing scheme, a total of twelve direct shear test results were obtained for a selected normal stress. Five and four different normal stress levels were used, respectively, for groups A & B, and group C. All shear tests were performed until failure. The shear strength data obtained made it possible to study the anisotropic behavior of the peak shear strength of the joints resulting from anisotropic surface roughness. For each group, about four shear tests were performed under different normal stresses on smooth horizontal joint surfaces made out of the model material to estimate its basic friction angle. Two tests were performed to determine the joint compressive strength of the model material for each group. For example, the test results obtained for group A are shown in Table 2. These results show that the peak shear strength is anisotropic and depends on the normal stress.

Figure 3 shows typical peak shear strength versus normal stress diagrams for group B. The corresponding roughness profiles for these shear strength data are shown in Part C of the figure. It can be seen that all strength curves are non-linear. These curves show that the peak shear strength can be different between the forward and backward shearing directions. Values predicted by Barton's equation are also shown in the figure for comparison purposes. The visual comparison method was used to estimate JRC values for the roughness profiles. Considerable differences exist between the measured values and the values predicted by Barton's equation.

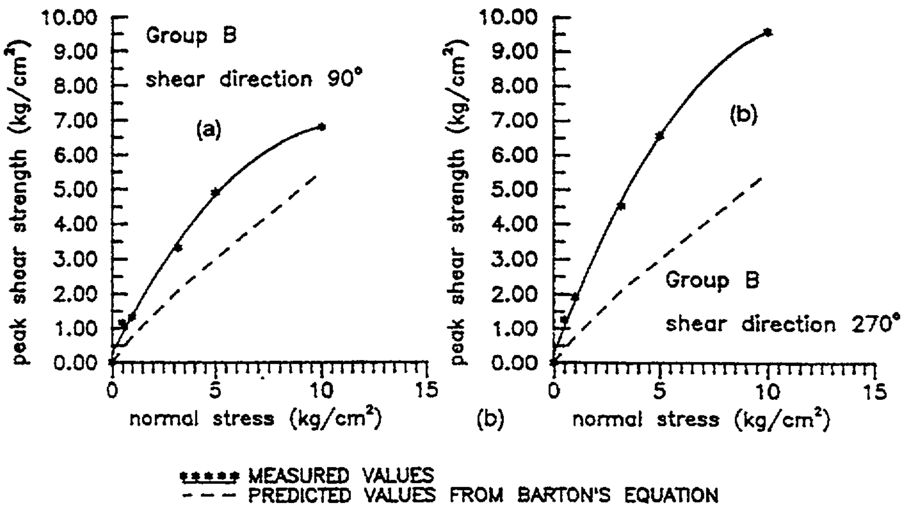

Fig. 3 Typical direct shear test results for group B (a) shear direction 90° (b) shear direction 270°.

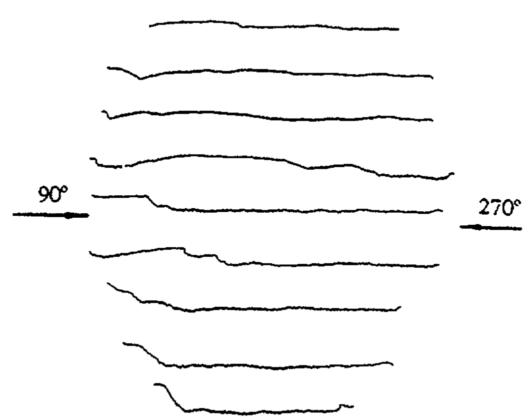

Fig. 3c Roughness profiles in the direction of 90° - 270° for group B.

4 QUANTIFICATION OF ROUGHNESS

The power spectral density is computed essentially by breaking a time series into a sum of sinusoidal components with its own wavelength, amplitude and phase. When performing spectral analysis, the variable "time" can be replaced by a space variable. In this research, spectral analysis is applied to joint profile height, treating it as a function of a one-dimensional space variable. The squared amplitude of each sinusoidal component is referred to as its power and a plot of power versus frequency or inverse of wavelength is referred to as the power spectrum. Spectra are usually expressed in terms of density (amount per interval of frequency) because they are regarded as continuous functions of frequency, rather than as existing only at discrete frequencies. The phase indicates the position of the first peak of each sinusoidal component relative to all others. The phase spectrum is a plot of the phase as a function

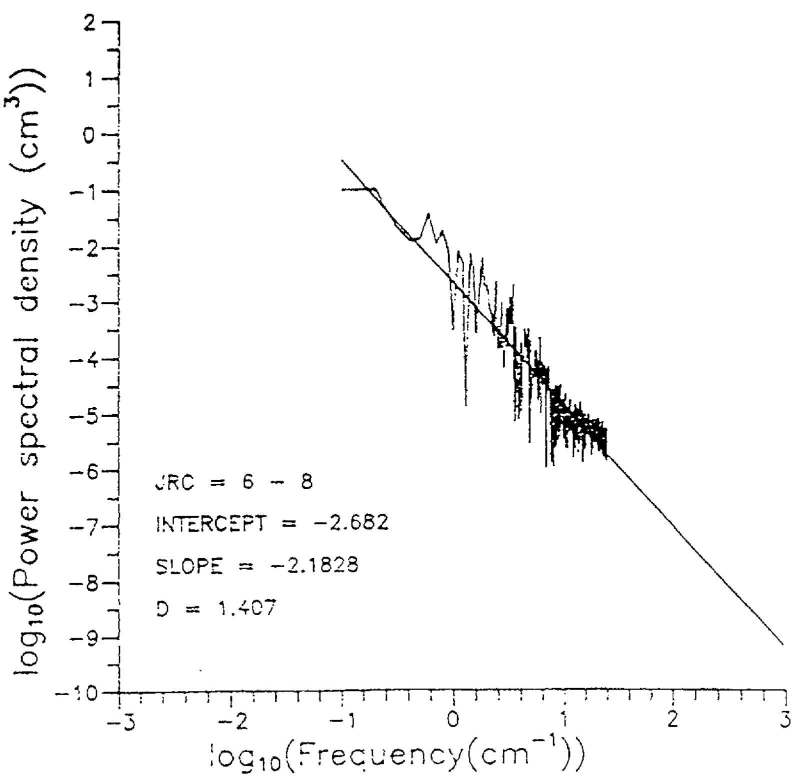

Fig. 4 Spectral analysis results for Barton's profile having JRC = 6 - 8.

of the frequency. Phase spectra for rough surfaces are typically random and there is no consistent relationship between phase and frequency. The fractal dimension of a roughness profile of a surface may be related to the power spectral density, S(f), of the profile if one assumes a spectral density with power-law form, such that $S(f) = K_s f^{-\beta}$, where f is the frequency (e.g. in units of cyc/mm for the profiles) and K_s and β are coefficients in the equation. It is important to note that only the stationary profiles have the chance of satisfying the power law. Fractal dimension, D, may be related to the exponent β of the spectral density according to $\beta = 5 - 2D$ (Berry and Lewis, 1980). Since 1 < D < 2 for roughness profiles, β should be between 1 and 3 for the same. It may be important to note that the zeroth moment and the second moment of the power spectrum are the variance of heights and slope of the profile, respectively.

For the total profile, the root mean square value, σ_{rms}, can be expressed as follows (Power and Tullis, 1991):

$$\sigma_{rms} = \left(\frac{K_s}{4-2D}\right)^{1/2} L^{(4-2D)/2} \qquad (1)$$

where L is the profile length and $D = 2.5 - \beta/2$. This equation shows that not only D, but also K_s, is related to roughness. Therefore, D alone is not sufficient to describe stationary roughness. In order to show that both fractal dimension D and K_s are important in quantifying the stationary roughness of a profile, spectral analysis was performed on all digitized data of Barton's ten standard profiles. Typical results from spectral analysis are shown in Fig. 4. From the spectral plot, K_s and D were estimated.

Table 3 Results of regression between JRC and K_s and D combinations

Combination	Multiple R	Std. Est. Error
K_s	0.7397	4.3223
$D - 1$	0.4909	5.5956
$K_s+(D - 1)$	0.5370	6.8202
K_s*D	0.7586	4.1844
$K_s-(D-1)$	0.44781	5.6409
K_s/D	0.7158	4.4847
$K_s^bD^c$	0.8423	3.7014

Regression analyses were then performed between JRC and different combinations of K_s and D. Results are given in Table 3. The best correlation is given by the lowest standard error estimate and the highest multiple R value. It can be seen that the correlation between K_s and JRC is better than that between JRC and D. The parameter $K_s^b D^c$ seems to be the best for describing stationary roughness; the exponents b and c can be determined by performing regression analysis on experimental data.

Surface profiles of many natural rock joints can be non-stationary. At the simplest level, the non-stationarity of the joint surface can be represented by a linear function with a positive or a negative slope. This slope can be denoted by I and can be estimated through regression analysis. For most joint surfaces, this linear function will be sufficient to model the non-stationary portion of the joint surface profile. Therefore, in this investigation, to model the non-stationary part or the large-scale undulations of the surface profile, the parameter I was selected. To model the stationary part of the surface profile, K_s and D parameters were estimated through spectral analysis. The non-stationary part of the profile was removed before calculating the power spectrum of a rock joint profile.

To estimate I of the surface in a certain direction, each I of the nine profiles in the selected direction was calculated; then, using the following formula, the weighted average value of I was estimated and used as one of the roughness parameters.

$$\text{Average } I = \frac{1}{\sum_{j=1}^{9} l_j} \sum_{j=1}^{9} I_j l_j \tag{2}$$

where $I_j = I$ angle estimated for jth surface height profile in the direction considered

Since, for a circular cross-sectional joint surface, the lengths of the profiles are different, the weighted average values computed by the following equations were used to obtain average K_s and D.

$$\text{Average } K_s = \frac{\sum_{j=1}^{9} (K_s)_j l_j}{\sum_{j=1}^{9} l_j} \tag{3a}$$

$$\text{Average } D = \frac{\sum_{j=1}^{9} (D)_j l_j}{\sum_{j=1}^{9} l_j} \tag{3b}$$

where $(K_s)_j = K_s$ estimated for jth surface height profile in the direction considered

$(D)_j = D$ estimated for jth surface height profile in the direction considered.

l_j = straight length of the jth surface height profile in the direction considered.

5 SUGGESTED PEAK SHEAR STRENGTH CRITERION

The following equation was used as the model for the peak shear strength of a rock joint at low normal effective stresses:

$$\tau = \sigma \tan\left(\phi + a(SRP)^c \left[\log_{10}\left(\frac{\sigma_J}{\sigma}\right)\right]^d + I\right) \tag{4}$$

where a, c and d are coefficients to be determined by performing regression analysis on the experimental data. Here, several combined functions of K_s and D were considered to represent the term $a(SRP)^c$ (see Table 4).

Values of K_s and D calculated through spectral analysis in the different directions for the profiles belonging to the three groups demonstrated the roughness anisotropy of the joint surfaces (Shou, 1994). For groups A and B, the peak shear strength data for σ = 0.5, 3.2 and 10.0 kg/cm^2 were used, along with the K_s, D, I, ϕ and σ_J values to estimate the coefficients a, b, c and d through multiple linear regression analysis. For group C, the peak shear strength data for σ = 1.0 and 10.0 kg/cm^2 were used for regression analysis. Regression results for group A are given in Table 4. It can be seen that the combination of $K_s^b D^c$ gives the best correlation. Therefore, the following general equation is suggested to predict peak shear strength of rock joints under low normal effective stresses:

$$\tau = \sigma \tan\left(\phi + a K_s^b D^c \left[\log_{10}\left(\frac{\sigma_J}{\sigma}\right)\right]^d + I\right) \tag{5}$$

The following specific equations were obtained through regression analysis at multiple correlation coefficient R values of 0.9572, 0.9407 and 0.9315, respectively, for groups A, B and C.

Table 4 Regression analysis results for equation (5) using group A data

<table>
<tr><th>Roughness Parameters</th><th>a</th><th colspan="2">c</th><th>d</th><th>Multiple R</th><th>Std. Error of Est.</th></tr>
<tr><td>K_s</td><td>27.08</td><td colspan="2">0.038</td><td>0.8887</td><td>0.9357</td><td>0.1755</td></tr>
<tr><td>$D - 1$</td><td>18.45</td><td colspan="2">-0.3339</td><td>0.8887</td><td>0.9339</td><td>0.1756</td></tr>
<tr><td>$K_s+(D - 1)$</td><td>18.46</td><td colspan="2">-0.3352</td><td>0.8887</td><td>0.9340</td><td>0.1750</td></tr>
<tr><td>K_s*D</td><td>26.55</td><td colspan="2">0.0381</td><td>0.8887</td><td>0.9272</td><td>0.1980</td></tr>
<tr><td>$K_s-(D-1)$</td><td>18.44</td><td colspan="2">-0.3324</td><td>0.8887</td><td>0.9365</td><td>0.1532</td></tr>
<tr><td>K_s/D</td><td>27.57</td><td colspan="2">0.0377</td><td>0.8887</td><td>0.9371</td><td>0.1530</td></tr>
<tr><td rowspan="2">$K_s^b D^c$</td><td rowspan="2">32.49</td><td>b</td><td>c</td><td rowspan="2">0.8887</td><td rowspan="2">0.9572</td><td rowspan="2">0.1354</td></tr>
<tr><td>0.020</td><td>-.573</td></tr>
</table>

$$\tau = \sigma \tan\left(\phi + 32.49 K_s^{0.0204} D^{-0.571}\left[\log_{10}\left(\frac{\sigma_J}{\sigma}\right)\right]^{0.8887} + I\right) \qquad (6a)$$

$$\tau = \sigma \tan\left(\phi + 34.99 K_s^{0.0471} D^{-0.22}\left[\log_{10}\left(\frac{\sigma_J}{\sigma}\right)\right]^{0.9691} + I\right) \qquad (6b)$$

$$\tau = \sigma \tan\left(\phi + 9.828 K_s^{0.0203} D^{1.3485}\left[\log_{10}\left(\frac{\sigma_J}{\sigma}\right)\right]^{0.7552} + I\right) \qquad (6c)$$

5.1 Comparison between Predicted and Measured Peak Shear Strengths

Three researchers who work in rock mechanics were asked to estimate JRC values by visual method for all the roughness profiles considered in this research. The mean value obtained for each profile from the three researchers were used in equation (7) to estimate an average JRC value for each of the six directions considered.

$$\text{Average } JRC = \frac{1}{\sum_{j=1}^{9} l_j} \sum_{j=1}^{9} (JRC)_j l_j \qquad (7)$$

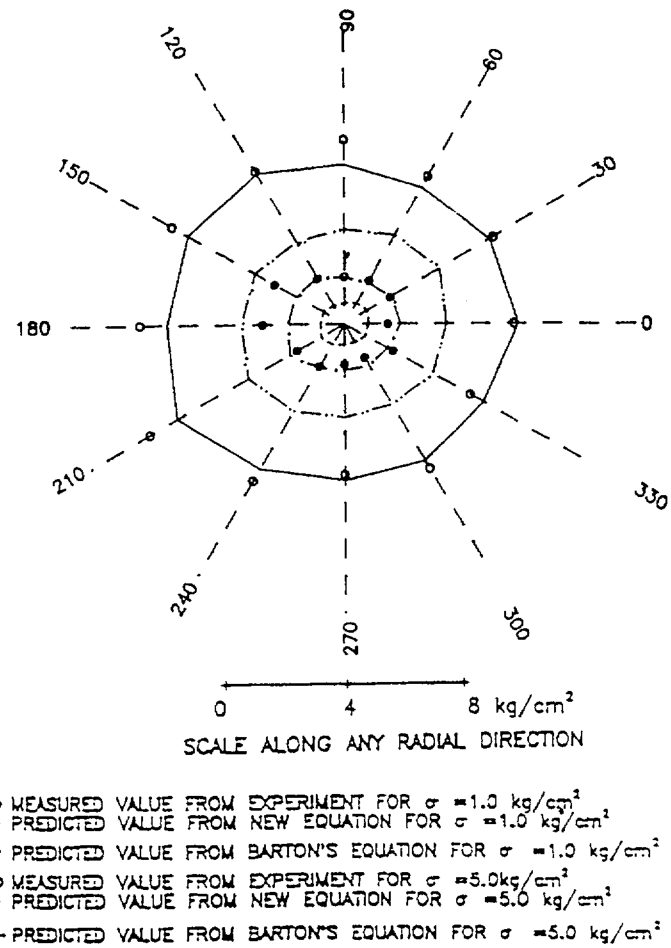

Fig. 5 Comparison between the predicted (based on the new equation and Barton's equation) and measured peak shear strength for group A.

where $(JRC)_j = JRC$ value for jth profile in the direction considered

l_j = straight length of the jth surface height profile in the direction considered.

The average JRC value estimated for each direction was then used in Barton's shear strength equation to predict the peak shear strength of the model joint in the direction considered for $\sigma = 1.0$ and 5.0 kg/cm^2 for groups A and B. For group C, predictions were made for $\sigma = 2.5$ and 5.0 kg/cm^2.

Equations (6a-c) were used, respectively, to predict the peak shear strength at $\sigma = 1.0$ and 5.0 kg/cm^2 in different directions for groups A and B, and at $\sigma = 2.5$ and 5.0 kg/cm^2 for group C. Peak shear strengths were also predicted based on Barton's equation. These predicted peak shear strengths are plotted along with the measured values for group A in Fig. 5. It is clear that the peak shear strength predicted by Barton's equation underestimates the actual peak shear strength significantly in all directions. Also, note that Barton's equation is incapable of capturing the anisotropic shear strength occurring between the forward and backward directions during shearing in any particular direction. It should be

Table 5 Back calculated JRC values using measured shear strengths for group A

Shearing direction (degrees)	Back calculated *JRC* at following σ (kg/cm^2) values				
	σ=0.5	σ=1.0	σ=3.2	σ=5.0	σ=10
0	20.0	16.5	14.1	21.2	20.3
180	25.2	26.2	24.8	26.8	26.1
30	21.7	19.8	19.4	21.6	22.3
210	23.9	20.2	26.8	29.7	27.8
60	17.2	18.8	14.8	21.0	18.1
240	20.0	18.8	22.3	23.5	20.9
90	18.7	17.9	23.5	23.6	23.3
270	16.4	15.6	17.5	17.9	21.3
120	20.1	19.7	21.1	22.3	24.4
300	16.6	15.0	18.7	21.2	17.7
150	20.6	25.9	24.5	25.8	27.0
330	18.2	20.1	14.5	16.2	21.6

Basic friction angle of model material $\phi = 24°$

Joint compressive strength of model material $\sigma_J = 40 kg/cm^2$

noted that when JRC values were estimated by back calculation using the measured shear test data, many of them were found to be greater than 20 [for example, see Table 5]. It is clear that equation (5) has better capability than Barton's equation to predict peak shear strength of rock joints. The new strength criterion can capture the anisotropy of peak shear strength due to the anisotropy of joint surface roughness.

6 DISCUSSION AND CONCLUSIONS

Roughness along a single joint profile can be separated into non-stationary and stationary components. At the simplest level, only one parameter is required to quantify non-stationary joint roughness. Average inclination angle I along the direction of the joint surface considered, estimated using a regression procedure is suggested to capture the non-stationary roughness. Angle I can take positive or negative values, depending on whether it is an inclination or a declination, respectively. Angle I can capture the difference in peak shear strength of a rough joint observed between forward and backward shearing along a particular direction. The suggested new peak shear strength criterion for rock joints takes the following form:

$$\tau = \sigma \tan\left\{\phi + a K_s^b D^c \left[\log_{10}\left(\frac{\sigma_J}{\sigma}\right)\right]^d + I\right\} \qquad (8)$$

Joint roughness data should be used to estimate I, D and K_s. The coefficients a, b, c and d in this strength criterion should be determined by performing regression analysis on experimental shear strength data. Both joint roughness and joint peak shear strength were found to be anisotropic for the three joint surfaces investigated here. This means that, in order to capture anisotropy, it is necessary to quantify all roughness parameters in different directions. When quantifying roughness in a certain chosen direction, first the non-stationary roughness should be estimated using the natural surface profile. Next, this non-stationary part should be removed from the natural surface profile to obtain the stationary component of the surface profile. This component should then be used to calculate the stationary roughness parameters. It is important to note that K_s is a scale-dependent parameter; thus, it can be used to model the scale effect. On the other hand, D reflects the rate of change in length in response to change in the scale of measurement r.

It is also important to note that, for smooth joint surfaces, the value of $a K_s^b D^c$ becomes 0. Then, equation (8) reduces to equation (9), which is applicable for smooth inclined joint surfaces.

$$\tau = \sigma \tan(\phi + I) \tag{9}$$

For horizontal smooth joint surfaces, angle I is zero. This reduces equation (9) to equation (10), which is applicable for smooth horizontal joint surfaces.

$$\tau = \sigma \tan \phi \tag{10}$$

The suggested new equation has a good capability for predicting the anisotropic peak shear strength of joints; it is applicable to either stationary or non-stationary rock joint surfaces. The new equation provides more accurate predictions than Barton's equation for joint peak shear strength. In practice, to allow for modelling uncertainties, the new equation should be used along with a factor of safety of about 1.5.

It is important to note that the suggested new equation is based on experimental shear strength data for model material joints and roughness data for rock joints. It is essential to collect experimental shear strength and roughness data on rock joints to check the applicability of this new shear strength criterion for rock joints. The investigation reported in this paper can be considered as a research initiation on the topic. Further experimental, theoretical and analytical research is very much needed to produce improved techniques for both anisotropic roughness characterization and anisotropic peak shear strength modelling of natural rock joints.

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DRY FRACTURE METHOD FOR SIMULTANEOUS MEASUREMENT OF IN-SITU STRESS STATE AND MATERIAL PROPERTIES

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ABSTRACT: Based on the dry fracture principle (Serata *et al.*, 1992), a computerized borehole probe has been developed to measure stress state and material properties, simultaneously. The probe is designed to obtain a series of measurements in a continuing sequence along a borehole length, without any interruptive measures, such as resetting packers, taking indentation of borehole wall, overcoring, etc. The new dry fracture probe for the single fracture method is designed to overcome the difficulties posed by its ancestor which was based on the double fracture method (Sakuma *et al.*, 1989). The accuracy of the single fracture method is confirmed by a close agreement with the theory, FE modeling and laboratory testing.

1 INTRODUCTION

Numerical methods used in design and analysis of underground structures have been advanced with increasing potential for field application. There is, however, a major barrier against the application: that is the difficulty in determining accurate in-situ stress state and material properties. Without data from actual measurements, not only the input parameters are not sufficiently reliable, but also the output results can not be validated. Unfortunately, conventional measurement methods including overcoring, hydrofracturing and core relaxation are not directly intended for such application.

Recognizing such problems, Serata Geomechanics, Inc. (SGI) has developed a computerized borehole probe based on a new and innovative technique called dry fracture method. The probe is constructed to measure stress state and material properties simultaneously, and is designed for applications in a broad range of complex ground conditions including inelasticity, time-dependency, and pre-fracturing.

This paper summarizes the development of dry fracture method from the early double fracture probe to the latest single fracture probe. Various problems encountered during field applications are identified. Case examples are presented from the measurements at the Yucca mountain repository site and at an underground excavation in Japan.

2 DEVELOPMENT OF DOUBLE FRACTURE METHOD

Dry fracture borehole probe was initially developed base on double fracture method (Serata *et al.*, 1992). The design principle is similar to the hydrofracturing method, excepts that a plastic tube is used for borehole loading. Since the plastic tube does not allow the loading liquid to escape from the probe, the loading pressure can be increased beyond the initiation of the first fracture. The increasing pressure, therefore, induces the second fracture in the direction perpendicular to the first one. Opening of the fractures is detected by LVDT's mounted inside the tube (Fig. 1).

As shown in the profile of Fig. 1, four LVDT's mounted 45 degrees apart detect the fracture opening on the borehole wall. The plastic tube enables us to eliminate the need for setting borehole packers and for taking fracture indentation. These lead to a non-interruptive measurements for a consecutive sequence over the length of the borehole.

Typical results of double fracture method can be observed in real-time on the computer screen during measurement. Fig. 2 gives a form of measuring displays showing loading pressure vs. diametral deformation (p-D diagram). The diagram reflects the surrounding ground behavior under loading and unloading cycles. Initiation of the first fracture is represented by the deviation of the p-D curve from the elastic straight line at point B in the figure. Considerable amount of ground consolidation is detected by the non-recoverable deformation (represented by D). Re-opening of the previously-induced fractures is detected by point E which is found in the second loading cycle. The pressure difference between B and E represents the borehole tensile strength of the ground. Each measurement provides a set of four p-D curves (i.e. four LVDT's), and hence in an intact ground, the directions of the maximum and minimum stresses on the plane perpendicular to the borehole can be identified (Fig. 3). Here the stress state is determined from the upper and lower deviation pressures of the four curves. Assuming a linear elastic material, the maximum and minimum stresses (P_o & Q_o) can be calculated from the following relations (Serata *et al.*, 1992):

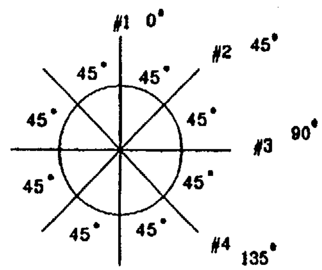
$$P_o = (p_1^E + 3p_2^E)/8 \tag{1a}$$

$$Q_o = (3p_1^E + p_2^E)/8 \tag{1b}$$

Orientation of the stresses is determined from the diametral deformation rosette (Fig. 4). In ground with complex behavior, where the material properties are not isotropic, the slopes of p-D curves is taken into consideration to eliminate the orientation effects. More specifically, the following ratio is plotted in a rosette form to replace the deformation rosette:

$$R = \cot \theta_F / \cot \theta_E \tag{2}$$

where θ_E is slope of p-D curve before E, and θ_F is slope of p-D curve after E. The



Probe O.D.	=	56 mm
Probe Length	=	950 mm
Loading Tube Length	=	500 mm
Max. Borehole Deformation	=	6 mm
Stress Measurement	=	Maximum stress, minimum stres & stres orientation
Property Measurement	=	Young's modulus, Tensile Strength, Failure Strength, Consolidation, Viscoelasticity, Viscoplasticity
Borehole Diameter	=	57 - 60 mm
Borehole Length (Max.)	=	60 m
Data Aquisition System		On-site Real Time Analysis & Display

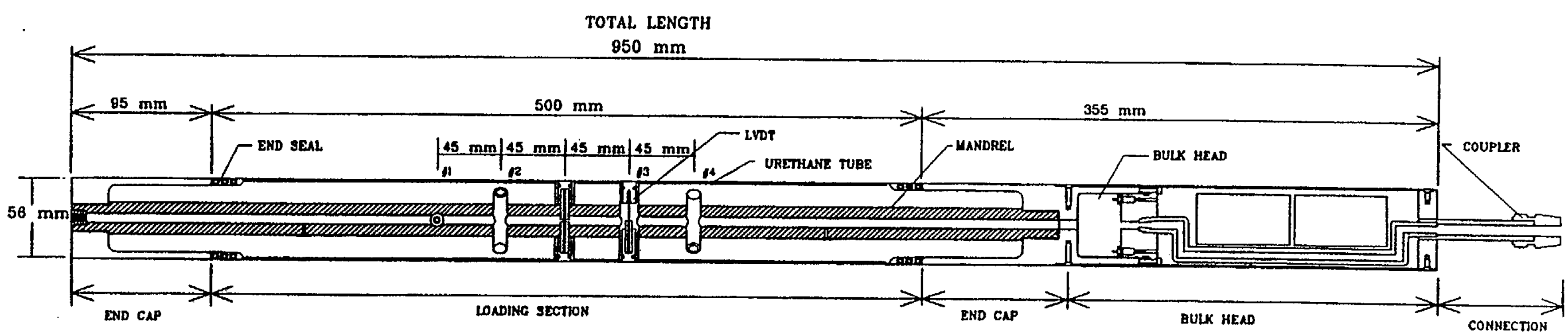


Fig. 1 Profile of miniaturized stress-property measurement probe S-300.

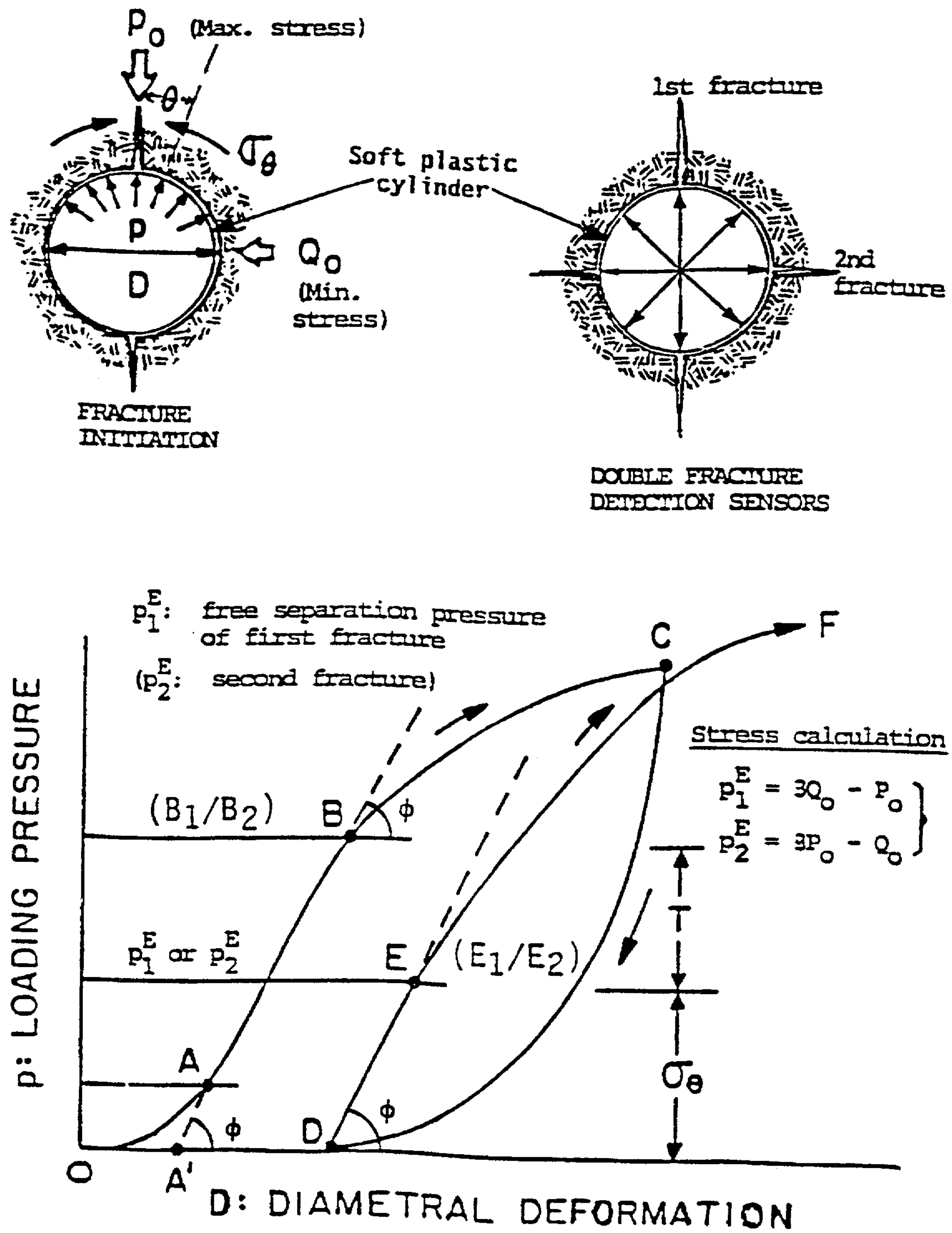


Fig. 2 Idealized loaidng pressure vs. dimetral deformation(p-D) curve under cyclic loading, showing initial fracture point B and free reopening point E in double fracture method.

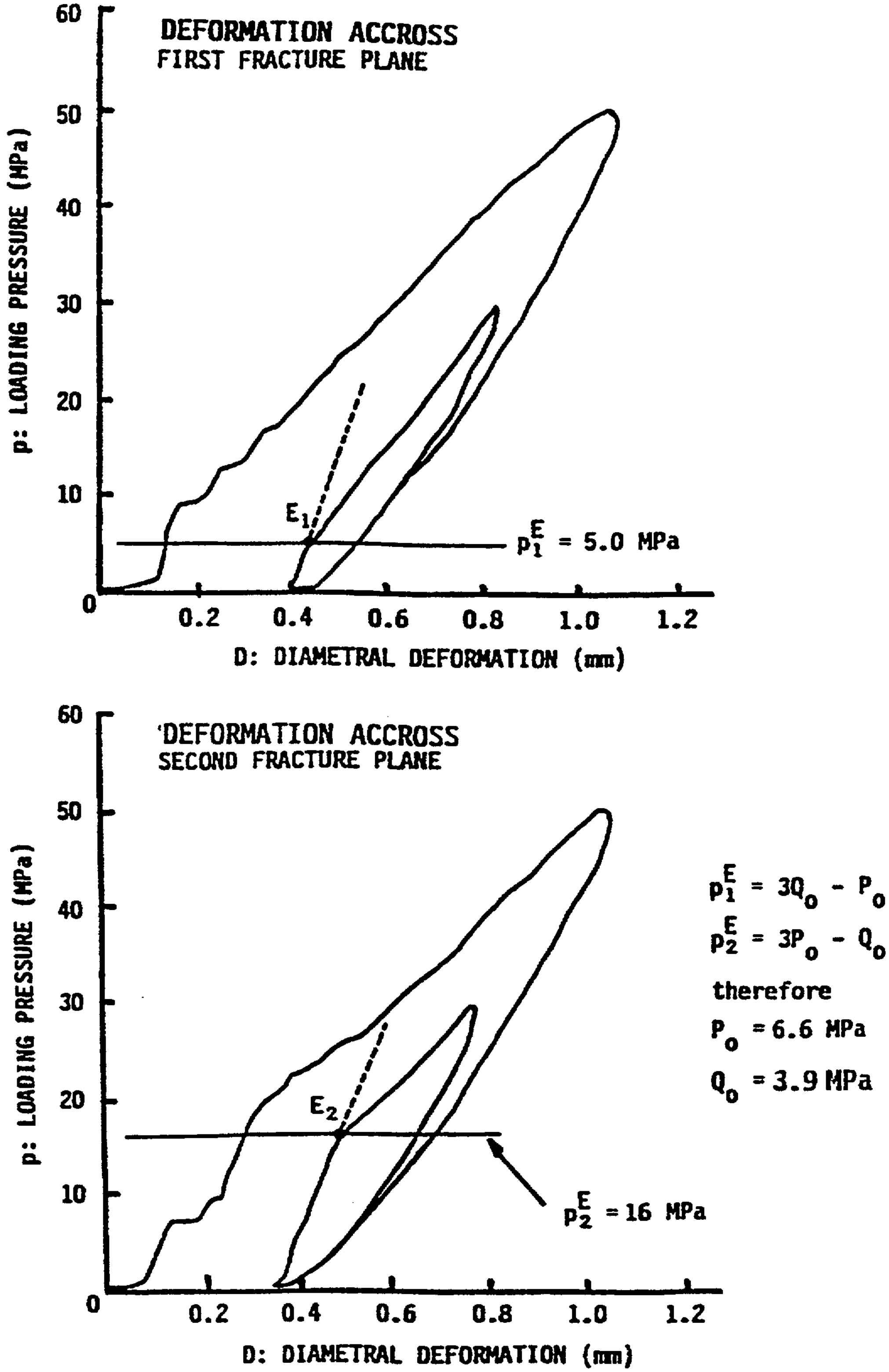


Fig. 3 Typical field observation of pair of p-D diagrams taken in the two mutually perpendicular directions of principal stresses directly disclosing stress state and material properties.

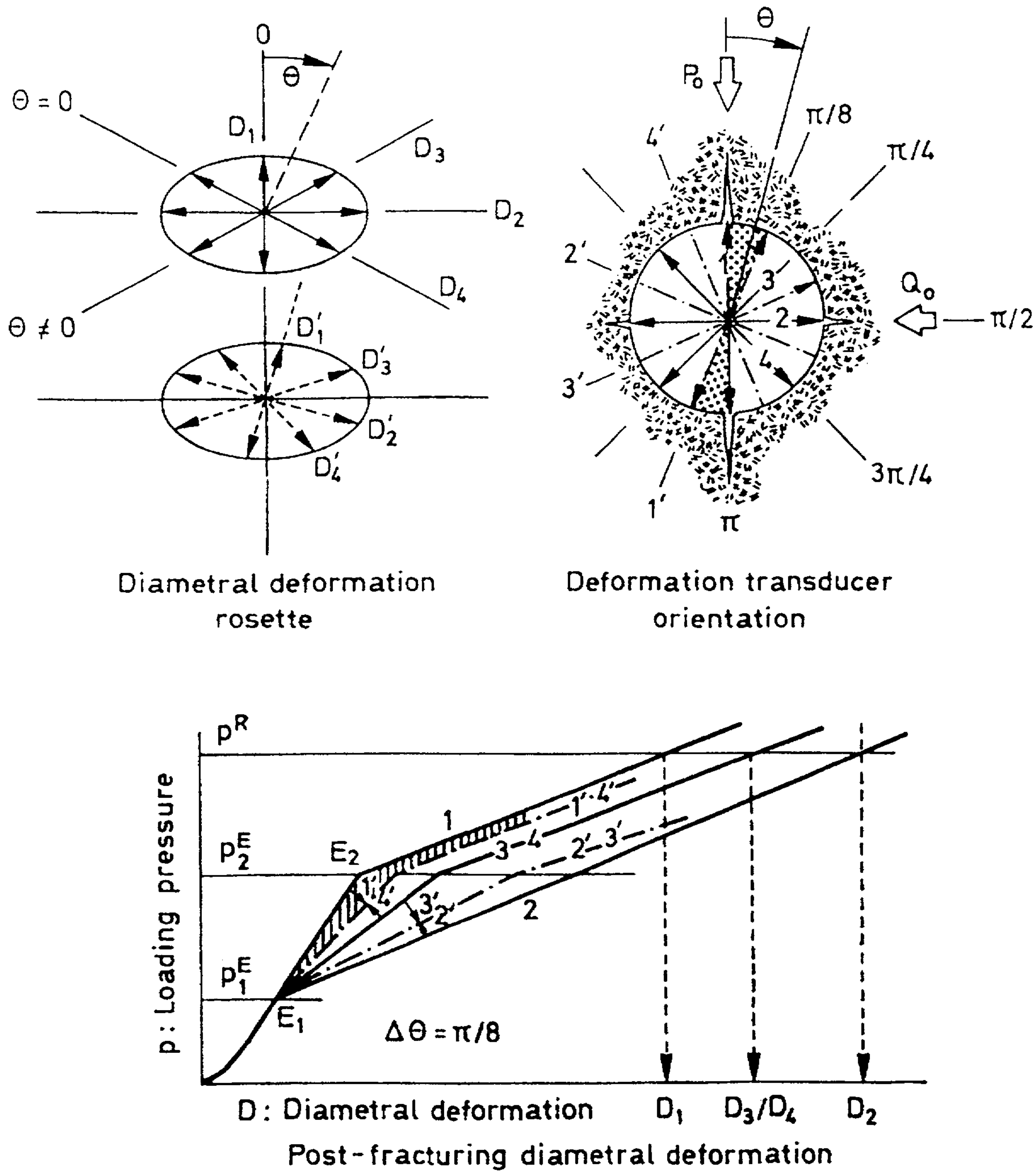


Fig. 4 Two identical deformation rossettes being obtained by double fracture probe set at two different angular orientations, compared with corresponding set of 4 p-D curves monitored by the probe set at $\theta = 0$.

orientational analysis from the R-rosette is similar to the diametral deformation example of Fig. 4.

3 SIMULTANEOUS MEASUREMENT OF MATERIAL PROPERTIES

The material properties of the ground are measured automatically at the same time with the stress measurement. Some common property coefficients are obtained directly from the p-D curves, as shown in Fig. 5. The properties are determined in regard to a set of four angular orientations set by the individual LVDT's mounted in the probe at 45 degrees apart. By rotating the probe at a smaller angular interval, a more detailed information with regard to the ground properties can be obtained, if necessary.

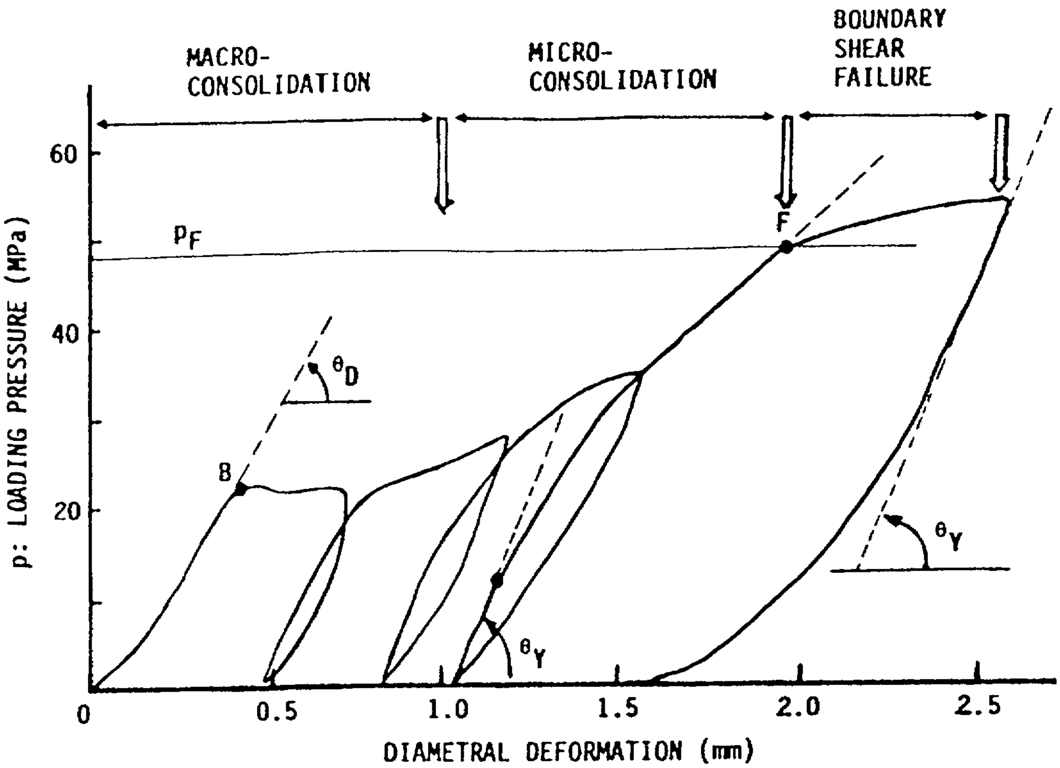
A broad spectrum of p-D curves obtained from our previous field applications ranging from clay to hard granite is shown in Fig. 6. Field examples for more complex properties of the ground are shown in Fig. 7. The p-D diagrams reveal the alteration characteristics of the ground caused by the probe loading. It should be noted in Fig. 7 that the stress measurements are made possible by the consolidation of the soft and loose ground to a pseudo-elastic state due to the cyclic loading. This enables to detect E which, otherwise, will never be detected. This is one of the fundamental advantages over the hydrofracturing method; that is the capability of applying in fractured ground.

Data obtained from this method readily give the coefficient values, such as Young's modulus E_Y, deformation modulus E_D, non-recoverable deformation E_N, and borehole tensile strength T. Here, the time dependency such as visco-elasticity and visco-plasticity can also be determined from the creep deformation under constant probe loading. If more comprehensive collection of property coefficients are required, they can be obtained from back analysis modeling of the borehole behavior using a generalized constitutive equation which is developed specifically for this purpose (Serata and Fuenkajorn, 1992; Fuenkajorn and Serata, 1993 & 1994).

The constitutive equation is graphically show in Fig. 8. This rheological constitution relates nine basic behavioral components of elasticity, visco-elasticity, visco-plasticity, strain softening dilation, permeability and thermal effect in the simplest possible form (Serata and Fuenkajorn, 1992). This formulation is supported by field validation work conducted over the past 30 years. One well-known example is the application at the Sifto salt mine in Goderich, Ontario, in which the mining operation was saved by the design modification to Stress Control. The success is attributed to the combined use of the constitutive model and the probe application (Serata, 1982; Dickie *et al.*, 1986).

4 DIFFICULTIES OF DOUBLE FRACTURE PROBE

The double fracture probe has been applied effectively to a wide range of ground conditions encountered in the underground coal mines (Serata and Fuenkajorn, 1991; Serata



E_Y = Young's modulus
$= 2\,a\,(1 + \nu)\,\tan\theta_Y$

E_D = Deformation modulus
$= 2\,a\,(1 + \nu)\,\tan\theta_D$

ΔE = Consolidation modulus
$= E_Y - E_D$

S_F = Uniaxial failure strength = P_F

Where:
a = Borehole radius
ν = Poisson's ratio
$\tan\theta = \Delta p/\Delta D$

Fig. 5 One p-D curve obtained by one channel of double fracture probe illustrating how to determine common material property coefficient values in each one of four sensor orientations.

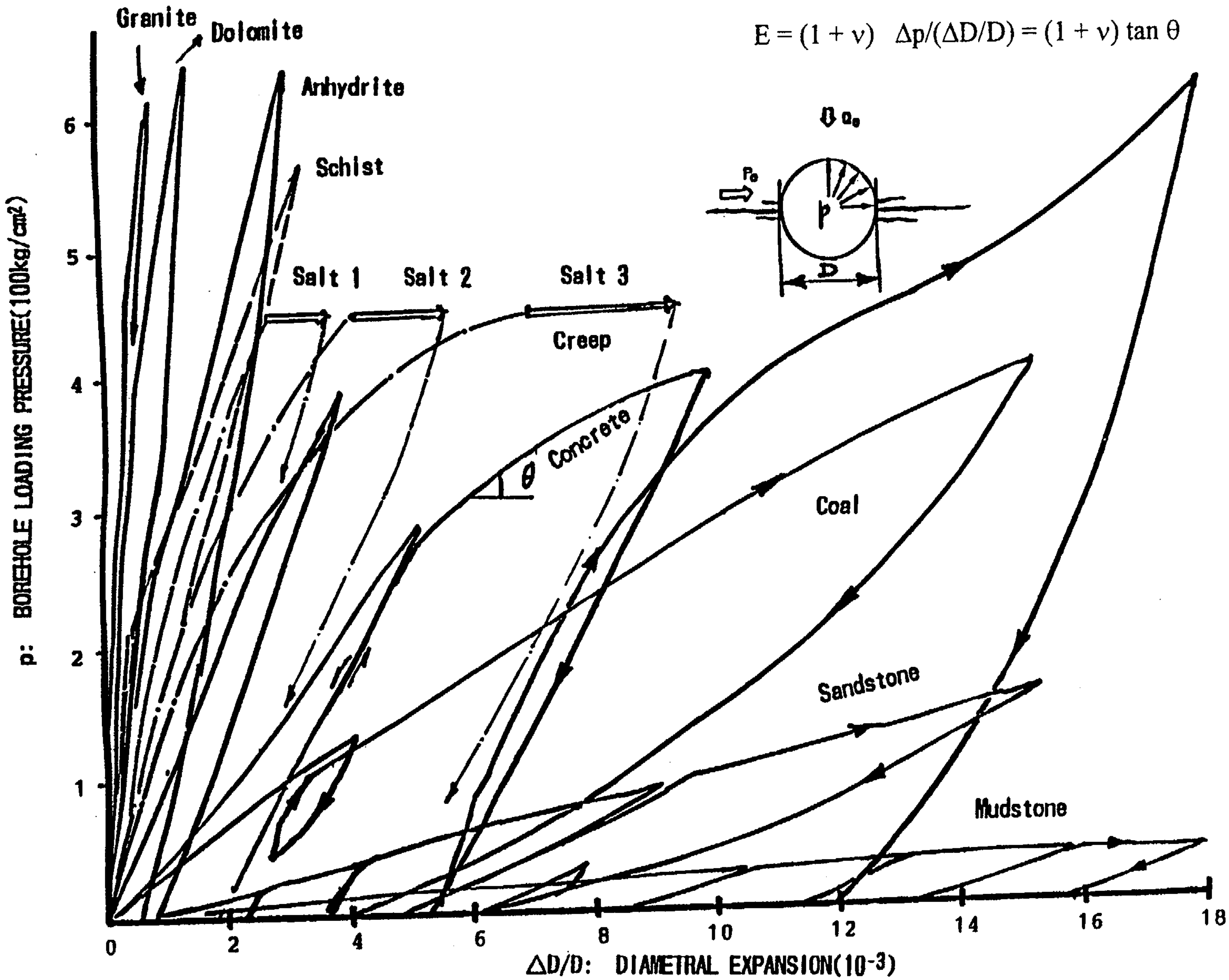


Fig. 6 Comparison of various p-D curves obtained from widely defferednt grounds ranging from soft mudston to hard granite, illustrating effectiveness of material property determination, simultaneously and automatically made with the stress measurement.

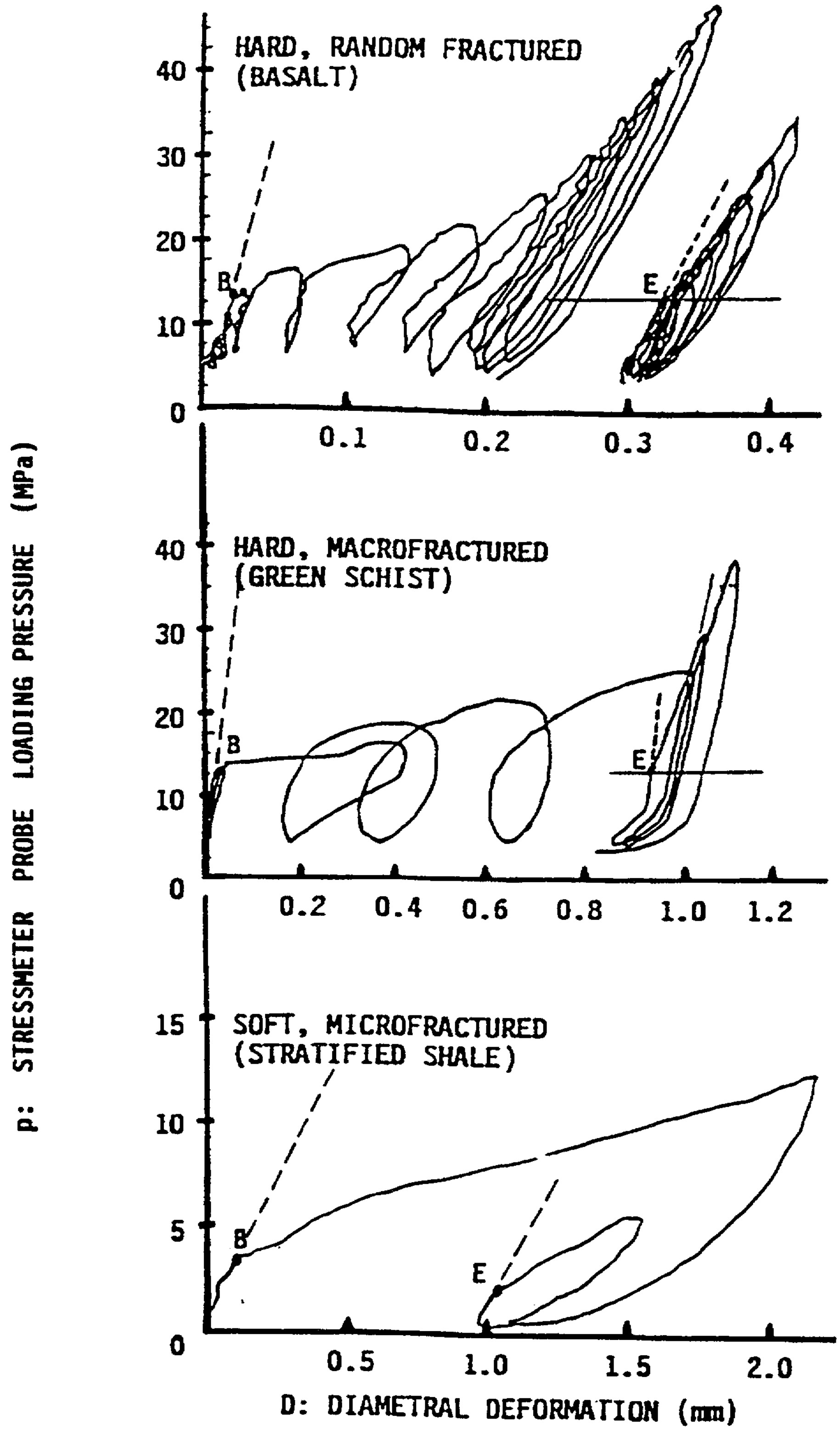
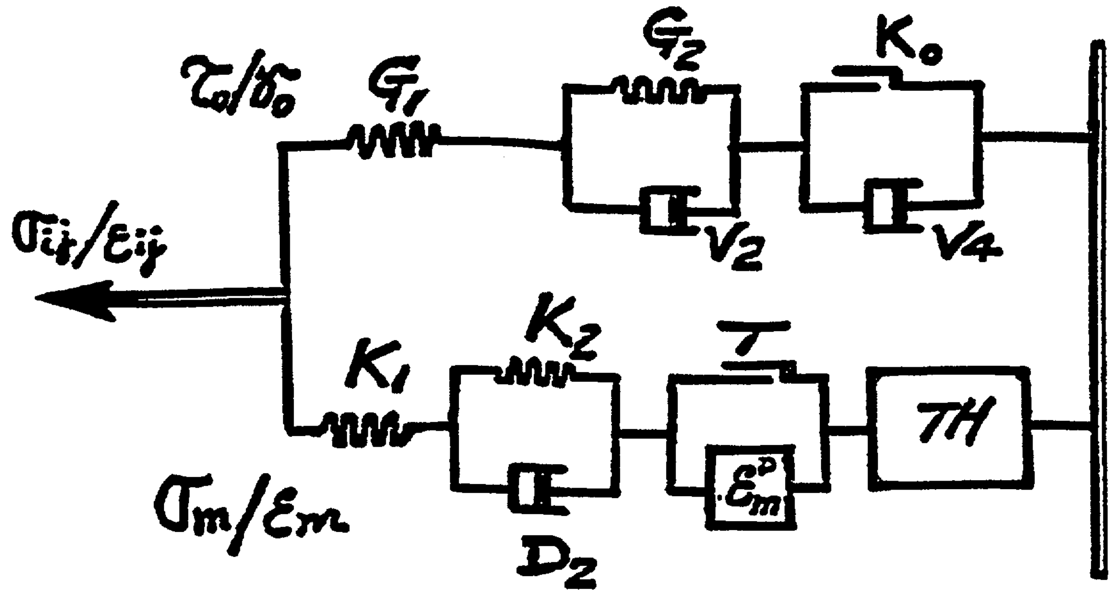


Fig. 7 Three p-D diagrams obtained in three different grounds, respectiively demonstrating alteration of the complex media property to pseudo elastic state adequate for stress measurement.



STRESS AND STRAIN

σ_{ij}: stress tensor

ϵ_{ij}: strain tensor

τ_o : shear stress

γ_o : shear strain

σ_m : hydrostatic stress

ϵ_m : hydrostatic strain

MATERIAL PROPERTY PARAMETERS

G_1: shear modulus

G_2: retarded shear modulus in viscoelastic state

V_2: elastoviscosity in viscoelastic state

V_4: Plastoviscosity in viscoplastic state

K_o: octahedral shear strength

K_1: bulk modulus

K_2: retarded bulk modulus in viscoelastic state

D_2: hydrostatic elastoviscosity in viscoelastic state

T : tensile strength

ϵ_m: volumetric function of deterioration

TH: thermal strain function

Fig. 8 Constitutive relation of generalized earth material utilized for back analysis of material properties by modelling non-elastic and time-dependent borehole behavior.

et al., 1993), and salt and potash mines in North America (Dickie *et al.*, 1992; Serata, 1988) and in the construction work in Japan (Sakuma *et al.*, 1989). In spite of the successful applications, some difficulties have been recognized during the measurements, as summarized below:

4.1 Effect of Pre-Existing Fractures

Pre-existing fractures (macro and micro) are abundant in most ground. Near the borehole, these fractures are usually opened by the tensile stresses during probe loading. Even though these fractures are sometimes small and are not detectable by borehole video logging, their effects on the measurements can be detected, as illustrated in the upper diagram of Fig. 9. Therefore, the ideal p-D diagrams (Figs 2 and 3) are usually not expected. To deal with the randomness of the effects of these fractures, statistical methods are introduced to achieved a desired accuracy of measurements. These statistical improvements require a number of measurements taken at various angular positions at each measurement. Even with this improvement, the stress measurement accuracy very much better than 1.0 MPa can not be achieved in pre-fractured ground.

4.2 Inhomogeneity of Materials

Localized inhomogeneity frequently found in natural rock undermines the applicability of the double fracture method. Laboratory observations and field measurements on this issue are given in detail by (Serata *et al.*, 1992).

4.3 Effect from the Stress State

Double fracture method is strongly affected by the initial state of stresses. The measurement is less reliable when the maximum to minimum stress ratio (P_o/Q_o) becomes larger than 1.5. Under such stress state, inducing the second fracture in the direction perpendicular to the maximum stress becomes uncertain or impossible. This is to impose a severe limitation to the double fracture method because such stress state is found to be very common in nature.

4.4 Probe Weight

The double fracture probes have been designed for 76 and 100 mm diameter test holes. The probes made for the holes weigh between 40 to 60 kg. Underground application of these heavy weight probes is found to be very cumbersome and time consuming, particularly in small tunnels and mine openings.

4.5 High Cost of Measurements

Field application of the probe is found to be much more economical than all others.

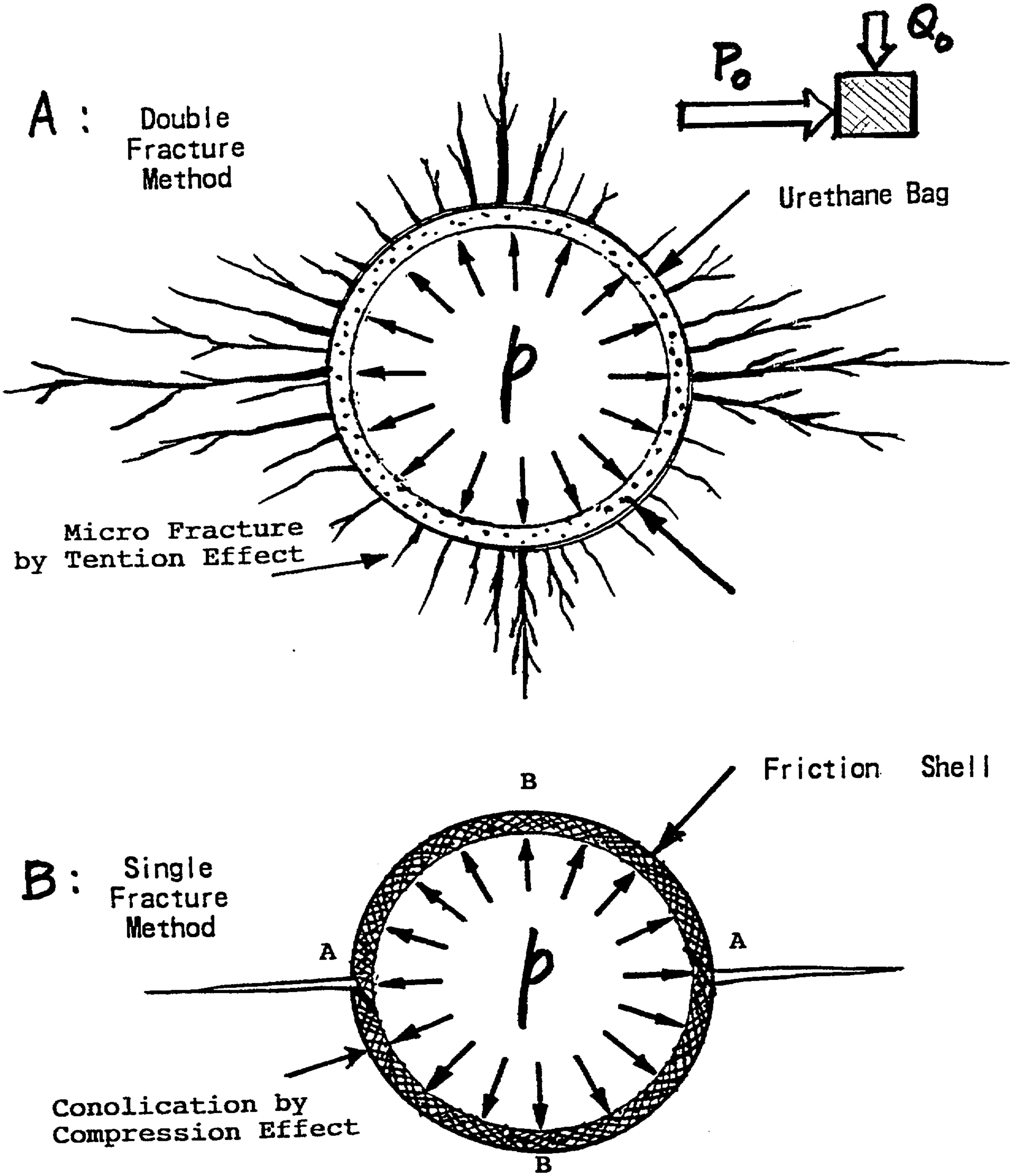


Fig. 9 Comparison of (a) boundary deterioration due to tension effecy caused by double fracture technique of loading with (b) boundary consolidation due to compression effect caused by single fracture technique of loading.

But the economic advantage is shown to be not so much as expected due to the high cost of probe manufacturing, test hole drilling and probe manipulating.

5 DEVELOPMENT OF SINGLE FRACTURE METHOD

The dry fracture probe is drastically improved by introduction of single fracture method to overcome the problems mentioned above. The new method uses the same plastic loading tube to induce dry fracture around borehole by attaching a pair of half-cylindrical shells with high friction completely surrounding the plastic loading tube, as shown in Fig. 9. The friction shells are made of flexible but having extremely high strength in tension to prevent fracture development under the shells so that the tangential tensile stresses are concentrated only at the gap between the two shells (A-A direction in Fig. 9). Fig. 10 illustrates the effect of the shells on the tangential stress distribution at the hole boundary under the loading. The induced tangential stress σ_θ relates to the angular area covered by the shell, expressed here by angle β (from 0 to 90 degrees). Assuming friction locked interface, the tangential stresses under the shell (σ_θ^B) and those not covered by the shells (σ_θ^A) can be calculated using the estimated solutions by Goodman et al. (1968). From the solutions, the tangential stresses for two extreme cases of very soft and very hard rocks can be expressed as follows:

for general rock ($E << 30 \times 10^6$ psi)

$$\sigma_\theta^A = \sigma_\theta - (4\beta/\pi)p \tag{3}$$

$$\sigma_\theta^B = \sigma_\theta + \{2\beta/\pi + (1/2\pi)\ \Sigma\ 3/(m+1)\sin(m+1)\ 2\beta\}\ p \tag{4}$$

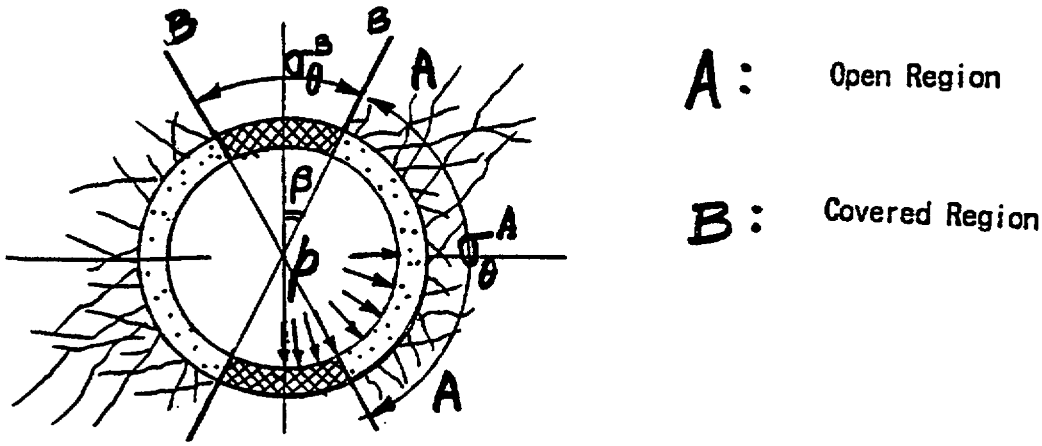
for extremely hard rock ($E > 10 \times 10^6$ psi)

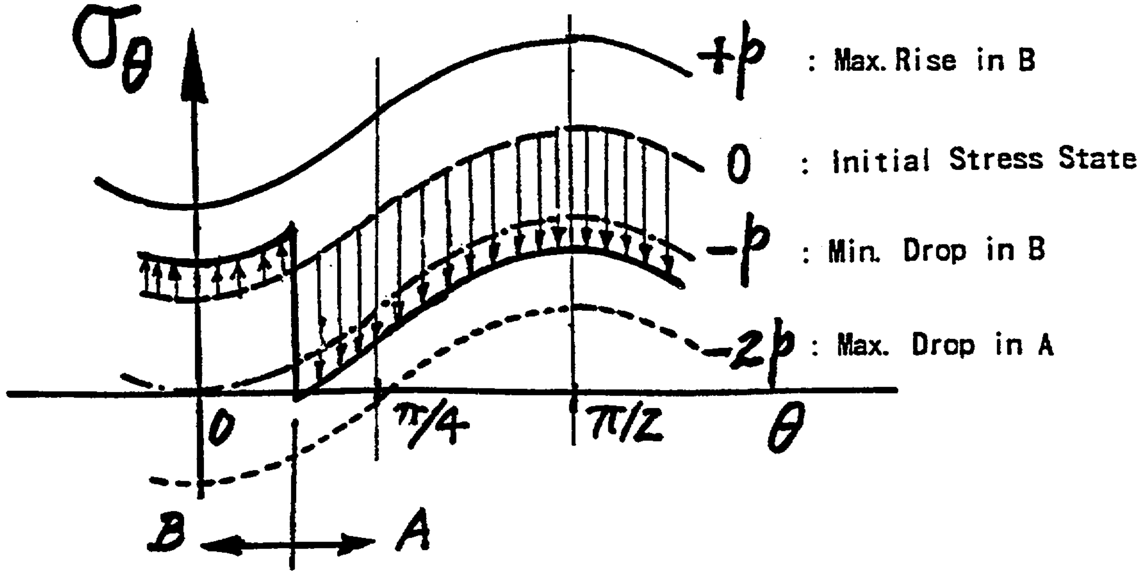
$$\sigma_\theta^A = \sigma_\theta - (4\beta/\pi)\ p \tag{5}$$

$$\sigma_\theta^B = \sigma_\theta + P - (4\beta/\pi)\ p \tag{6}$$

The stress distribution in Fig. 10 illustrates the conditions created by the shells with an arbitrary cover angle of $0 < \beta < \pi/2$. When the angle β approaches $\pi/2$, the stress distribution becomes unique; the tensile stresses are induced only at the gaps between the shells as illustrated in Fig. 11 in which the fracture plane is set in the Q_o direction. It is interesting to note that the tension effect is concentrated at the separations of the shells with a constant value of $\sigma_\theta^A = 2p$, regardless of stiffness of the ground. Equations (3) and (5) are identical and are used to calculated the stress state (P_o, Q_o and θ) using the p value at $p = p^E_i$. This leads to a relation:

$$p_i^E = (1/2)[(3Q_o-P_o) + 4(P_o-Q_o)\sin^2(\theta+\alpha_i)] \tag{7}$$





With general rock (E << 30 x 10^6 psi)

$$\sigma_\theta^A = \sigma_\theta - (4\beta/\pi)\,P$$

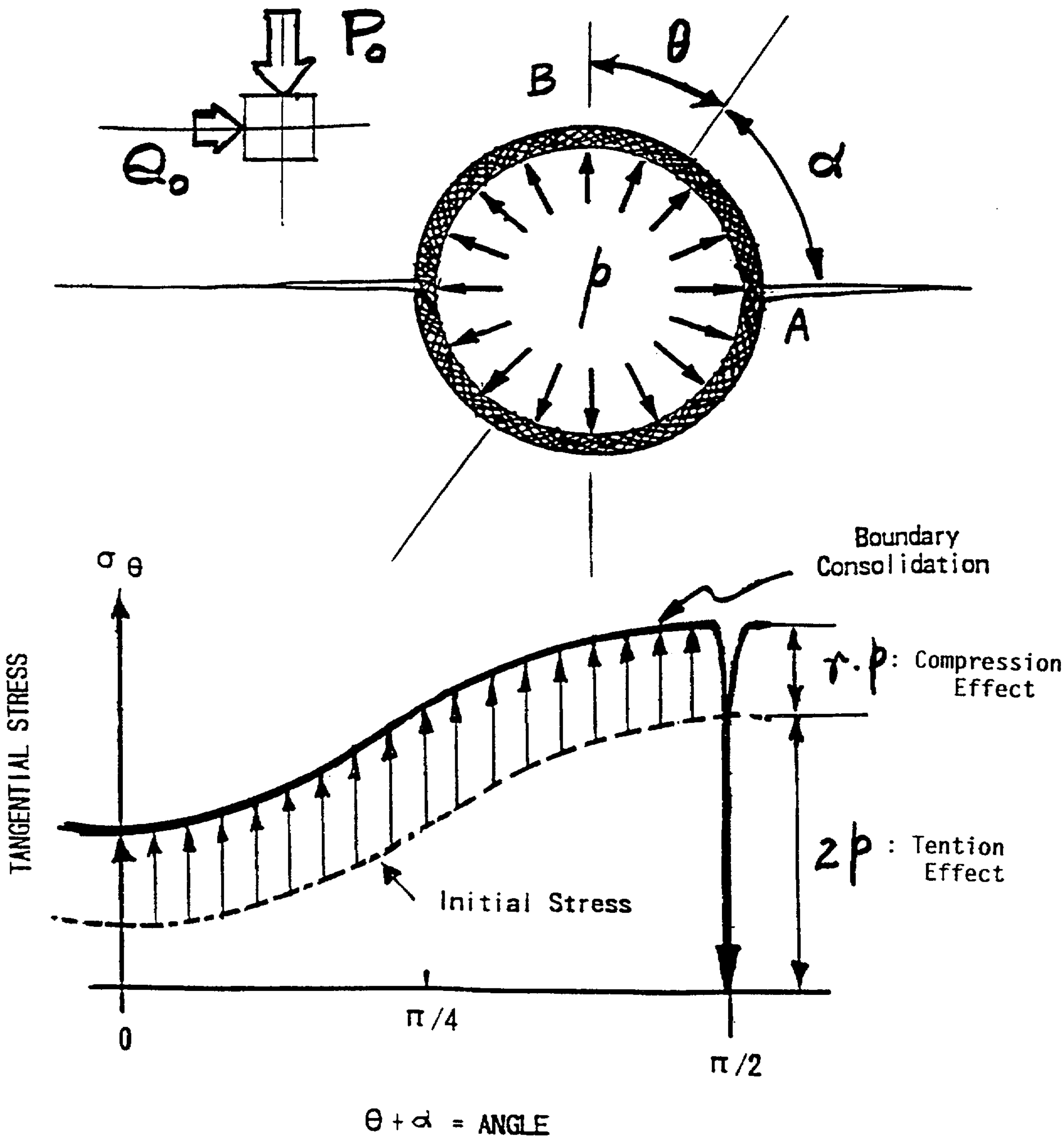
$$\sigma_\theta^B = \sigma_\theta + \{2\beta/\pi + (1/2\pi)\sum_{0}^{\infty} 3/(m+1)\sin(m+1)\,2\beta\}\,P$$

With extremely hard rock (E > 10 x 10^6 psi)

$$\sigma_\theta^A = \sigma_\theta - (4\beta/\pi)\,P$$

$$\sigma_\theta^B = \sigma_\theta + P - (4\beta/\pi)\,P$$

Fig. 10 Strong effect of flexible friction shells on tangential stress distribution around borehole boundary with increasing compression over covered area and increasing tention over non-covered area, having their intensity determined by cover angle of β.



At Fracture Point($\theta + \alpha = \pi/2$)

$\sigma_{\theta A} = (3Po + Po) - 2p$

Over covered area($-\pi/2 \sim 0 \sim + \pi/2$)

$\sigma_{\theta B} = 3Q_o + P_o + 4(P_o - Q_o)\sin^2(\theta + \alpha) + p$

Fig. 11 Optimum design of shell by total achieving maximum compression effect over covered boundary with maximum tension effect concentrated at points of single fracture separation.

where θ is angle of P_o from probe datum direction and α_i is angle of fracture plane from the datum. In order to determine the three unknowns, measurements are made for a set of three different α_i values, usually at 0, 60 and 120 degrees.

6 EVALUATION OF SINGLE FRACTURE METHOD

This theoretical finding was tested by comparing with the finite element modeling with the normalized loading of $P_o = Q_o = 0$, to focus the attention on the mechanism of fracture opening. The stress distribution calculated around the borehole is presented in Fig. 12 for varying hardness of the ground. Close agreement between the simulations and the theoretical solutions on the tension stress concentrated at the fracture points is found in the modeled the p-D curve, as shown in Fig. 13.

Laboratory experiment is conducted on 1 x 1 x 1 m block of cement, with 76 mm diameter hole at the center. The block is biaxially loaded with external stresses of $\sigma_1 = P_o$, $\sigma_2 = Q_o$, and $\sigma_3 = 0$. A single fracture plane was created by applying the double fracture loading p to create only a single fracture in the P_o-direction in the homogeneous and non-fractured test block under the different P_o and Q_o values. Without using the friction shells, re-opening of the single fracture was examined by applying uniform probe loading. Results of the test are presented by p-D diagram in Fig. 14. A close agreement is found between the theory and experiment with the measurement accuracy of about 0.1 MPa. This indicates that the probe with or without shells is capable of measuring the stress with a high accuracy if only one fracture plane is involved. With the introduction of the friction shells, the accuracy and reliability of the stress measurements is expected to be further improved.

For determination of material properties, the single fracture method is found to be more reliable than the double fracture method. This is due to the fact that each single fracture measurement is made specifically for a given orientation. For any probe orientation, the ordinary property coefficient values are obtained according to the theoretical relations, as follows (Goodman *et al.*, 1970).

Young's modulus: $$E_E = (1+\nu)(D/\Delta D_E)\Delta p \tag{8}$$

Deformation modulus: $$E_T = (1+\nu)(D/\Delta D_T)\Delta p \tag{9}$$

Non-elastic coefficient: $$\Delta E = (1+\nu)D\ \Delta p\ (\Delta D_T - \Delta D_E)/(\Delta D_T\ \Delta D_E) \tag{10}$$

Tensile strength: $$T = 2(p^E - p^B) \tag{11}$$

where:
ν = Poisson's ratio,
D = borehole diameter,
ΔD_E = elastic portion of diametral deformation,
ΔD_T = total diametral deformation,

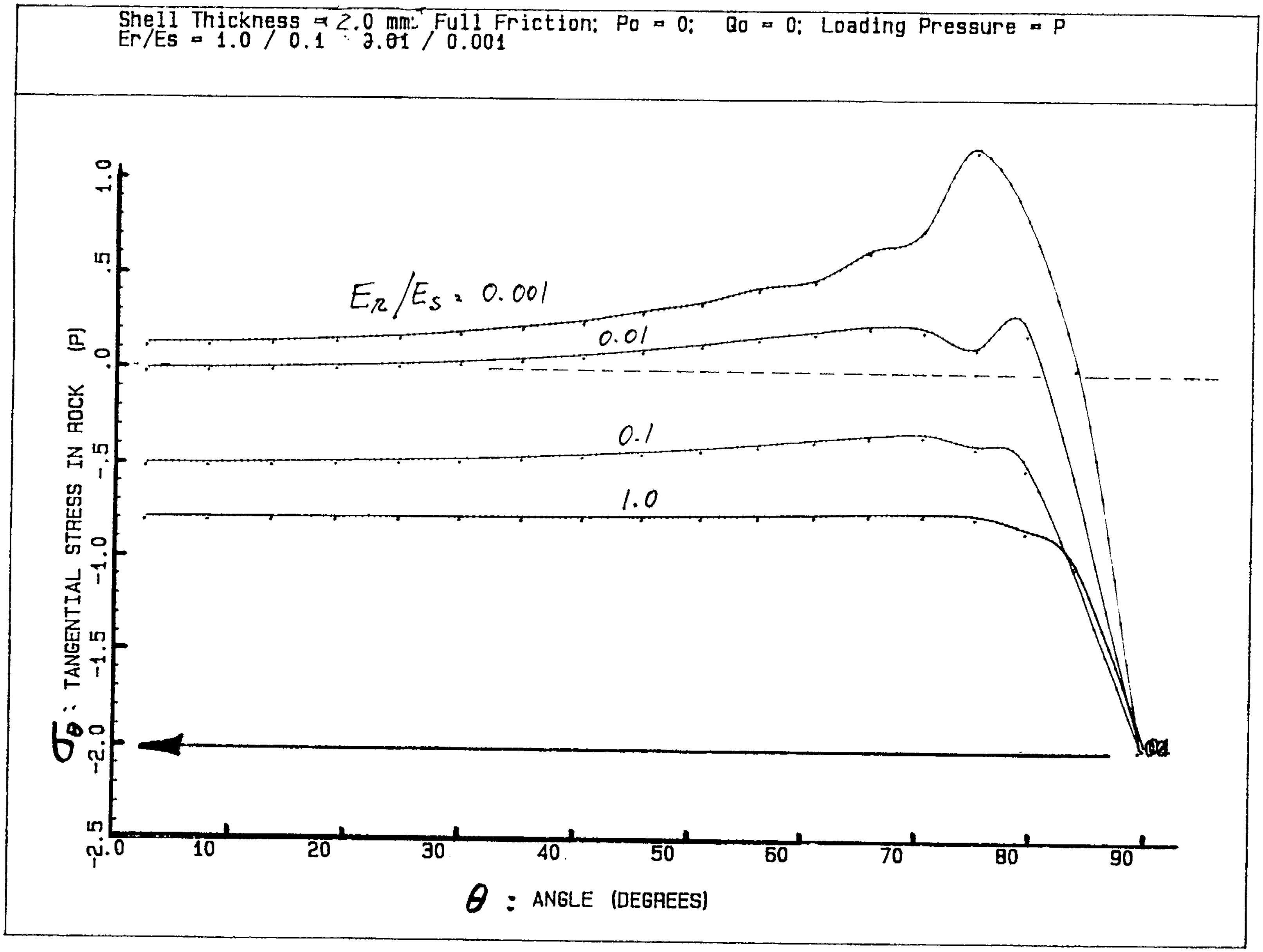


Fig. 12 Distribution of tangential stress on borehole boundary behind flexible friction shell with borehoe loading pressure p with ω = 90°, creating tensile stress of 2P for single fracture opening at θ = 90°.

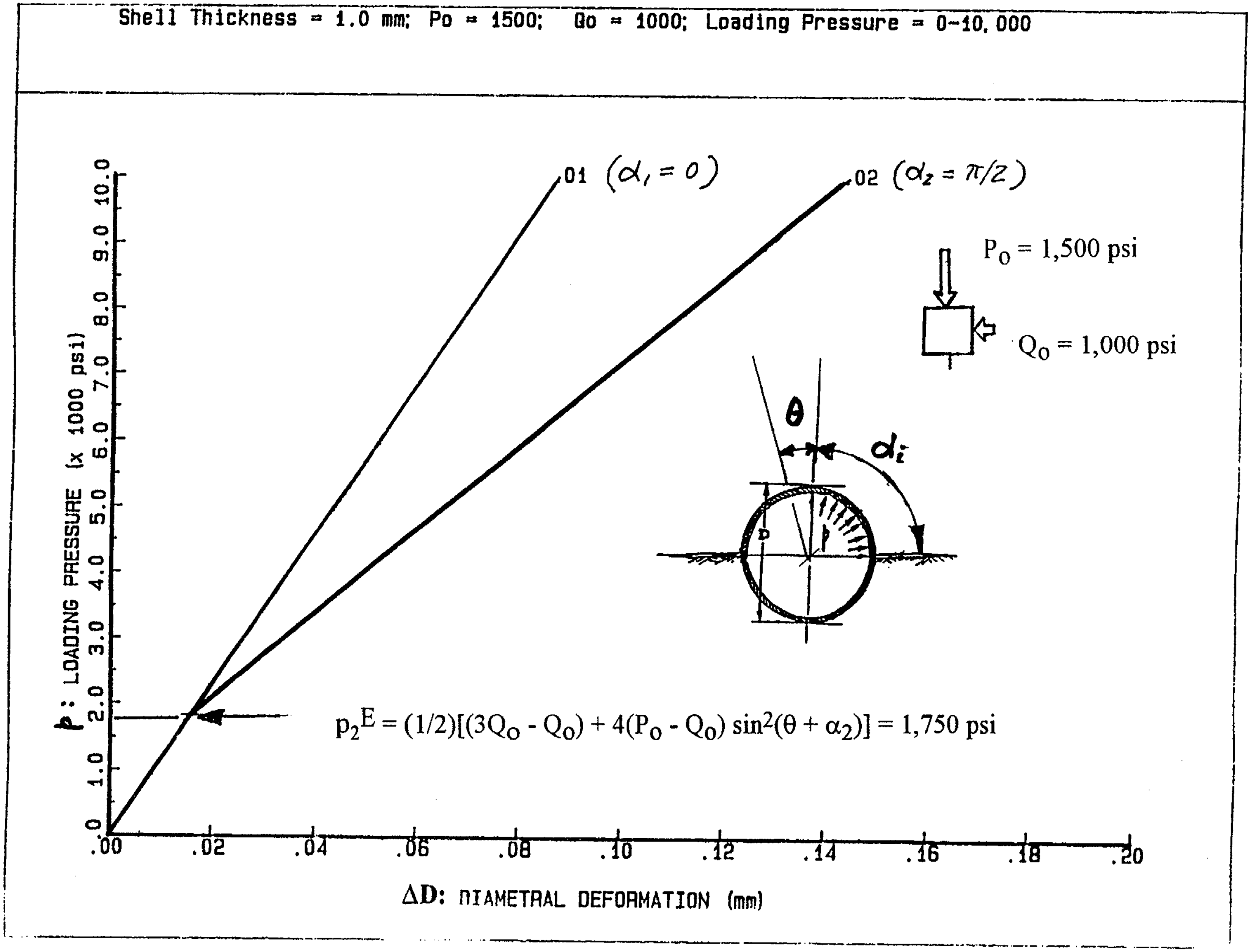
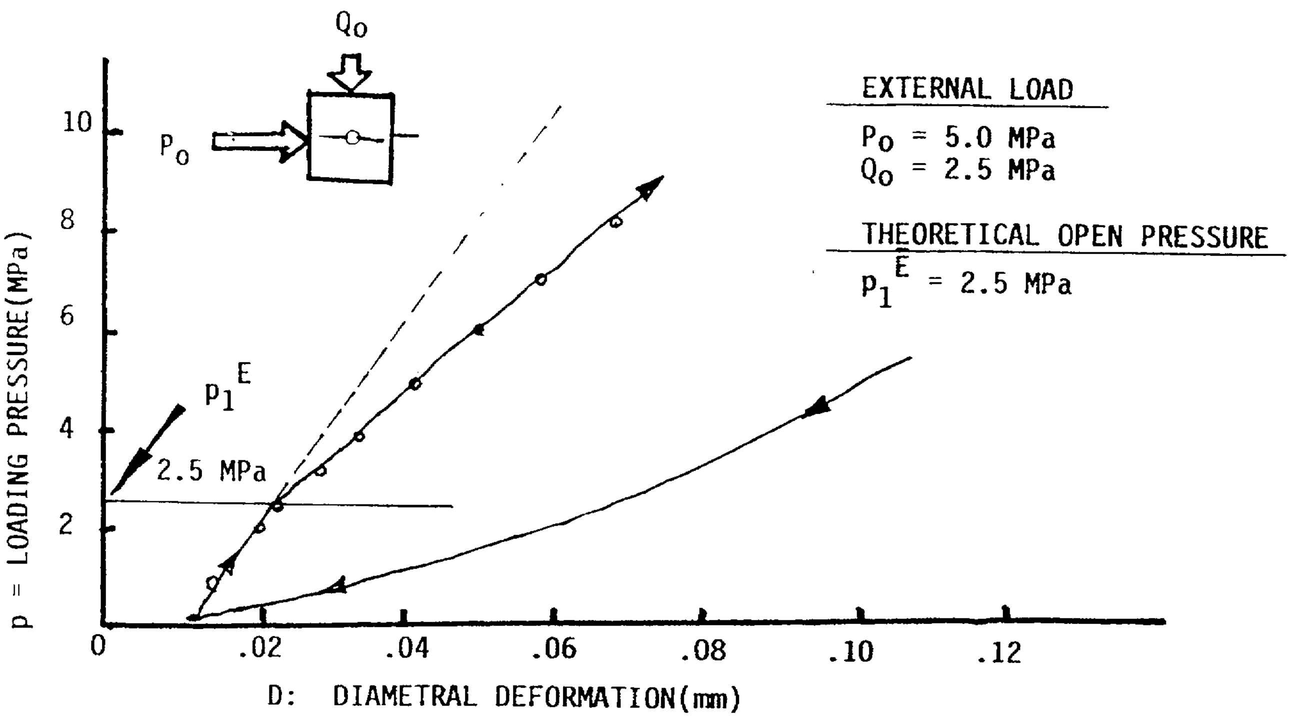


Fig. 13 FEM model of single fracture probe simulating two p-D curves, one in parallel(01) and other in normal(02) to the single fracture plane, demonstrating close agreement between theory and model in accurate determination of stress state.



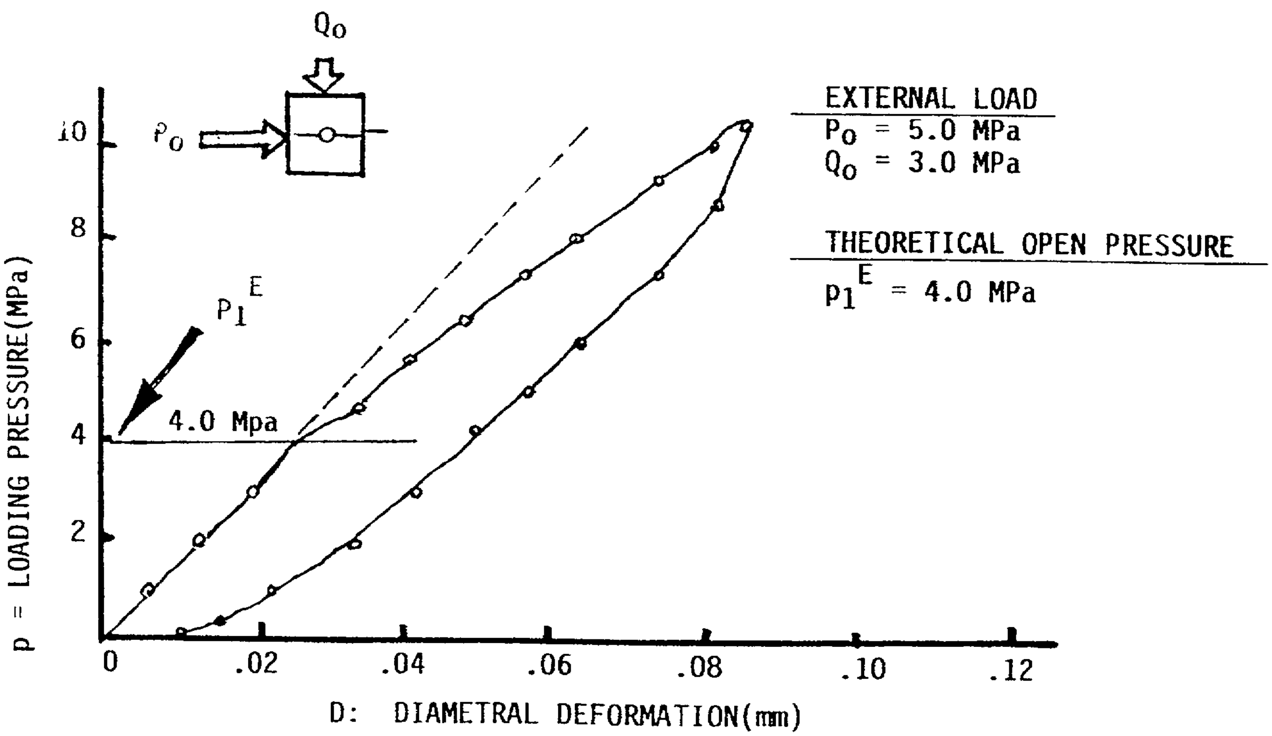


Fig. 14 Laboratory demonstration of detecting reopening presure p_1^E of single fracture made in uniform property concrete test block(1m x 1m x 1m) subject to initial uniform stress loading, demonstrating stress measurement accuracy for single fracture method to be expected at

Δp = applied pressure,
p^B = fracture initiation pressure, and
p^E = pressure required to reopen previously-induced fracture

7 TESTING AT YUCCA MOUNTAIN

Applicability of the dry fracture method to Yucca mountain tuff was tested by using the 76 mm diameter double fracture probe in G-tunnel in the Nevada Test Site in November 1988 (Serata Geomechanics, Inc., 1988). An existing horizontal borehole (IBH 1) was used in the testing.

Results of the raw data observed on the computer screen during the measurements are given in Figs 15 and 16. They represent typical p-D curves in four angular directions of the borehole. Fig. 15 shows the condition of the non-welded tuff at 5 ft from the face of the test entry, disclosing non-recoverable deformation. The deformation was different for different angular direction indicating the high non-uniformity of the ground. This behavior is more pronounced at depth of 23.1 ft, as shown in Fig. 16.

The stresses and their orientation for the two measurement positions are given in the figures. Only slight stress increase is found at the greater depth indicating the stress distribution around the opening is highly distorted from the ideal elastic condition due to the highly non-elastic nature of the ground. This assessment of the ground is supported by the visual observation of the extensive creep sloughing of the exposed ground all around the entries of the tunnel.

The test results were critically reviewed by the rock mechanics group of Sandia National Laboratories who pointed out the shortcoming of the double fracture method. They also pointed out a great potential for the use in the future project. This potential is now being achieved by the single fracture probe, as demonstrated by the close agreement among the theoretical analysis, model simulation and laboratory testing.

8 DISCUSSIONS AND CONCLUSIONS

It can be concluded from the scrutiny of double fracture method that the computerized borehole probe based on the double fracture method is suffering the five major problems for effective field applications to general earthwork. The problems are analyzed which results in an innovation of the single fracture method. The effectiveness of the new probe is illustrated by the close agreement among the theoretical analysis, FE simulation and laboratory experiment. The solutions to the problems are discussed as follows.

8.1 Effect of Pre-Existing Fractures

The distinctive effect to the dry fracture method of stress measurement from pre-

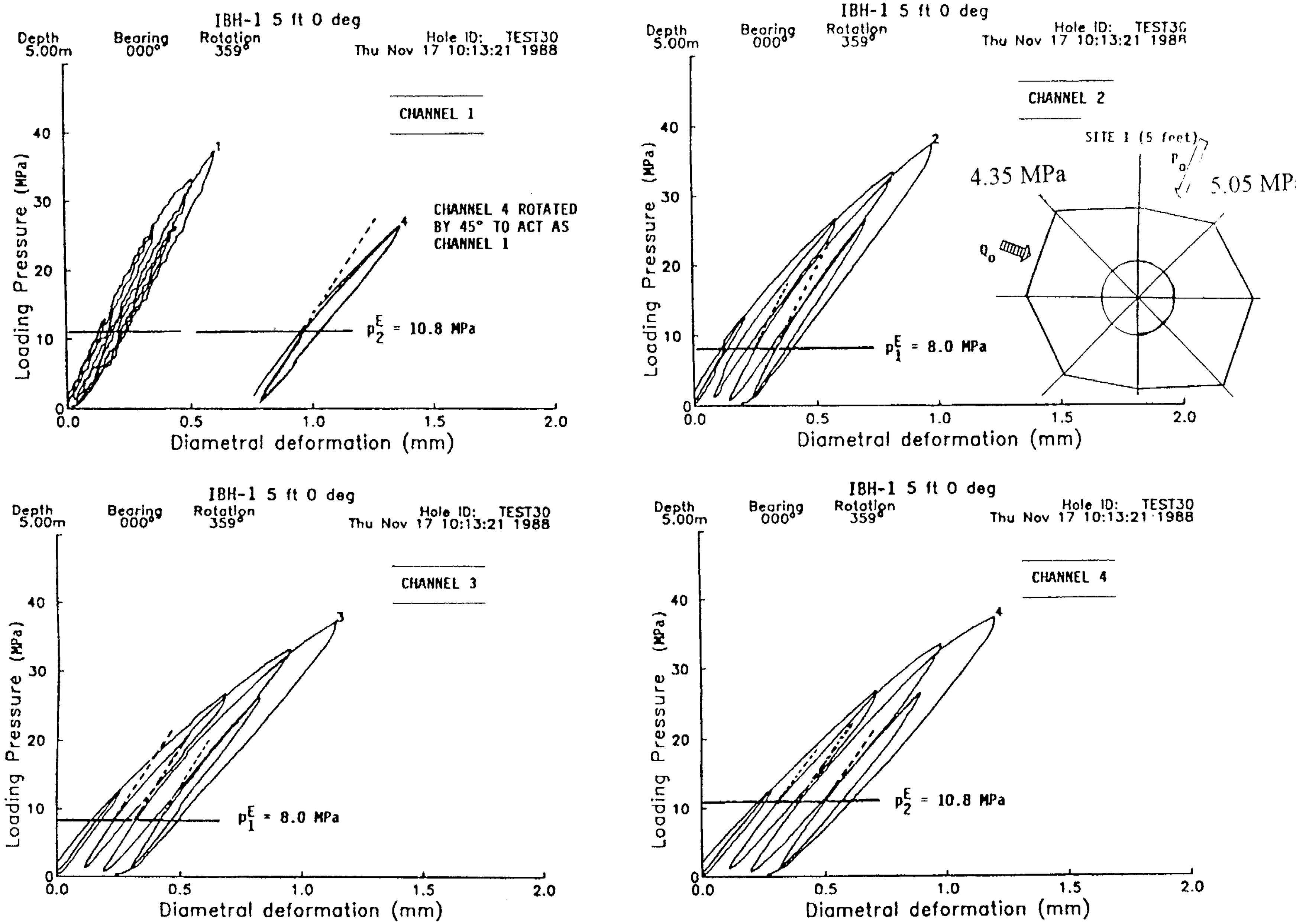


Fig. 15 Set of four p-D curves monitored in computer screen during measurement at 5 ft. in test hole IBH-1 in G-Tunnel, Nevada Test Site instantaneously disclosing complexity of stress state and material properties of the tuff formation.

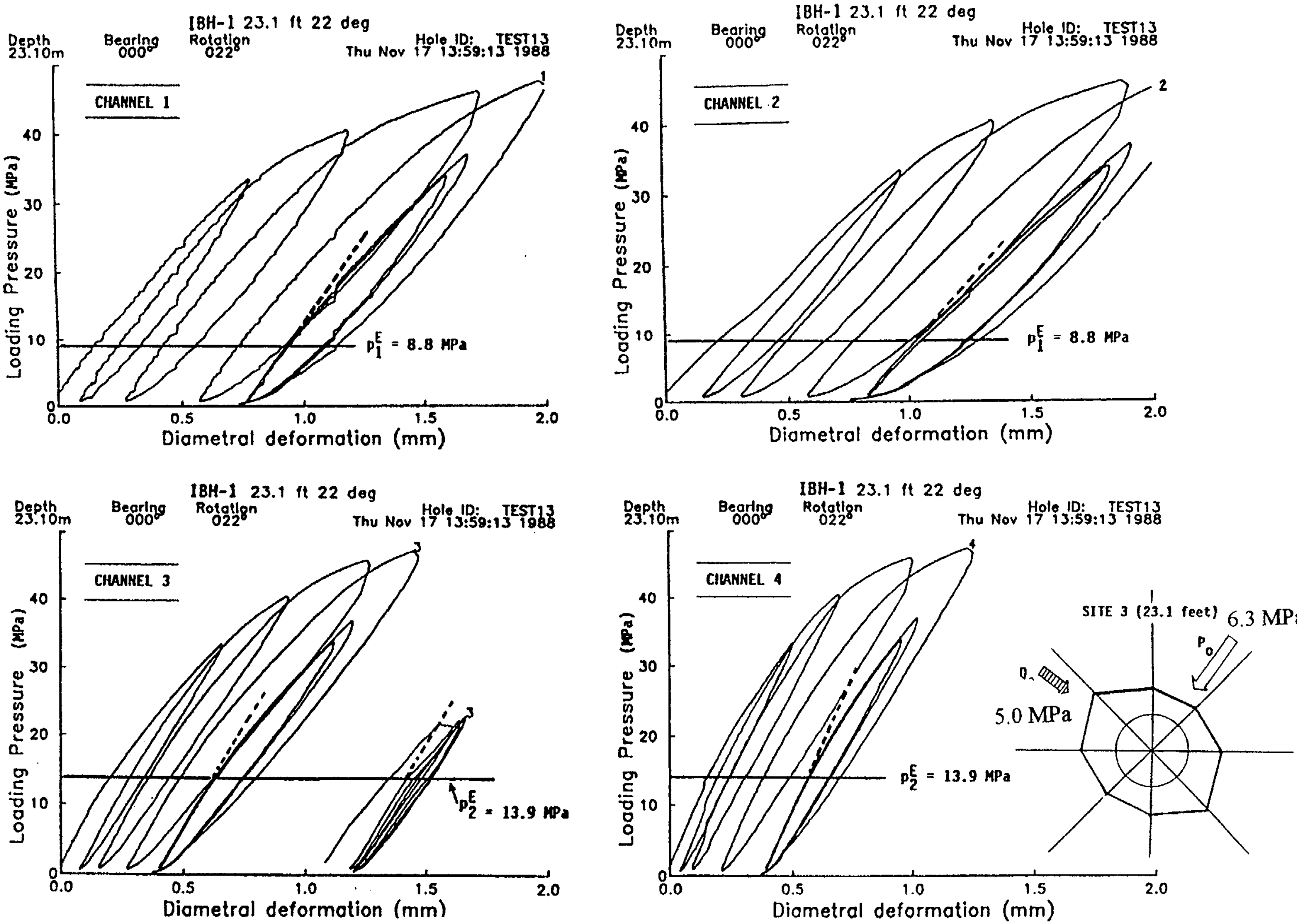


Fig. 16 Set of four p-D curves monitored in computer screen during measurement at 23,1 ft. in test hoel IBH-1 in G-Tunnel, Nevada Test Site instantaneously disclosing complesity stress state and material properties of the tuff formation.

existing fractures can be eliminated by the flexible friction shells of single fracture probe resulting in substantial improvement of the measurement accuracy.

8.2 Inhomogeneity of Materials

The effect from the inhomogeneity of the ground on the accuracy of the stress measurement is less pronounced with the single fracture probe due to the application of the shells.

8.3 Effect from Stress State

The severe limitation imposed to the double fracture probe due to the maximum to minimum stress ratio is eliminated by the single fracture probe.

8.4 Probe Weight

Design and manufacturing of the probe have advanced significantly through the past 15 years of development work. The weight of the probe is reduced from 60 kg to as small as 3 kg by successfully miniaturizing the mechanical and electronics components. The size reduction reduces the manufacturing cost as well.

8.5 Acceptance of Probe

Cost of measurements has been reduced by an order of magnitude while the accuracy of measurements has been increased. With this improvement, the probe is now being used for a wide range of applications by the Japanese government organizations including the studies for nuclear safety by the Ministry of International Trade and Industry (MITI), and for the underground structures by the Ministry of Construction. The Japanese electric power industry is experimenting for its use in determining long-term safety of dams and underground space, as well as for the earthquake prediction studies.

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STIFFNESS AND STRENGTH PROPERTIES OF NATURAL FRACTURES FROM NORTH RAMP DRILL HOLES[1]

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ABSTRACT: Cores containing natural fractures were obtained from drillholes UE 25 NRG–4 and USW NRG–6 at Yucca Mountain, Nevada. Seven selected fractures were sheared at constant normal stress, either 5 or 10 MPa, in the air-dry condition. Detailed profilometer data were collected from each fracture surface before testing. The tests yielded the normal closure as a function of normal stress, and the shear stress and dilation as a function of shear offset. The constitutive properties resulting from the measurements were: normal stiffness, shear stiffness, shear strength and coefficient of friction, and dilation. Peak friction ranged from 0.89 to 1.11; residual friction ranged from 0.76 to 1.00. The lowest initial dilation angle was found to be 5.29° and the highest was 11.28°. The roughness characteristics of the fracture surfaces agree qualitatively with the simple mathematical model of Brown (1994) derived from fracture data in many other rock types.

1. INTRODUCTION

The Yucca Mountain Site Characterization Project (YMP) of the Office of Civilian Radioactive Waste Management (OCRWM) Program has been assigned the task of determining the suitability of the Yucca Mountain site as a potential repository of high level nuclear waste. Among the concerns being investigated, the characterization of the mechanical properties of the fractures present in the host rock has direct relevance to repository design, and the pre- and post-closure performance assessment. This report summarizes data collected in mechanical properties experiments on natural fractures from drillholes UE 25 NRG–4 and USW NRG–6. This work is in direct support of design and performance assessment of the Exploratory Studies Facility (ESF).

Most rock masses contain natural fractures with little or no visible offset called joints. The presence of these features in the rock mass have important effects on the overall thermal/mechanical and hydromechanical response of the mass. They can increase the compliance, reduce the strength, alter the thermal conductivity and act as pathways for fluid movement. Because of their potential importance in design and performance assessment, the YMP Site Characterization Plan (SCP) calls for the measurement of certain fracture properties: normal stiffness, shear stiffness, cohesion, and coefficient of friction. Application of a cycle of normal stress to a pre-selected value, then holding normal stress constant while shearing the joint at a constant slip rate will give all the specified data, provided that shear and normal displacements have been recorded.

In this report we give shear stiffness; normal stiffness; shear strength as a function of normal stress, from which the coefficient of friction and cohesion is derived; and fracture dilation. In anticipation of needs brought out by further developments in theoretical models relating surface topography and joint hydromechanical and thermomechanical response, we also collected detailed topographic data for each joint.

[1]This work was performed under the auspices of the U.S. Department of Energy, Office of Civilian Radioactive Waste Management, Yucca Mountain Site Characterization Project, under contract DE-AC04-94AL85000.

2. EXPERIMENTAL TECHNIQUE

The results reported here were obtained in rotary shear. This technique has been used for at least twenty years and has certain advantages over other configurations used for shear testing. Details on implementation of this test technique along with discussions of its relative advantages and disadvantages and many important results can be found in a number of papers (Biegel *et al.*, 1992; Christensen *et al.*, 1974; Kutter, 1974; Olsson, 1987, 1988; Olsson and Brown, 1993; Xu and Frietas, 1988; Tullis and Weeks, 1986; Weeks and Tullis, 1985; Yoshioka and Scholz, 1989).

In this type of experiment, the sample is composed of two, short, hollow tubes of rock that are pressed together under controlled load, and then torque is applied to cause sliding on the interface (Figure 1). Further details may be obtained from (Olsson, 1987).

For this study, samples of the fracture surfaces were obtained by taking subcores perpendicular to the fracture. A second core of about half the diameter of the first was drilled coaxial with the first resulting in a short hollow cylinder divided into two by the fracture. Nominal outer diameters and inner diameters were, respectively, 60 and 29 mm. End-pieces were potted into metal sample holders with gypsum cement (Figure 1); the metal sample holders were then bolted into the load-frame.

Torque and axial load were used to compute, respectively, the shear stress and normal stress. The shear stress (τ, MPa) during sliding is given by

$$\tau = \frac{3T}{2\pi(R_o^3 - R_i^3)} \tag{1}$$

where T is the torque (NM), and R_o and R_i are, respectively the outer and inner diameters (m).

The parameters identified in the SCP, shear strength, angle of friction, cohesion, and shear and normal stiffnesses plus the dilation angle, are all obtained from the same basic normal-load/shear-load test. In the first phase of a test, the normal stress is increased from zero to the pre-selected normal stress. An example of the resulting closure curves is shown in Figure 2. The joint is then unloaded and then reloaded to the previous maximum value of normal stress. The data reported here are for fractures that are fitted together, i.e., mated. The instantaneous tangent of the curve is plotted on the right and is called the normal stiffness. As indicated in Figure 1, the gauge length for the normal closure includes some of the intact rock. Therefore, the stiffness is a lower bound on the true joint stiffness; however, the inclusion of the intact rock introduces an error of less than 3%.

Shear displacement is then applied resulting in a curve similar to the one shown in the upper part of Figure 3. All shear stress-slip and dilation-slip curves are traversed in the clockwise sense. The bottom left of Figure 2 shows shear stress plotted against slip only up to peak stress. The shear stress is plotted against the instantaneous tangent, i.e., the shear stiffness, on the lower right. The full data set (Figure 3) shows that the shear stress descends to a residual value. Also, in Figure 3, the dilation is plotted against the slip.

When the peak stress for each test is plotted against the normal stress for that test, there usually results a nearly straight line with slope $\mu = \tan\phi$, where ϕ is the angle of friction and μ is the coefficient of friction. The intercept for fractures of this type are usually close to zero, called c.

To simplify nomenclature, a short experiment identification number was used; the correlation between experiment number and sample number is given in Table 1.

Experiment Identifier	Sample Number	σ MPa	μ_{peak}	μ_{resid}	i_{peak} degrees	Slip Rate 10^{-3} mm/s
YM1	NRG-6-297.4-297.7-SNL	5	1.03	1.00	7.15	1
YM2	NRG-4-608.7-609.2-SNL	5	-	-	-	2
YM2A	NRG-4-608.7-609.2-SNL	5	0.89	0.83	7.39	4
YM3	NRG-6-27.6-28.1-SNL	5	1.03	0.99	10.22	4
YM4	NRG-4-537.8-538.2-SNL	5	1.10	0.96	5.72	4
YM5	NRG-6-424.0-424.5-SNL	5	1.06	0.80	5.23	4
YM6	NRG-6-485.9-486.3-SNL	10	1.02	0.78	8.95	4
YM7	NRG-6-401.5-401.9-SNL	10	1.11	0.78	7.03	4

σ normal stress; μ_{peak} peak friction; μ_{resid} residual friction; i_{peak} dilation angle at peak stress

Table 1: Experiment identifier, sample number and strength data for each experiment.

3. PROFILOMETER DATA

In anticipation of needs brought out by further developments in theoretical models relating surface topography to joint hydromechanical and thermomechanical response, we have made detailed measurements of the surface topography for most samples. The methods of measurement and data analysis are the same as those described in Brown (1994).

The topography of each fracture surface was measured with a non-contacting laser profilometer. This instrument consists of a precision three-axis positioning system which moves a laser distance measurement probe over the surface, recording surface height. The probe is moved along parallel lines to record a series of one-dimensional surface profiles.

Each sample consists of two hollow tubes with the joint surfaces exposed on the ends. Three surface profiles were taken in a circular path around each of the two surfaces comprising a sample. The three profiles were separated from each other along the sample radius by 5 mm, with the center profile at a radius of 21.5 mm. The topographic heights were sampled at an increment of every 0.06 degrees for a total of 360 degrees. The spacing between data points on the center profile is therefore 0.0225 mm.

The samples were mounted in the profilometer in such a way as to allow matched pairs of profiles from each surface to be closely refitted in the subsequent analysis. Several preliminary data processing steps were done once the profiles are taken. First, each pair of profiles from the two halves of the fracture were fitted together and the standard deviation of the "composite topography" (negative of the aperture distribution or the local distance between the surfaces) was computed. Then the two profiles were repeatedly shifted relative to one another and the composite topography was recomputed. The best match is obtained when the standard deviation of the composite topography reaches a minimum. Then the mean level and the linear slope were removed from both profiles and the composite topography. Following the surface topography model presented by Brown (1994), several surface roughness statistics were calculated and are presented in Table 2. An example plot of surface profile data is shown in Figure 4.

Sample Number	D_{lin}	D_{log}	λ_c^1 mm	λ_c^2 mm	σ_p mm	σ_a mm
NRG-6-297.4-297.7-SNL			– no data –			
NRG-4-608.7-609.2-SNL	1.45	1.35	9.07	1.44	0.30	0.27
NRG-6-27.6-28.1-SNL	1.42	1.25	3.26	1.19	0.4	0.17
NRG-4-537.8-538.2-SNL	1.43	1.25	5.0	1.83	0.33	0.28
NRG-6-424.0-424.5-SNL	1.51	1.40	10.3	0.76	0.48	0.51
NRG-6-485.9-486.3-SNL	1.45	1.19	2.38	1.33	0.55	0.26
NRG-6-401.5-401.9-SNL	1.47	1.38	7.39	1.70	0.41	0.34

D_{lin} fractal dimension with uniform weights in least squares fit
D_{log} fractal dimension with logarithmic weights in least squares fit
λ_c^1, λ_c^2 estimates of mismatch length scale
σ_p standard deviation of surface for a 23 mm profile segment
σ_a standard deviation of aperture for a 23 mm profile segment

Table 2: Profilometer data for each sample. Refer to Brown (1994) for a further explanation of each roughness parameter.

The roughness characteristics of the fracture surfaces agree qualitatively with the simple mathematical model of Brown (1994) derived from fracture data in many other rock types. The surfaces themselves have power-law power spectral density functions indicating self-affine fractal geometry and the two surfaces comprising each fracture are closely matched at length scales above a few millimeters. The only discrepancy from the model is that the 6 NRG samples studied here have probability density functions for the aperture which are not as close to a perfect Gaussian shape as the samples described by Brown (1994).

4. RESULTS OF COMPRESSION AND SHEAR EXPERIMENTS

Each sample was subjected to a normal compression cycle in the mated condition before shearing. Tests YM1 through YM5 were compressed to 5 MPa normal stress; tests YM6 and YM7 were compressed to 10 MPa. All curves show hysteresis and permanent closure after the first cycle. Only the first loading in compression is shown in Figure 2. The fractures always had an increase in stiffness when comparing the first loading to the second loading up to the normal stress required for the shear test. The normal stiffness is the rate of change of the normal stress with respect to normal displacement. As mentioned earlier, the elastic contribution by the intact rock included in the gage length has not been accounted for, but the error introduced by this is less than 3%.

Note that two sequential experiments, YM2 and YM2A, are indicated in Table 1 for the same sample, NRG-4-608.7-609.2-SNL. This is a result of the first test being terminated by an unanticipated loading system shut-down; the applied loads dropped instantaneously to zero causing no damage to the sample. Therefore, the second experiment, YM2A, was carried out on a joint that had previously undergone 0.7 mm of slip at 5 MPa, showing how some previous sliding affects the surface mechanical properties.

All sample were sheared at constant normal stress at room temperature and in the air-dry condition (prevailing laboratory temperature of about 20°C and relative humidity of approximately 20%). The shear stress/slip curves are all characterized by peak stress followed by a more or less gradual descent to a residual value. All dilation curves start at a negative value because of the finite normal stress application. Furthermore, they all show an initial decrease to a local minimum with the onset of slip. Then the dilation rate becomes positive and nearly constant for the remainder of the first loading phase. The reported values of dilation angle in Table 1 are computed at the slip corresponding to the peak stress in each experiment.

Figure 5 presents the peak shear stress (τ_{peak}) data plotted versus the normal stress (σ_{normal}) data for each of the seven experiments. The linear best-fit to the data is

$$\tau_{peak} = 1.11\,\sigma_{normal} - 0.430\,MPa.$$

5. SUMMARY

Cores containing natural fractures were obtained from drillholes NRG–4 and NRG–6 at Yucca Mountain, Nevada. Seven selected fractures were sheared at constant normal stress, either 5 or 10 MPa, in the air-dry condition. Detailed profilometer data was collected from each fracture surface before testing. The tests yielded the normal closure as a function of normal stress, and the shear stress and dilation as a function of shear offset. The constitutive properties resulting from the measurements were: normal stiffness, shear stiffness, shear strength and coefficient of friction, and dilation. Peak friction ranged from 0.89 to 1.11; residual friction ranged from 0.76 to 1.00. The lowest initial dilation angle was found to be 5.29° and the highest was 11.28°. The roughness characteristics of the fracture surfaces agree qualitatively with the simple mathematical model of Brown (1994) derived from fracture data in many other rock types.

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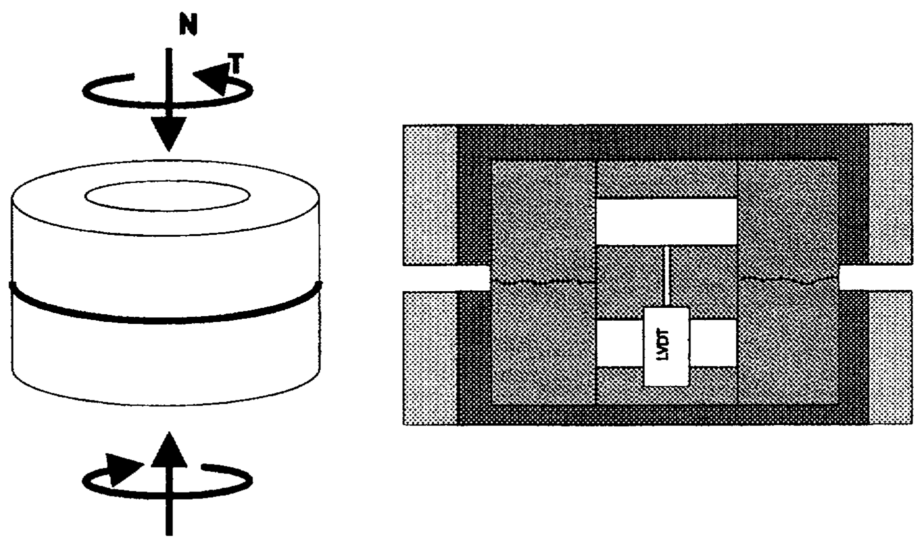

Figure 1: Sample configuration for rotary shear experiments on joints. Left is a perspective view showing applied forces. Right is a cross-section; light grey is aluminum holder, dark grey is potting compound, medium grey is rock sample, white is LVDT body and core, and mounting supports. Setup also included LVDT's mounted diametrally to outside of aluminum ring, measuring across joint. Not to scale.

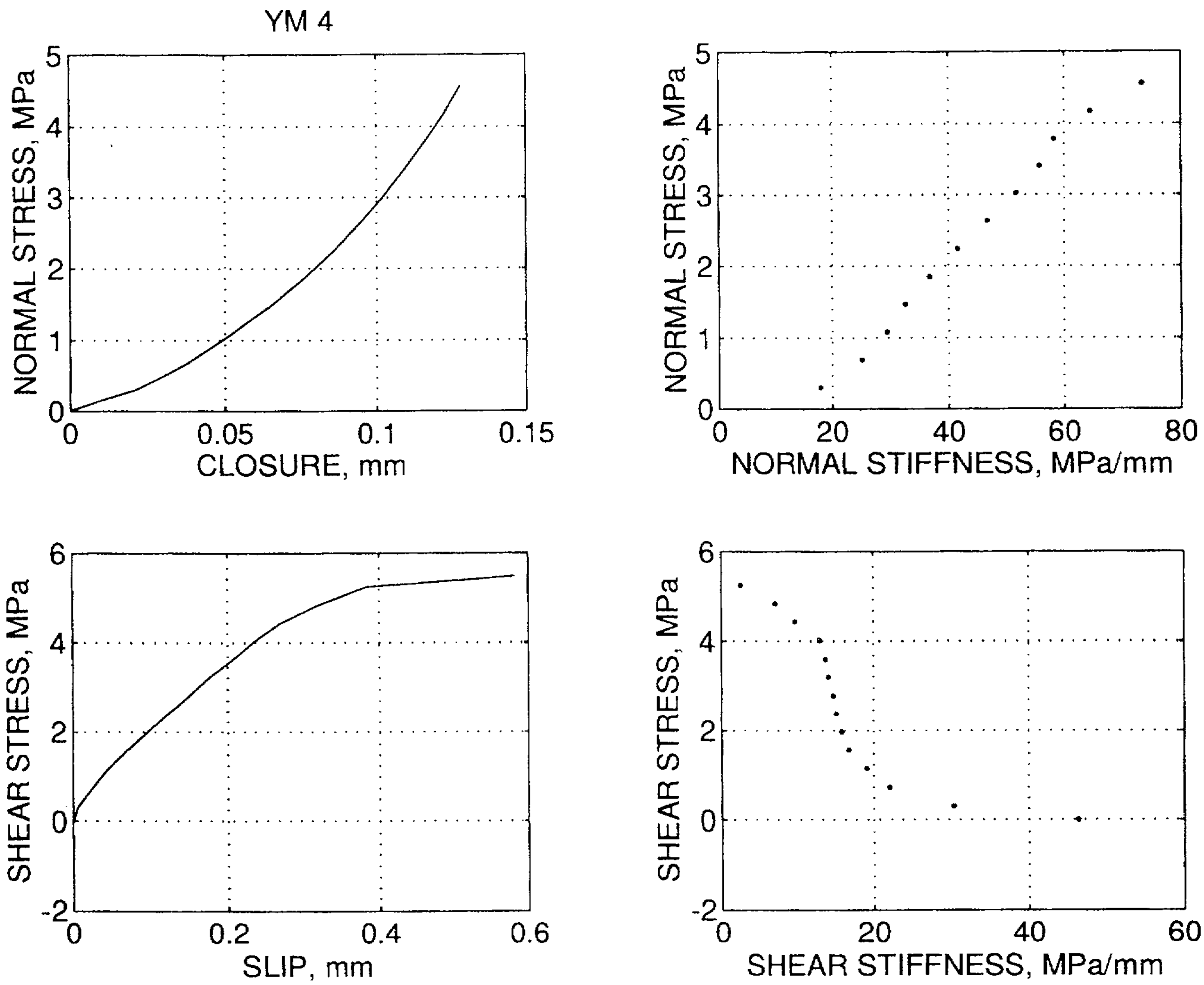

Figure 2: Shear and normal stiffness data for YM 4.

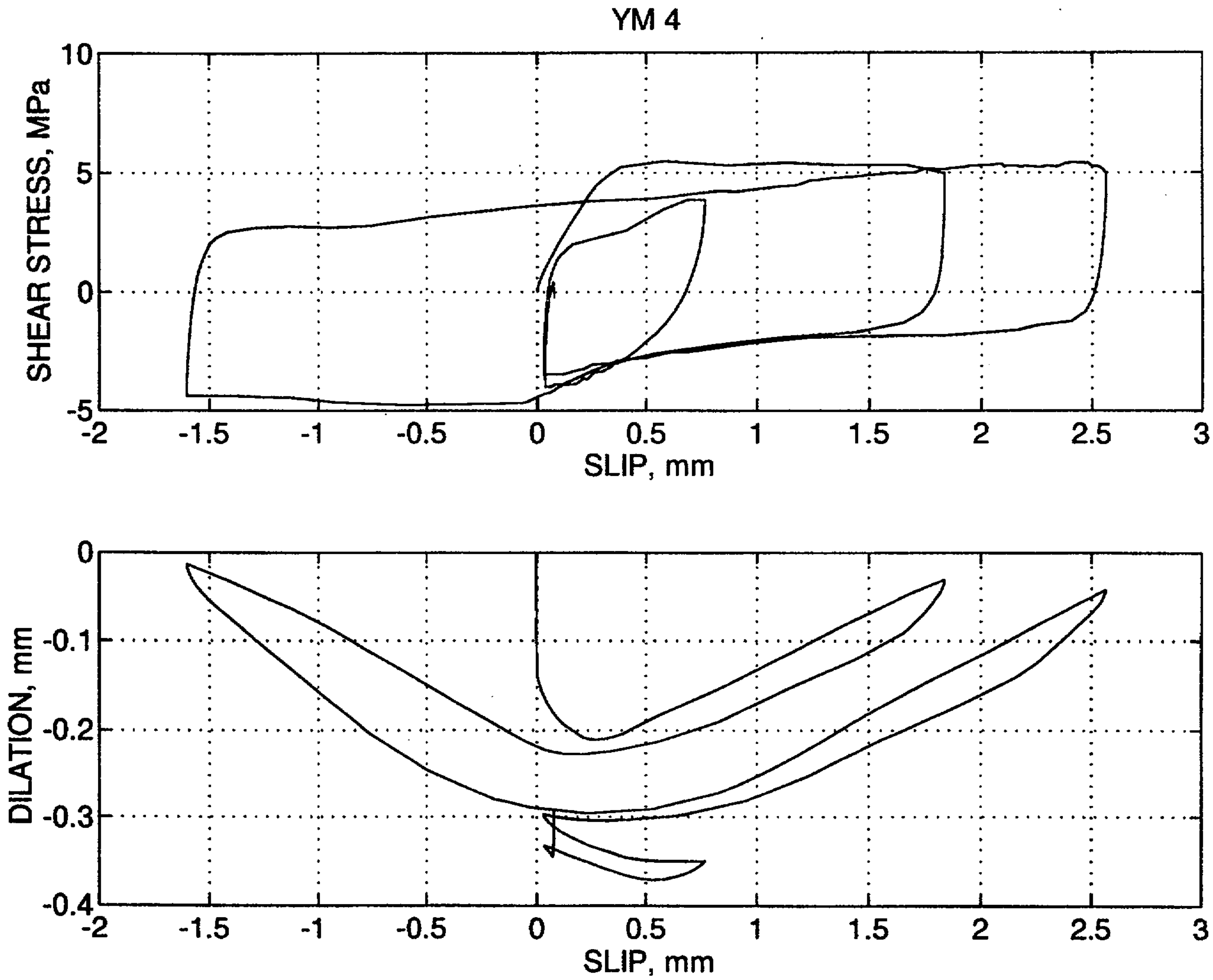

Figure 3: Shear strength and dilation data for YM 4.

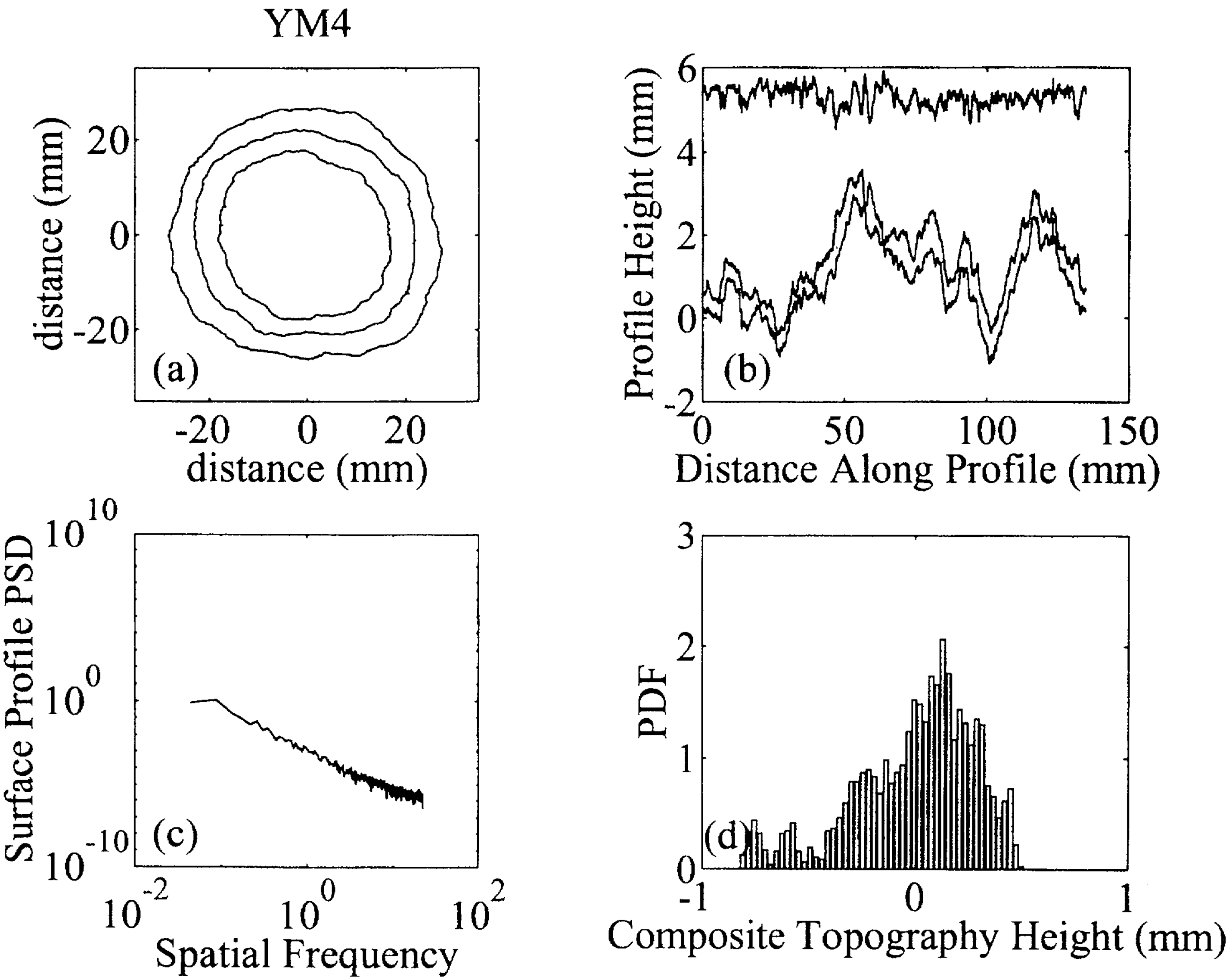

Figure 4: Profilometer data for YM 4. (**a**) Three circular profiles on one fracture surface. The average (circular) radius of each profile represents the position of the profile in millimeters from the center of the sample. The radial variation from this circle represents the topographic height in millimeters. Variations in the topography of the surfaces around the sample surface can be seen in this representation. (**b**) The center profiles on each surface are unwrapped and matched in the two bottom curves. The "composite topography" (negative of the aperture) is plotted in the upper curve. Local maxima of the composite topography are potential contact spots. (**c**) Average power spectral density function for all profiles on both fracture surfaces. The nearly linear form of this function on a log-log plot indicates that this surface is a self- affine fractal where the slope is proportional to the fractal dimension. (**d**) Probability density function for heights on the composite topography averaged over all three pairs of matched profiles.

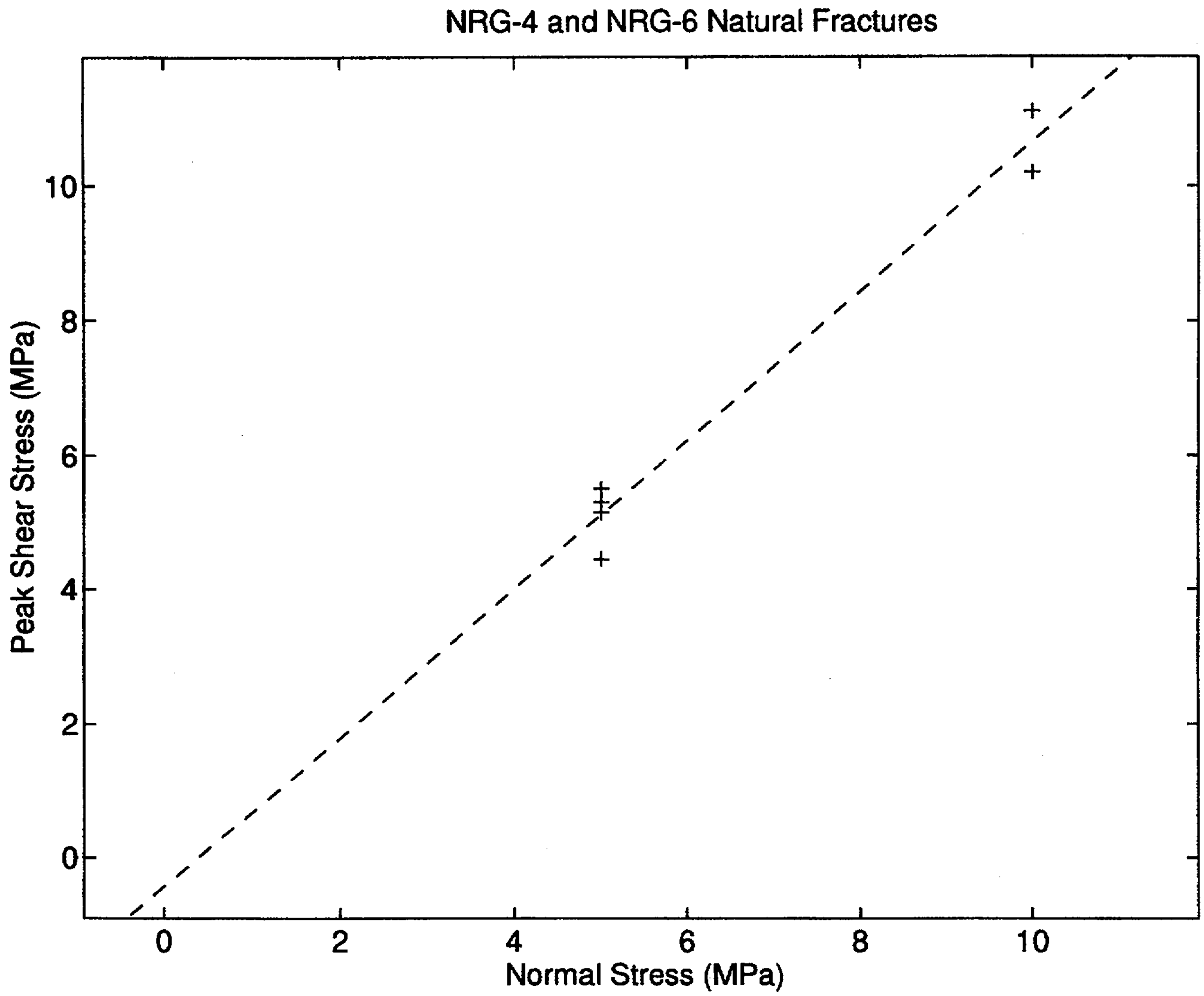

Figure 5: Peak shear stress versus normal stress data for all seven experiments.

MECHANICAL AND BULK PROPERTIES OF INTACT ROCK COLLECTED IN THE LABORATORY IN SUPPORT OF THE YUCCA MOUNTAIN SITE CHARACTERIZATION PROJECT[1]

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ABSTRACT: A comprehensive laboratory investigation is determining the mechanical properties of tuffs for the Yucca Mountain Site Characterization Project (YMP). Most recently, experiments have been performed on tuff samples from a series of drill holes along the planned alignment of the Exploratory Study Facilities (ESF) north ramp. Unconfined compression and indirect tension experiments were performed and the results are being analyzed with the help of bulk property information. The results on samples from eight of the drill holes are presented. In general, the properties vary widely, but are highly dependent on the sample porosity. The developed relationships between mechanical properties and porosity are powerful tools in the effort to model the rock mass response of Yucca Mountain to the emplacement of the potential high-level radioactive waste repository.

1. INTRODUCTION

The applicability of laboratory data for engineering and modeling purposes is a prime concern for the design and licensing of a potential nuclear waste repository at Yucca Mountain, Nevada. One specific example involves mechanical property results obtained from experiments in a rock mechanics laboratory. In order for these data to be realistic, they must be scaled to *in situ* size and conditions. The standard approach to this problem is for the mechanical behavior of the intact (i.e., non-fractured) rock and fractured rock to be evaluated in separate experiments, with the results then utilized in constitutive models to predict rock mass response to certain sets of environmental conditions. In order to validate these models, the predicted results are compared to data obtained in large scale experiments that are performed *in situ.*

The study described here involves the testing of intact tuff samples in support of ramp and drift design for the ESF (Exploratory Studies Facility). The experiments have been performed on samples taken from a series of core holes designated as NRG (North Ramp Geology). A large number of data have been collected on bulk properties (average grain density, dry and saturated bulk densities, and porosity), elastic properties (dynamic and static Young's modulus and Poisson's ratio), and strength properties (unconfined and confined compression and indirect tension). Samples from eight of the drill holes were tested and the results are presented. The data exhibit the large

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scatter expected with tuff, a naturally inhomogeneous material. However, the data are internally consistent, with the mechanical properties being highly dependent on porosity, which varies from less than 0.10 to more than 0.50 (volume fraction). The functional forms of the property versus porosity trends are very similar to those observed in earlier studies on smaller Yucca Mountain tuff samples. The differences in the fitting parameters can be explained in terms of the sample size difference and, possibly, the clay content. These relationships between the mechanical properties and porosity are important because much more porosity information is gathered over the entire mountain, and these values will be used to confidently back-calculate the mechanical properties without having to perform additional (and more expensive) mechanical property experiments.

2. WORK

Raw cores were collected from each of the NRG drill holes, followed by the selection of appropriate samples of each rock unit (lithologic and thermal/mechanical stratigraphies) for the determination of mechanical and bulk properties. Each core piece was examined and a decision made on the appropriate uses for the available material (e.g., an unconfined compression sample, a Brazilian or indirect tension specimen, or both experiment types). A specimen from every core piece was taken for an average grain density measurement. To date, only larger diameter core with relatively homogeneous tuff has been used for testing the effect of pressure on moduli and strength. Each mechanical sample was then machined to the appropriate dimensions in preparation for testing. Following machining, the test samples were dried and then saturated, for the determination of the bulk densities and to prepare for testing under nominally saturated conditions.

A macroscopic description of all samples was performed in hand-specimen detail; however, the unconfined compression samples were also characterized by taking a CT (computerized tomography) X-ray scan of one slice through each specimen. In addition, the compressional (P-) and shear (S-) wave velocities were measured on the unconfined compression samples at both dry and saturated conditions. These techniques provide an analysis of the sample interior prior to testing and aid the post-test analysis of mechanical property results (Price, Martin, and Boyd, 1993).

The unconfined compression experiments were performed at the following standard set of conditions for baseline testing: a saturated sample with a diameter of approximately 50 mm and a length-to-diameter ratio of 2:1 is tested under room temperature, room pressure, and drained conditions at a constant axial strain rate of $10^{-5}s^{-1}$. The triaxial experiments were performed at pressures of 5 and 10 MPa, with the other conditions being similar to the unconfined experiments. The Brazilian experiments were performed using standard techniques on saturated samples.

As mentioned above, the collection of bulk properties data is very important in the interpretation of the mechanical data and will support YMP modeling efforts. As has been shown in earlier studies of tuff from Yucca Mountain, the elastic and strength properties of the tuffs are highly dependent on the material porosity (Price, 1983; Price and Bauer, 1985). For example, when plotted versus porosity, the Young's modulus data are relatively linear on a semi-log plot and the unconfined strengths are linear on a log-log plot (Price, 1983). Price and Bauer (1985) refined these fits by adding the volume of clay content for the determination of the sample's "functional porosity". Work has been initiated to measure the clay content in at least some of the samples tested in this study. In addition, preliminary analyses of the CT scans have produced promise that with a relatively small effort, the pore distribution information collected in the CT scans can be used to further improve the property versus porosity relationships (Price, Martin, and Boyd, 1993).

3. RESULTS

The Young's modulus data from the unconfined experiments have been analyzed as a function of porosity. The data have a relatively linear trend on a semi-log plot. The trend is approximately parallel to the functional form that was fit to earlier strength-functional porosity data from experiments on smaller samples (25.4 mm in diameter) tested under the same set of conditions (Price and Bauer, 1985). The data collected in this study yield a relationship as follows:

$$E = 74.3\, e^{-8.60\phi},$$

where E is the Young's modulus (GPa) and ϕ is the porosity (volume fraction). Figure 1 contains the new data on larger samples, the best fit to that data (solid line), and the fit to the earlier data on smaller samples (dashed line).

Poisson's ratio data have also been analyzed and there is little correlation with porosity. The Poisson's ratios are widely scattered between 0.0 and 0.5, with a mean value of approximately 0.2 (Figure 2). The lack of correlation with porosity indicates that Poisson's ratio may have a higher dependence on some other physical characteristic. For example, the distribution of the porosity may be a significant factor in the Poisson's ratio of a tuff sample. Future work on characterizing the porosity distribution will evaluate this hypothesis.

A similar comparison has been made between the two sets of data for the ultimate sample strengths from the unconfined experiments. These data have a relatively linear trend on a log-log plot of strength versus porosity. As with the Young's modulus data, the earlier study (Price and Bauer, 1985) produced a relationship approximately parallel to the following functional form determined from the data collected by this study:

$$\sigma_u = 0.95\phi^{-2.14},$$

where σ_u is the ultimate strength (MPa) and ϕ is the porosity (volume fraction). The data and two fits are plotted in Figure 3.

Figure 4 presents a plot of the compressional velocity of the unconfined samples in the dry condition versus the sample porosity. This graph illustrates the strong dependence of this dynamic property on the total sample porosity. Further study of the shear velocities and on the comparison of dynamic and static elastic moduli may be of help in putting some bounds on the seismic profiling work performed in the field. These analyses are continuing and the relationships are refined when appropriate.

Six sets of samples were tested at three confining pressures (0.1, 5 and 10 MPa) to determine the dependence of the elastic and strength properties on pressure. The samples were smaller diameter (about 25.4 mm) than the other unconfined experiments. The smaller-sized samples were necessary to have enough test specimens to isolate the effects of pressure by minimizing the lateral and vertical variability in the material properties. The remainder of the test conditions were the same as described for the unconfined experiments. An example set of the data from one piece of core is presented in Figure 5. Each of these sets of data are reduced to determine the parameters of the Coulomb criterion. Figure 6 presents the two Coulomb parameters for each of the six sets of data plotted against the average porosity of the tested samples.

The unconfined tensile strength results determined in indirect tension (Brazilian) tests have also been found to be distinctly related to porosity. The data collected in this study are best fit by the following equation:

$$T_0 = 20.2\, e^{-8.39\phi},$$

where T_0 is the unconfined tensile strength (MPa) and ϕ is the porosity (volume fraction). The data and fit are presented in Figure 7.

4. DISCUSSION

The difference in the two sets of fits on different-sized samples presented in the previous section are very small or can be explained in terms of the change in sample size and/or the lack of montmorillonite data for the latest set of samples.

The exponent (i.e., the slope in semilog space) of the fit to the new Young's modulus/porosity data is very similar to the slope for the smaller samples, indicating that the effect of porosity changes on mechanical behavior is the same for both sample sizes. In addition, the offset of the two fits is small. This result is not surprising, because no effect of sample size on this property was noted in an earlier study (Price, 1986) designed to investigate the effects of sample size on the mechanical properties of intact tuff.

The variability in Poisson's ratio has been very large in both sets of data. However, all of the data from the two sample sizes have clustered around a value of approximately 0.2, with the property apparently being independent of total porosity. Later study of the porosity distribution may shed some light on the scatter in this property.

The two fits of the unconfined compression strength data are approximately parallel; however, the fit of the larger samples is shifted downward from the earlier fit. This shift is relatively large, but can be explained in terms of the difference in sample size. The previous sample size study (Price, 1986) showed a significant decrease in strength of welded tuff samples by changing the sample diameter from 25.4 mm to 50.8 mm.

There are a few data points on the Young's modulus/porosity plot (Figure 1) and the strength/porosity plot (Figure 3) that fall relatively far below (or far to the left of) the best fits. These offsets tend to occur at the high porosity ends of the data. Generally, the samples of non-welded tuff have a higher probability of containing some montmorillonite. As the mineralogy data become available and the fits are made to functional porosity instead of simply pore volume, the data trend will probably become more distinct.

5. CONCLUSIONS

The mechanical and bulk property data discussed in this document are being collected in a detailed manner using standard laboratory procedures. The resultant data are wide ranging; however, all of the data are within expected bounds and the result of testing reasonably sized samples for characterizing the mechanical properties in the laboratory. The analyses of these data have supported the conclusion from earlier studies that porosity is the dominant factor in determining intact-rock, static and dynamic mechanical properties in the Yucca Mountain tuffs. The functional relationships between most of the mechanical properties and porosity will be valuable in the modeling of the potential repository at Yucca Mountain.

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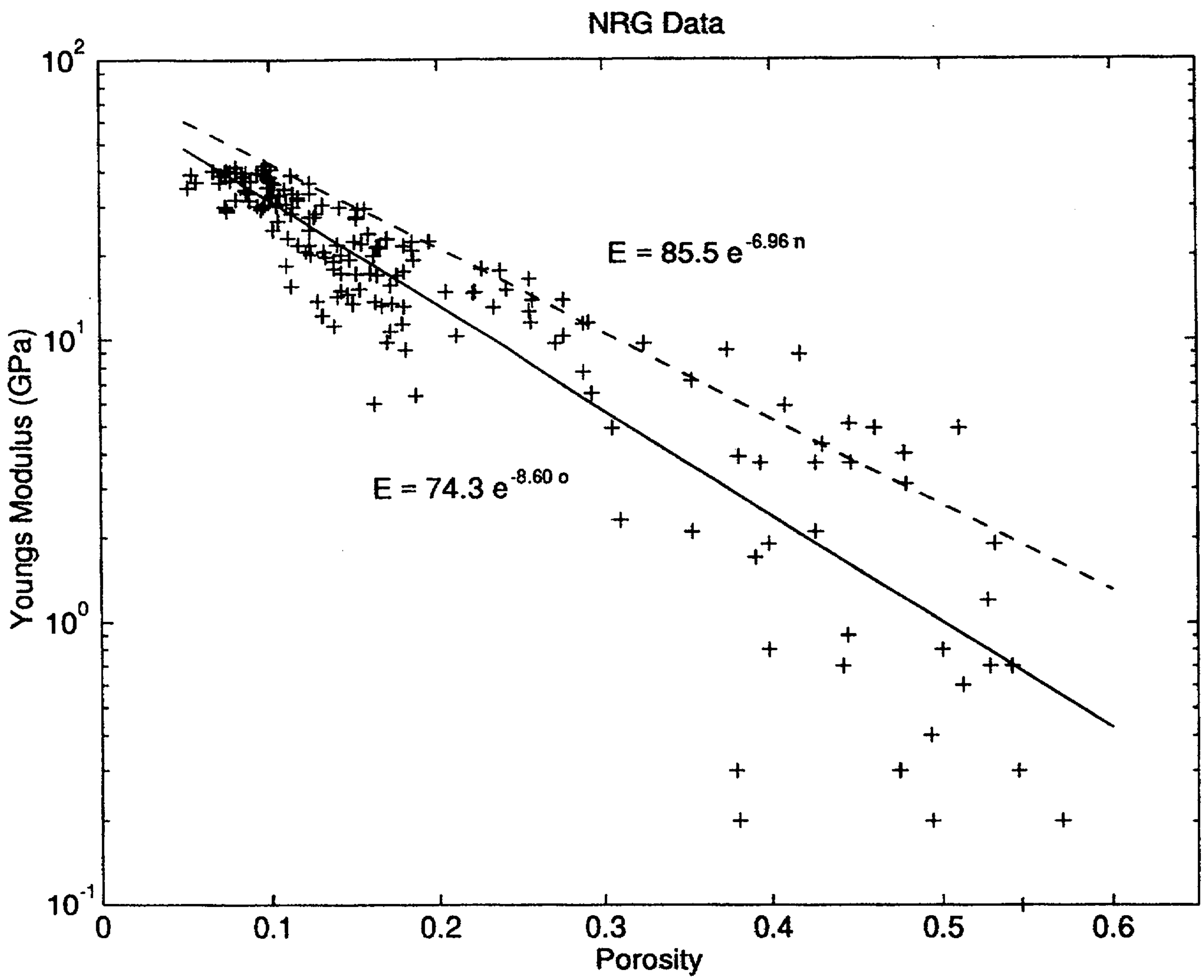

FIGURE 1.

Plot of Young's modulus versus porosity for water-saturated samples of Yucca Mountain tuff with diameters of 50.8 mm and tested under room temperature, room pressure, and drained conditions at a constant axial strain rate of $10^{-5}s^{-1}$. Solid line is a fit ($E = 74.3\,e^{-8.60\phi}$) to these data, and the dashed line is a fit ($E = 85.5\,e^{-6.96n}$) to data from smaller samples (25.4 mm) tested under the same set of conditions, where n is functional porosity.

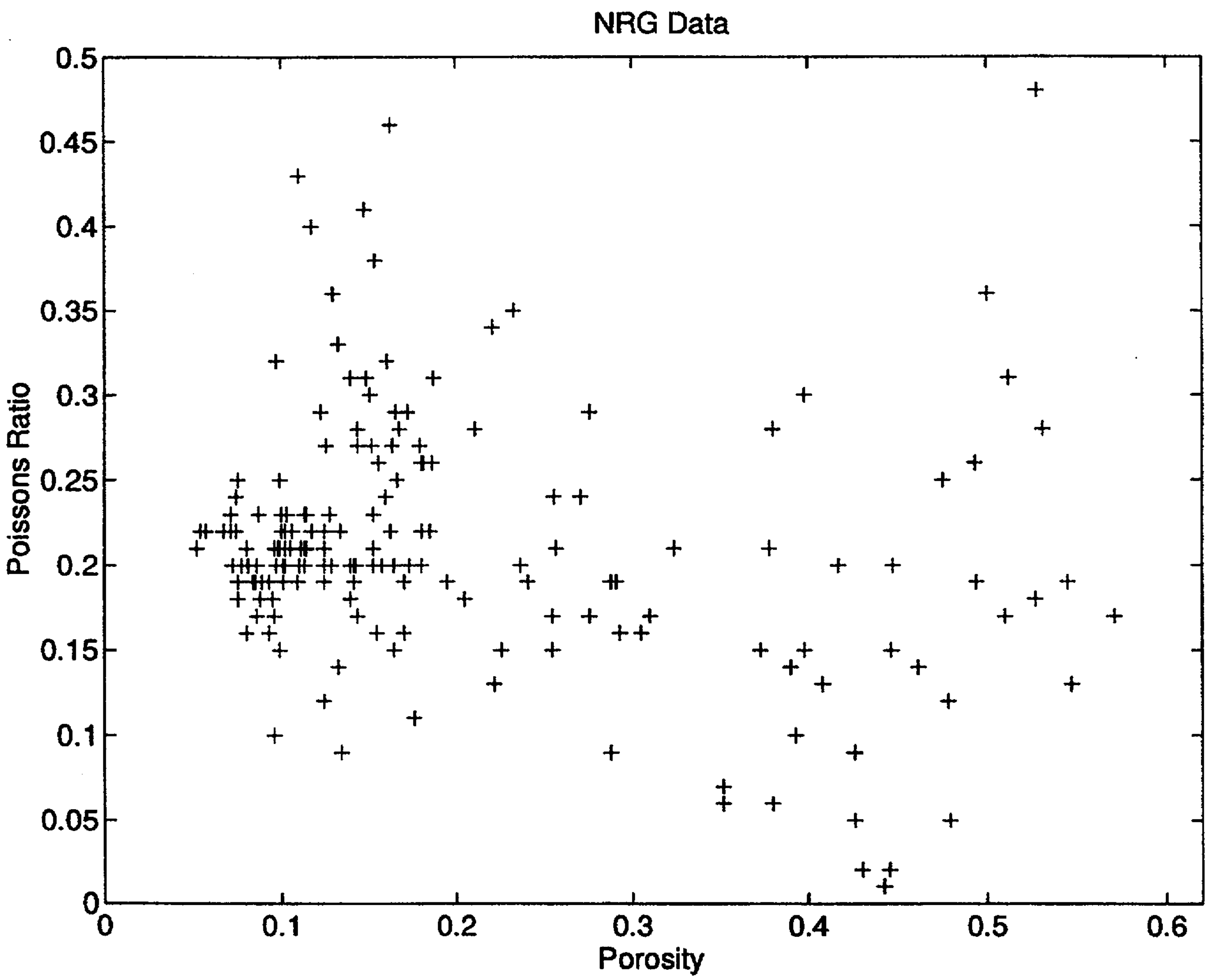

FIGURE 2.

Plot of Poisson's ratio versus porosity for water-saturated samples of Yucca Mountain tuff with diameters of 50.8 mm and tested under room temperature, room pressure, and drained conditions at a constant axial strain rate of $10^{-5}s^{-1}$.

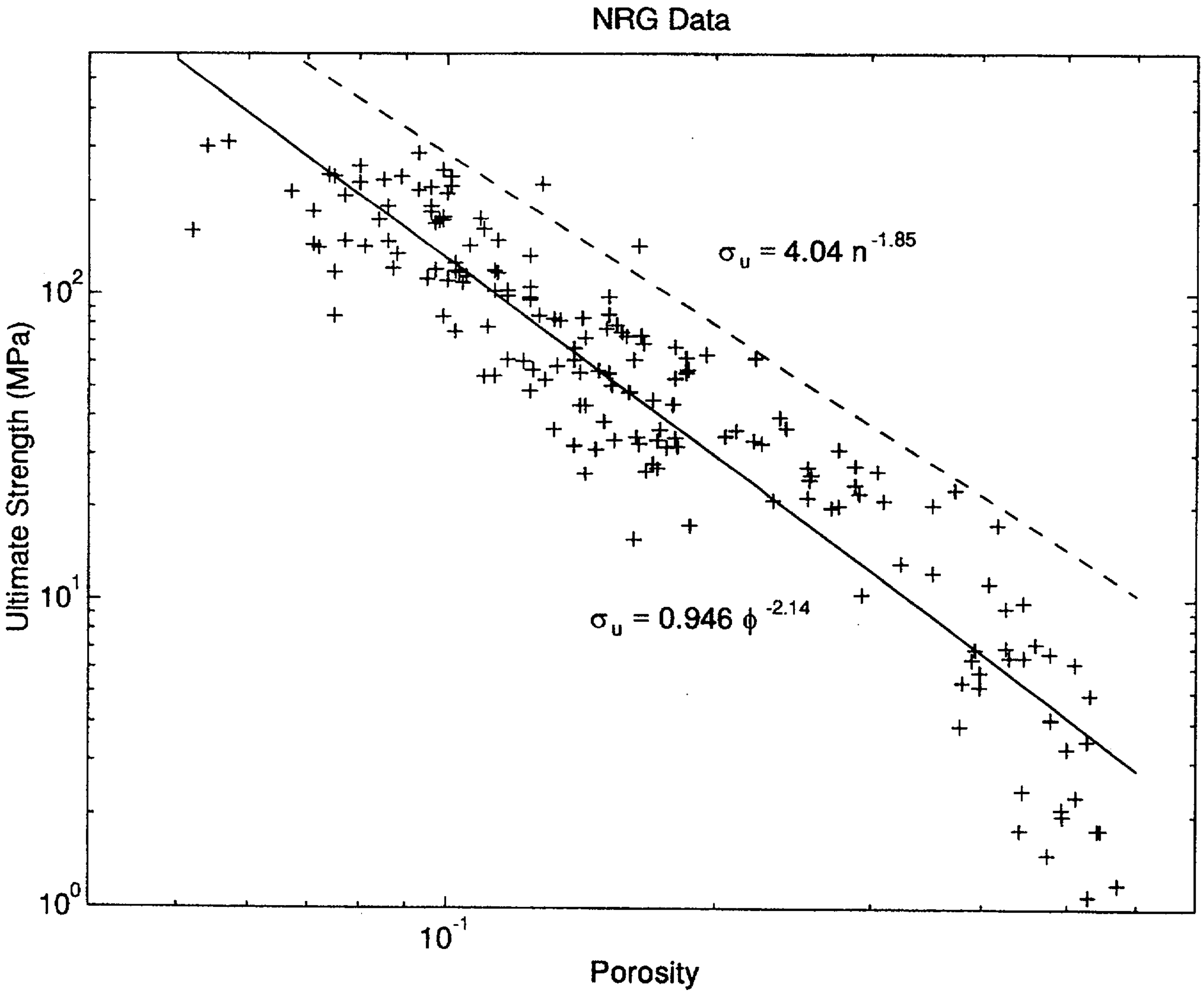

FIGURE 3.

Plot of ultimate strength versus porosity for water-saturated samples of Yucca Mountain tuff with diameters of 50.8 mm and tested under room temperature, room pressure, and drained conditions at a constant axial strain rate of $10^{-5}s^{-1}$. Solid line is a fit ($\sigma_u = 0.946\,\phi^{-2.14}$) to these data, and the dashed line is a fit ($\sigma_u = 4.04\,n^{-1.85}$) to data from smaller samples (25.4 mm) tested under the same set of conditions, where n is functional porosity.

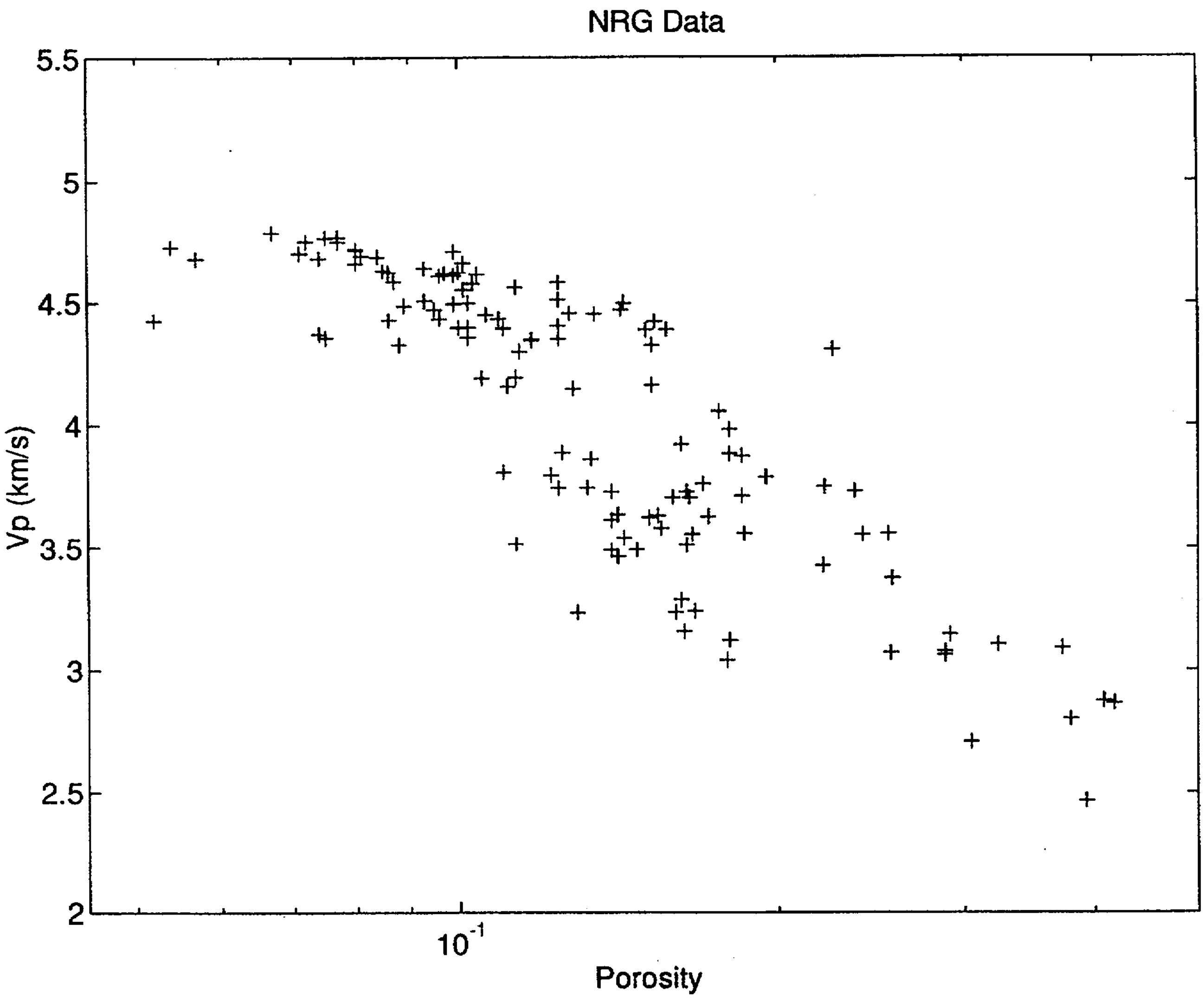

FIGURE 4.

Plot of p-wave velocity versus porosity for water-saturated samples of Yucca Mountain tuff with diameters of 50.8 mm and tested under room temperature, room pressure, and drained conditions at a constant axial strain rate of $10^{-5}s^{-1}$.

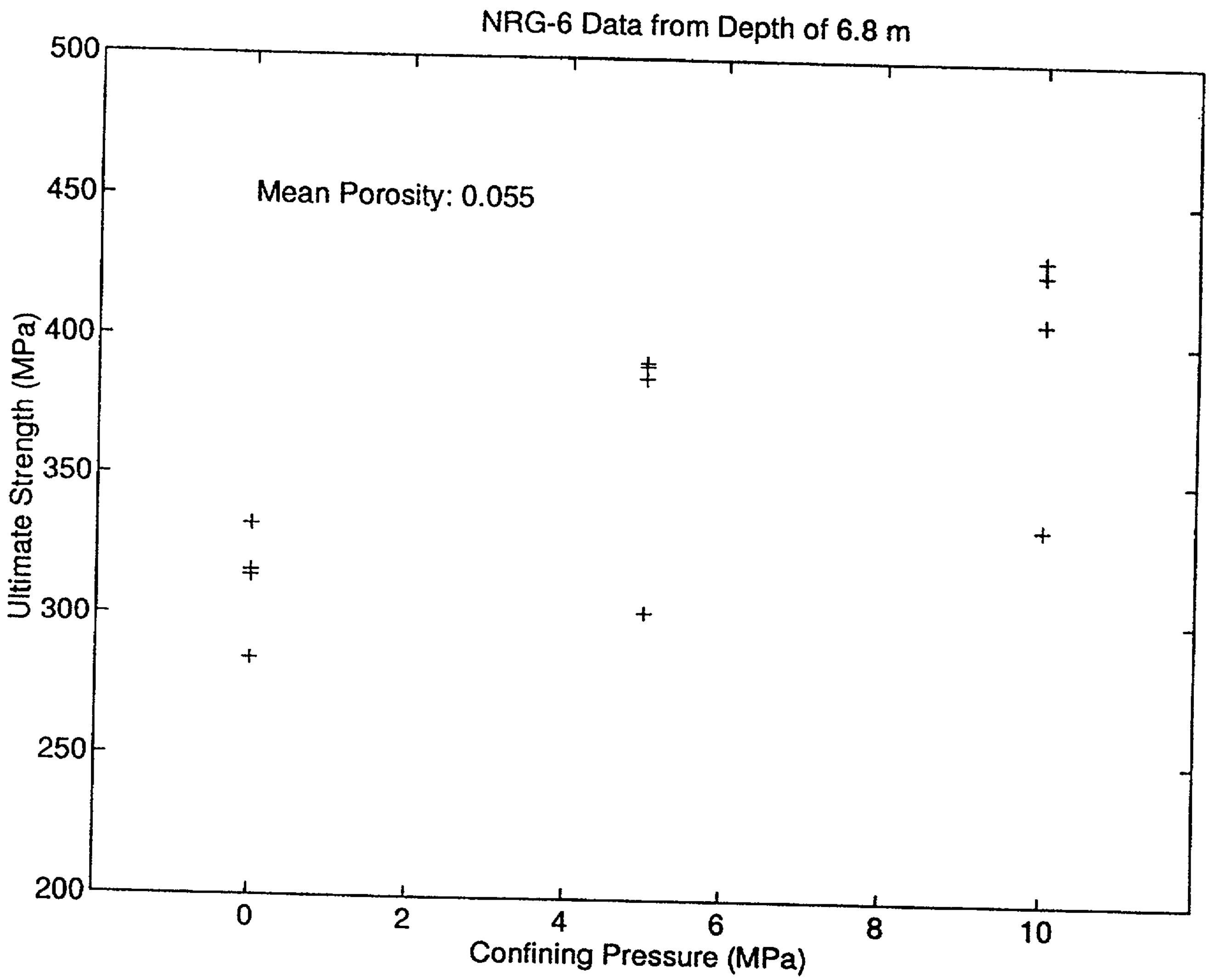

FIGURE 5.

Plot of ultimate strength versus confining pressure for NRG-6 samples from a depth of 6.8 m. The water-saturated samples with diameters of 50.8 mm were tested under room temperature, room pressure, and drained conditions at a constant axial strain rate of $10^{-5}s^{-1}$.

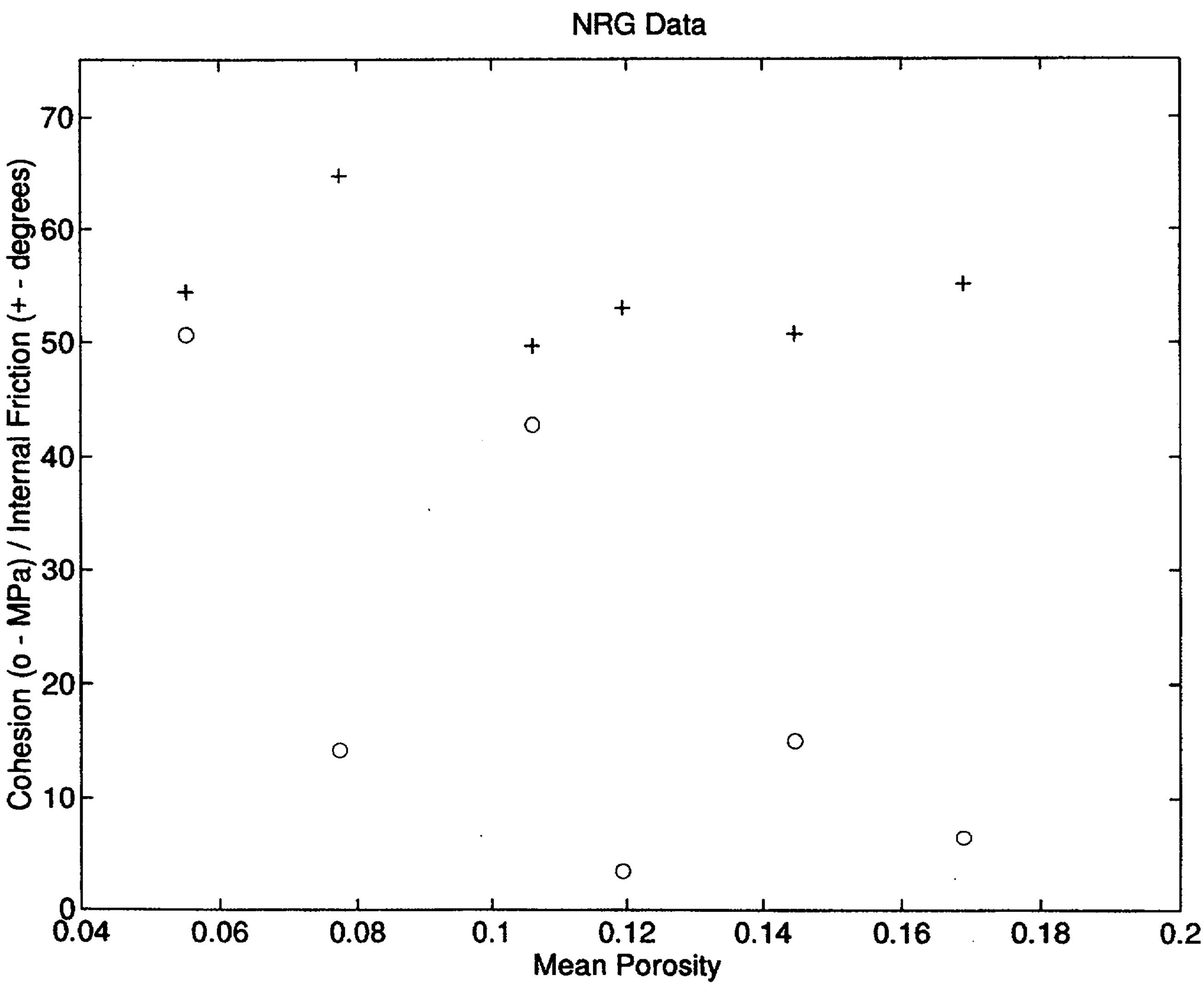

FIGURE 6.

Plots of cohesion and internal friction versus mean porosity for water-saturated samples of Yucca Mountain tuff with diameters of 25.4 mm and tested under room temperature, room pressure, and drained conditions at a constant axial strain rate of $10^{-5}s^{-1}$.

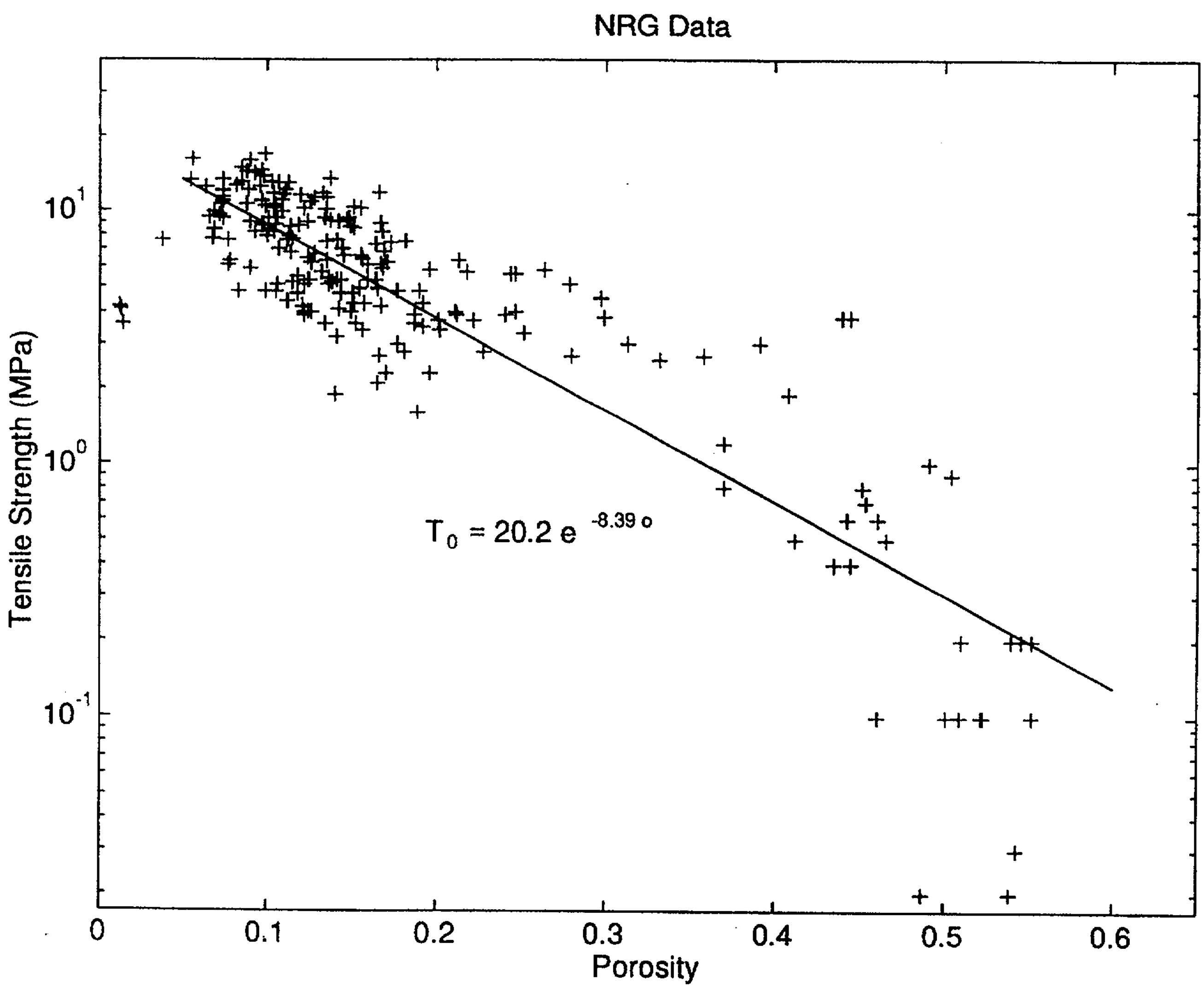

FIGURE 7.

Plot of tensile strength versus porosity for water-saturated samples of Yucca Mountain tuff with diameters of 50.8 mm and tested under room temperature, room pressure, and drained conditions in Brazil tests. Solid line is a fit ($T_0 = 20.2\,e^{-8.39\phi}$) to these data.

DRIFT-SCALE THERMOMECHANICAL ANALYSIS FOR THE RETRIEVABILITY SYSTEMS STUDY

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ABSTRACT: A numerical method was used to estimate the stability of potential emplacement drifts without considering a ground support system as a part of the Thermal Loading Systems Study for the Yucca Mountain Site Characterization Project. The stability of the drift is evaluated with two variables: the level of thermal loading and the diameter of the emplacement drift. The analyses include the thermomechanical effects generated by the excavation of the drift, subsequently by the thermal loads from heat-emitting waste packages, and finally by the thermal reduction resulting from rapid cooling ventilation required for the waste retrieval if required. The Discontinuous Deformation Analysis (DDA) code was used to analyze the thermomechanical response of the rock mass of multiple blocks separated by joints. The result of this stability analysis is used to discuss the geomechanical considerations for the advanced conceptual design (ACD) with respect to retrievability. In particular, based on the rock mass strength of the host rock described in the current version of the Reference Information Base, the computed thermal stresses, generated by 111 MTU/acre thermal loads in the near field at 100 years after waste emplacement, is beyond the criterion for the rock mass strength used to predict the stability of the rock mass surrounding the emplacement drift.

1. INTRODUCTION

Currently, the repository portion of the Mined Geologic Disposal System for the disposal of spent nuclear fuel and high level radioactive waste is in the ACD stage. A number of systems studies have been undertaken to support the development of ACD options for the repository design. The analyses presented in this report were done to support the Retrievability Period Systems Study. "Retrievability" is the capability provided by the repository system (by means of design approaches, construction methods, and operational procedures) to allow retrieval to be performed. In this report, the 100-year retrievability option is investigated. Within the views of retrievability, the design alternatives of emplacement drift size and thermal load are evaluated here. This evaluation is based on the judgment of the drift stability of the unsupported emplacement drift assessed by a numerical method.

Drift stability is both an important radiological and non-radiological safety issue during construction, waste emplacement, retrieval (if required), and closure of a nuclear repository. There is also a postclosure waste isolation issue related to the development of potential pathways for radionuclide migration. Stable drifts are produced through the use of appropriate excavation techniques (mechanical mining) and ground support systems based on a design that takes into account the effect of long-term loads, especially the waste-generated thermomechanical loads. Since empirical data in general is for underground design without thermal loads, numerical analysis is considered a useful tool in the process of determining the potential stability of repository drifts. The drift analyzed by the adopted numerical method does not include any tunnel support system because the results of this study may be used as input information for designing the ground support system.

The DDA code was used to analyze the thermomechanical response of the rock mass modelled as multiple blocks separated by joints. Using a perturbation scale to statistically choose the block sizes, rock blocks in the model were randomly generated with prescribed joint spaces and orientations. The change of joint openings was estimated using the change of contact condition between rock blocks which is the result of the movement of the entire system. Together with the joint openings, the stress field around the drift was used to assess the stability of the drift.

2. ANALYSIS METHOD

When a drift is excavated in a rock mass, the introduction of the boundary surfaces (drift walls) reduces their traction to zero from that corresponding to the natural state of stress. Such traction reduction induces the change of the stress state in the host rock around the drift, as well as displacements in the vicinity of the excavation. Furthermore, rock blocks of a highly fractured rock mass may dislocate from each other. Fractures in the rock mass may be extended or initiated, and existing fractures can be opened or closed by the change of the stress state. Thermally-induced stresses in rock can enhance the responses described above. In a two- or three-dimensional mechanical system, the fracture closure generated by the high thermal stress in one direction (when the displacement is confined in this direction) may induce the fracture opening in other directions due to the rigid body motion of rock blocks. With the possibility of rapid cooling of a repository drift performed for the retrieval of the WPs, the volume of rock blocks decreases with the decreasing temperature. Since the specified displacement boundary condition of the system does not change and the movement of rock blocks is not reversible upon the addition and removal of heat, the fracture size in the rock mass may further increase during the temperature reduction stage.

To assess the drift stability, thermomechanical analyses using the numerical code DDA were conducted. The opening and closing of fracture apertures in the vicinity of a waste emplacement drift, together with the induced stress state, are used to provide information for assessing the stability of the unsupported drift. Since the DDA code cannot model the initiation of new fractures (cracking) due to the change of stress state, only the dislocation and the opening/closing of existing fractures are simulated.

The DDA code is a two-dimensional numerical model for simulating mechanical processes of a discrete system which is physically divided into a finite number of blocks bounded by preexisting discontinuities. The code allows the relative movement between rock blocks that may be undergoing block deformation and rigid body motions. The motion of rock blocks includes inertial terms for deformation, rotation, and translation. For frictional and cohesive discontinuous surfaces between blocks, it is assumed that the Coulomb Frictional Law is applied.

The method of approximation used by the DDA code is similar to the method used in the finite element method where the solution of the complete system, as an assembly of its blocks, follows precisely the same rules as those applicable to a standard discrete system. Individual blocks are connected and form a block system by contacts between blocks and by displacement constraints on each. Information is required about block geometries, fixed points or directions on the boundaries, block material constants, discontinuity properties, initial stresses, body forces, and various external loads to describe the given block system. Elastic block and elastic/slipping contact between blocks are adopted in the DDA code.

Using the discrete time system simulates the dynamic behavior of a discontinuous system where motion over a long period of time is the accumulation of motion over multiple small time intervals. The recurrence relationship of the system behavior in a small discrete time interval is formulated using the minimum total potential energy theorem where the governing equations of Newton's law of motion, the elastic constitutive law, and the prescribed boundary conditions of all continuous blocks are solved simultaneously. The nonlinear contact behavior between blocks, which involves the frictional energy dissipation, is solved iteratively by choosing the trial contact positions between blocks. The theoretical detail and the formulations of the DDA code can be found in Shi (1993) and Tsai (1993).

The formulations for the mechanical behavior as a function of temperature changes were added to the original DDA code. The thermal loads are reflected by the change of temperature with time in each block. The immediate mechanical response of rock blocks caused by temperature changes is the variation of block strains. The variation of block strains at any instant of time is considered as a perturbation of the strain field. Using the linear elastic stress-strain relationship, a stress perturbation corresponding to the strain perturbation of a block can be expressed as $[\Delta \sigma_{ij}] = [E_{ijkl}][\Delta \epsilon_{kl}]$, where $[E_{ijkl}]$ is the elastic proper tensor of the block, $[\Delta \sigma_{ij}]$ is the incremental stress tensor, and $[\Delta \epsilon_{kl}]$is the incremental strain tensor. Using the linear thermal expansion law, the stress perturbation in a block can be expressed as $[\Delta \sigma_{ij}] = \alpha \Delta T[E_{ijkl}][\delta_{kl}]$ where α is the thermal expansion coefficient of the block at the current temperature level, $\Delta T$is the change of temperature at an instant of time, and δ_{kl} is the Kronecker delta. When the potential energy corresponding to the stress perturbation is taken into account for obtaining the dynamic equilibrium condition of the system, movements (deformations, rotations, and translations) of blocks generated by the thermal loads are induced.

Since the DDA code uses the discrete time system for the dynamic analysis, the temperature-dependent, nonlinear thermal expansion coefficient of rock block can be easily adopted. During a small discrete time interval for the computation of one time step (i.e., within the small change of temperatures occurred within a time step), the thermal expansion coefficient of a block is considered as a constant (according to the linear thermal expansion law). The nonlinearity of the thermal expansion coefficient is modelled by adopting the updated thermal expansion constant computed from the nonlinear function using the updated temperature at the beginning of each time step.

Although the DDA code is a numerical method used for simulating the dynamic process of a mechanical system, it can be used for the quasi-static analysis. By resetting the velocity vectors of all blocks to zero at the beginning of each time step computation, the solution of the governing equations after many time steps represents the quasi-static condition of the given mechanical system. Therefore, the time steps used in the quasi-static simulation are just a means to reach the convergence of an iterative solution process.

3. MODEL DESCRIPTION

Input information about the jointed-rock pattern, in situ stress condition, and the material properties of intact rock and rock joints is adopted from the RIB (DOE, 1994) of the Yucca Mountain Site Characterization Project and a Yucca Mountain Site Characterization report (Lin et al., 1993). Some modifications of the above-described information are made to obtain a simple model which can be reasonably analyzed using the DDA code. The input data for the DDA simulation is listed in Table 1. The nonlinear thermal expansion coefficients for the rock blocks as shown in Figure 1 are adopted from the preliminary results of a laboratory test performed by Sandia National Laboratories (SNL). The specimen used by this laboratory test was taken from borehole USW NRG-6 at the Yucca Mountain Site.

Table 1. Model Input Parameters

Joint Properties	Vertical Joints	Horizontal Joints	Intact Rock	
Orientation	100° dipping	10° dipping	Young's Modulus	32.7 GPa
Spacing	0.4 m	2.8 m	Poisson's Ratio	0.22
Frictional Angle	43 degrees	43 degrees	Bulk Density	2.3 g/cm^3
Cohesion	7.3 MPa	25.2 MPa		
Normal Stiffness	1.7×10^6 MPa/m	1.7×10^6 MPa/m		
Shear Stiffness	6.9×10^4 MPa/m	6.9×10^4 MPa/m		
Others				
Depth of Tunnel Invert		311 m		
Vertical In Situ Stress @ 311 m		7.00 MPa		
Horizontal In Situ Stress @ 311 m		3.85 MPa		
Ratio of Horizontal to Vertical Stresses		0.55		

Four computer runs were conducted using the DDA method to investigate the effects of drift size and the thermal load on the stability of the unsupported emplacement drift. Table 2 shows the identification of these computer runs. For a given thermal load expressed by the value of areal mass loading (AML), the drift spacing must be adjusted according to the chosen WP spacing. Therefore, only three drift layouts are needed for these four computer runs. The AML values listed in Table 2 were computed using average BWR and PWR fuel for which the conversion factor of the heat output is 1.03 KW/acre. In other words, 111 MTU/acre is equivalent to 114 KW/acre, and 83 MTU/acre is equivalent to 85 KW/acre.

Table 2. Computer Run Identification

Run ID	AML (MTU/acre)	Drift Diameter (m)	Drift Spacing (m)	WP Spacing (m)	Time (yrs)
#1	83	4.3	27.38	16.0	100
#2	111	4.3	20.48	16.0	100
#3	83	7.0	23.30	18.8	100
#4	111	7.0	23.30	14.0	100

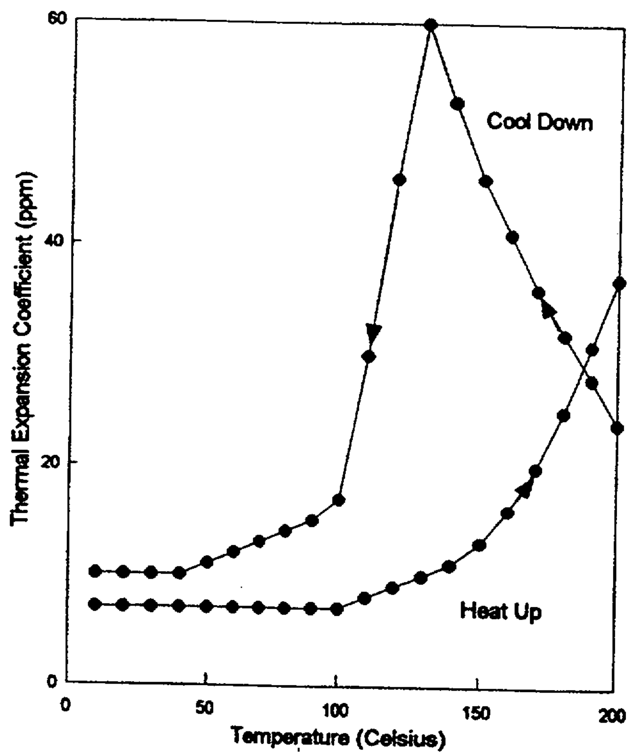


Figure 1 Instantaneous Linear Coefficient of Thermal Expansion

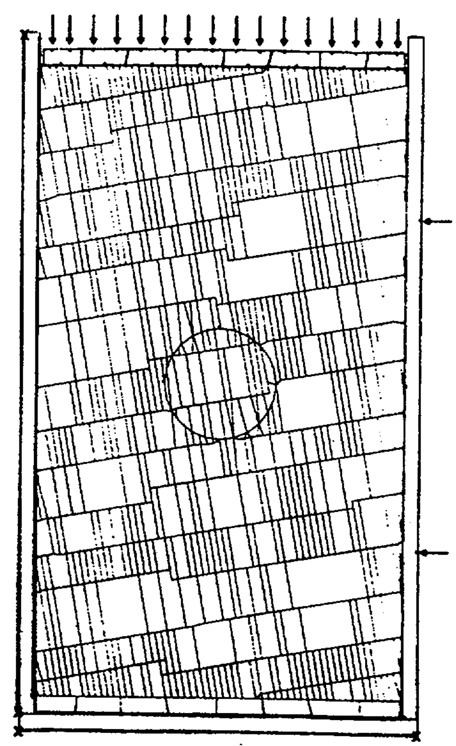


Figure 2 DDA Mesh for Run #4

Each computer run is performed in four stages: the initial stress loading stage, the excavation stage, the thermal loading stage, and the rapid cooling stage. For analysis of the initial stress loading, each mesh is composed of a jointed-rock mass of 40 m in height, and its width is the corresponding drift spacing. Each mesh is bounded with boundary and loading frames. Using the joint orientations and spacings listed in Table 1 and an arbitrary statistic perturbation scale, the rock blocks in the model are generated randomly. Figures 2 show the DDA models representing the jointed rock blocks of the rock mass which contains a 7.0-m diameter drift with 23.3-m drift spacing. It is the mesh of the initial stress-loading simulation for computer Runs #4. The left and bottom boundaries are fixed in the reference space (whereas the "X" marks shown in Figures 2 represent fixed points). The right frame is a lateral loading frame where external point forces (the arrow signs shown in Figures 2) corresponding to the horizontal in situ stresses are acting on the frame. Two points of application of the lateral loading are symmetric with respect to the horizontal center line of the right frame. The weight of the overburden rock formation is applied as the external vertical loads acts on a top-loading frame. The top-loading frame is broken into a number of loading blocks where the non-uniformly distributed displacements across the top of the mesh are allowed.

A number of load-bearing blocks are located between the bottom of the jointed rock mass and the bottom boundary frame. The Young's Modulus of these load-bearing blocks is one order magnitude smaller than the corresponding value for the rock blocks in the jointed-rock mass, so that non-uniformly distributed displacements across the bottom of the mass may be generated. In order to use the result of the initial stress-loading simulation as the initial condition of the excavation simulation, shapes of rock blocks in the center of the mesh are modified to fit the shape of drift.

In a conventional finite element analysis, it is not necessary to conduct the initial stress-loading simulation. The initial stresses can be a part of the input parameters to describe the initial condition. In the DDA analyses, the initial condition of the system also includes the relative position of block interfaces, although the initial stresses may be a part of input parameters. Contacts between blocks are established by the compression of contact springs which have no initial length. In other words, when two blocks are in contact, a small amount of geometrical overlap exists between them. Since this produces voids in the initially assigned system space, the rock blocks expand and block stresses are released. Although very stiff springs (as compared to the stiffness of rock blocks) can be used where only a very small amount of stress reduction is generated in each block, the large stiffness of contact springs creates the numerical difficulty for solving the system equations. To avoid such a difficulty, a DDA model can start with initial stresses and the geometries of overlapped blocks or with null initial stresses and rock blocks without overlapping surfaces. In the case of non-overlapped blocks with null initial stresses, the initial internal contacts are established by compacting the system using the external loads, which are statically equivalent to the in situ stresses.

After the quasi-static solution of the initial stress loading simulation has been obtained, rock blocks in contact are overlapped at their interface surfaces. To simulate the excavation process, the rock blocks within the drift are artificially removed. The remaining rock blocks with the deformed/displaced geometry are then used as the initial mesh for the excavation simulation. The corresponding final block stresses generated by the initial stress loading simulation are adopted as the initial block stresses for the excavation simulation. The lateral-loading frame is converted into a fixed displacement boundary frame to simulate the line of symmetry of the rock response induced by excavation of multiple drifts whose center-to-center distances are constant. The external vertical point forces remain the same as those for the initial stress-loading simulation to represent the weight of the overburden formation. The rock mass response induced by excavation is then obtained by searching the quasi-static solution of the above-described initial, boundary, and loading conditions.

Lingineni (1994) computed the temperatures of the WP and surrounding host rock as a function of time after waste emplacement. Thermal profiles in the near field were predicted for various AMLs, WP spacings, and drift diameters. The thermal calculation model is applicable to the in-drift emplacement mode where the modelled region consists of a single, infinitely long emplacement drift surrounded by the host rock. These computations were carried out in a one-dimensional, radial coordinate system. As a part of input data for studies reported here, Lingineni provides the 100-year, post-emplacement temperature at the center of each rock block in the deformed mesh generated by the excavation simulation for all runs identified in Table 2.

To analyze the thermal loading effects, the DDA computer run was restarted (without any change in the boundary and loading conditions) from the quasi-static solution of the excavation simulation with a perturbation of the block strains corresponding to the changes of block temperatures. By minimizing the total potential energy which includes the variation of the strain energy generated by the change of block temperatures, the DDA model computes the thermally induced block deformations and rigid body motions.

If the retrieval of waste canisters is required, it is possible that the air temperature in the drift will be cooled down by ventilation (rapid cooling) to facilitate the retrieval operation. The temperature distribution in the host rock during the retrieval operation is a function of the time history of the drift air temperature. The forced ventilation study, reported by Svalstad and Brandshaug (1983), demonstrated that the drift wall temperature is about 43°C, while the air temperature at the center of drift has been maintained at 26°C for about 0.6 years. The wet bulb globe temperature (WBGT) is a physiological heat stress index adopted by the National Institute for Occupational Safety and Health (NIOSH) which reflects the combination of air temperature, humidity, radiation, and wind speed. A WBGT of no higher then 26°C should be maintained for workers' stress control according to NIOSH regulations (1972).

Due to the lack of ventilation modelling work for the study reported here, the thermomechanical response of the rock mass was analyzed with the condition that the rock temperature at the surface of the drift wall has returned to 43°C. Lingineni's thermal computation model is used to calculate the time dependent temperature at the center of each rock block by keeping the temperature at the drift wall at 43°C. The temperature of rock blocks at three years after assigning this drift wall boundary condition is adopted for the rapid cooling simulation. It is noticed that the time at three years after the rapid cooling does not have the physical meaning of real time because of the lack of ventilation modelling work. The computed temperature profile at three years after the blast cooling is just a conceptual estimation of the temperature distribution during the retrieval period. Expressed in a one-dimensional radial coordinate system, the input temperature profile for thermal loading and cooling as a function of the radial distance from the center of the drift is shown in Figure 3 for Runs #1 and #2. The temperature profiles for Runs #3 and #4 are shown in Figure 4.

For the rapid cooling simulation, the DDA code computer run is again restarted from the quasi-static of the thermal loading simulation with another perturbation of the block strains corresponding to the change of temperatures induced by rapid cooling. The rock mass response of rapid cooling is sought through the quasi-static solution of the perturbed system equations.

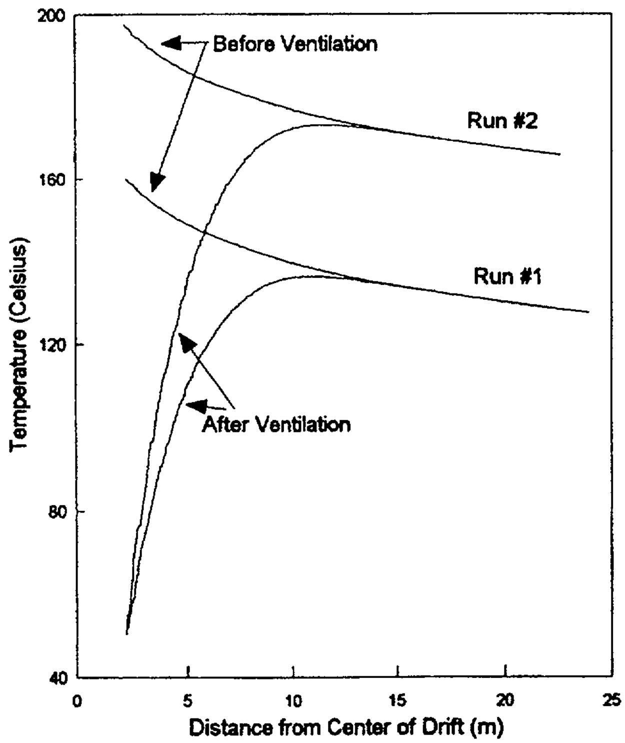


Figure 3 Temperature Profile for Runs #1 and #2.

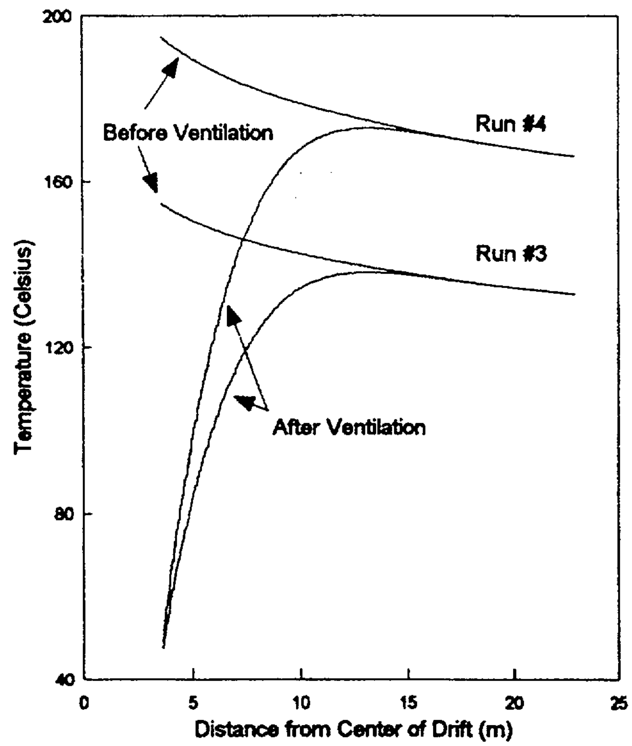


Figure 4 Temperature Profile for Runs #3 and #4.

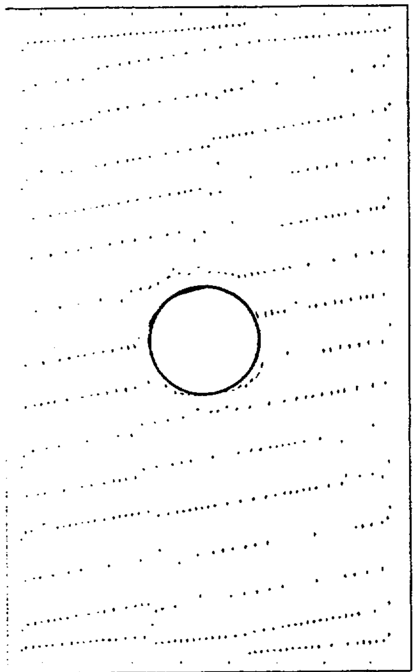
4. RESULTS

The results of each DDA run can be represented by the displaced geometry, the stress distribution, and the joint opening distribution of the entire system. Since the rock mass is very stiff, the displaced mesh cannot be differentiated visually from the initial mesh of each stage simulation. These stress fields can be presented as stress trajectories plotted at the center of each rock block. The orientation of a trajectory represents the direction of the principal stress. The magnitude of a principal stress is represented by the length of the trajectory. To observe the change of the character of joints, the status of contacts in the system has been presented graphically. A contact is referenced by the force/displacement relationship between a corner of a block and an edge of another block. If the contact corner of one block has invaded (overlapped) into the other block through the reference edge, this contact is closed. A joint is represented by two edges of different blocks nearly parallel to each other. Therefore, a joint is referenced by the pair of contacts where two contact corners and two contact edges are parts of the edges of the joint. If a contact corner of one block is close to but not touching the referenced contact edge of another block, the joint between these two blocks is open. A joint is closed only if two contacts used to reference the joint are both closed. The size of a joint opening can be approximately represented by the associated contact opening which is the distance from the corner of a block to the referenced contact edge measured perpendicular to the edge. Each open contact can be represented by a vector normal to the referenced contact edge. The length of the vector represents the magnitude of the contact opening.

The results of the DDA computer runs are explained in detail using the results of Run #4 as an example. The results of all computer runs were then presented in tabular form. For the initial stress-loading simulation, a fairly uniformly distributed stress field was generated by the external loads which is in static equilibrium with the in situ stresses. The vertical stresses of rock blocks near the top of the mesh are about 6.5 MPa, while they are about 7.4 MPa in the rock blocks near the bottom of the mesh. The ratio of the horizontal stress to the vertical stress remains at about 0.55 in each rock block. The vertical deformation along the center line of the entire mesh is about 9.2 mm. The horizontal deformation along the mid-height of the mesh is about 1.4 mm. The high vertical deformation is caused by the high vertical in situ stress and the large height-to-width ratio (average about 4.6 to 1) of the block size. Thus, very highly concentrated contact forces across horizontal joints are generated. The higher the contact forces, the more block overlaps are induced. Since non-uniform contact forces are generated among blocks of various sizes, the principal stresses of each rock block are slightly different. As a result of the initial rock mass loading, all joints are closed.

The final stress distribution and joint openings of the initial loading simulation for Run #4 is shown in Figure 5 and 6. The stress concentration after excavation around the drift forms a pressure arch which makes the drift opening structurally stable. Noticeable joint openings (as shown in Figure 6) after the excavation, occur only in the rock mass in the immediate vicinity of the drift walls (the invert and the roof). The effect is limited to a distance less than the half diameter of the drift (3.5 m). Except for joints formed by a few rock wedges at the upper left and the lower right corners of the drift, the size of joint openings in the immediate area into the drift wall is on the order of 0.01 mm or less. The vertical and horizontal drift closures at the excavation loading stage are about 1.6 and 0.3 mm, respectively. The center of the upper boundary of the modelled rock mass subsides at about 0.7 mm.

Figures 7 and 8 show the final stress distributions and joint openings of the thermal loading stage. The vertical stresses after thermal loading in all blocks remain close to their initial values (6.5 to 7.4 MPa) due to allowable upward rock mass thermal expansion. The horizontal stresses after thermal loading are significantly increased over the entire mesh, due to the fixed lateral displacement boundary conditions on both sides of the mesh. In general, the horizontal



Scale: 100MPa

Figure 5 Principal Stresses After Excavation for Run #4

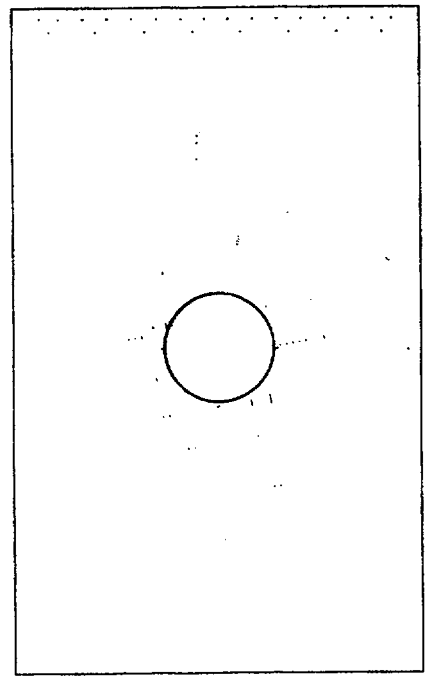


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Figure 6 Joint Openings After Excavation for Run #4

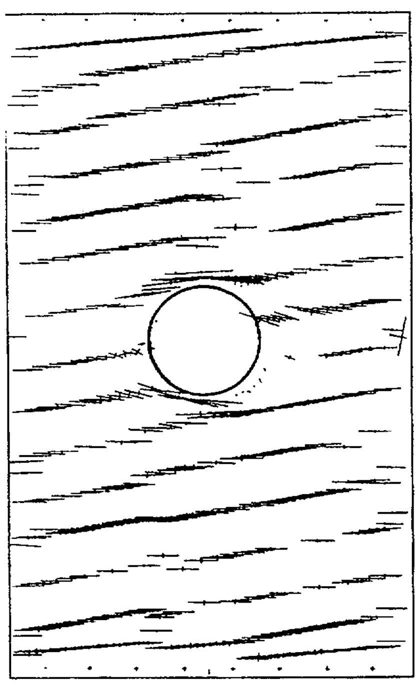

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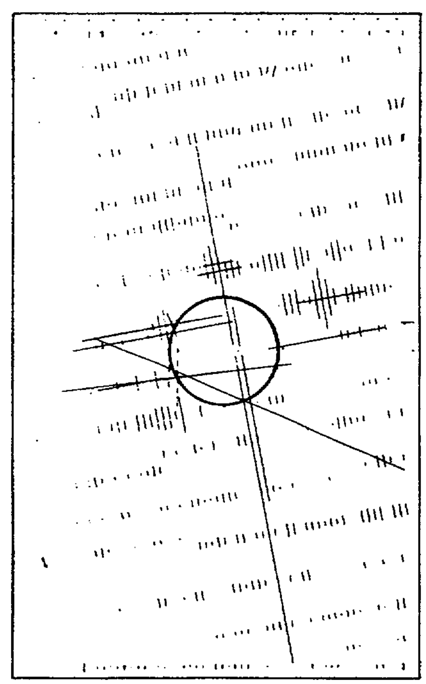
Figure 7 Principal Stresses After Thermal Loading for Run #4



Scale: 1mm

Figure 8 Joint Openings After Thermal Loading for Run #4 stresses in the rock mass at a few meters away from the drift are about 55 to 60 MPa. The model with fixed lateral displacement boundary condition represents the center of the repository only. In the edge of the repository, a certain amount of lateral thermal expansion of rock mass is allowed. Therefore, less horizontal stresses are built up after thermal loading.

In the immediate area around the drift, the major principal stress may reach 90 MPa. With this high principal stress in the jointed rock mass immediately around the drift, rock spalling may occur in the drift walls. The large joint openings after thermal loading in the rock mass immediately around the drift wall suggest that some rock wedges at the drift wall may become detached from the drift wall. The openings of horizontal joints are about 0.2 to 0.7 mm in the area within about 10 m radius from the drift wall. In general, all of the vertical joints remain closed under the thermal loads. In contrast to the 1.6 mm vertical drift closure and 0.3 mm horizontal drift closure generated in the excavation stage, the drift height measured through the center of the drift increases (expands) 13.4 mm, while the drift width measured through the center of the drift closes another 13.3 mm during the thermal loading stage. Thermal loading also raises the top of the modelled rock mass 83.2 mm.

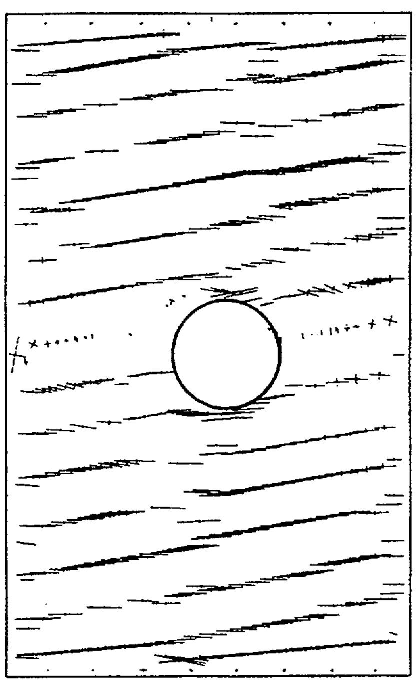


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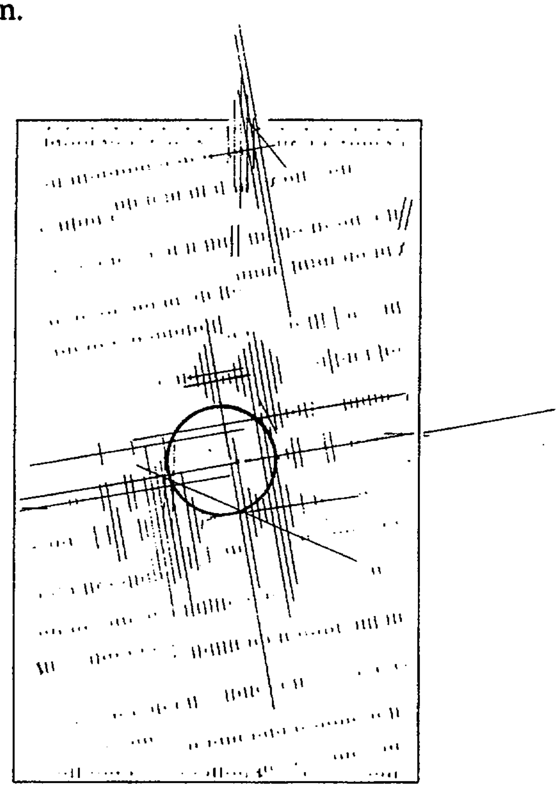
Figure 9 Principal Stresses After Rapid Cooling for Run #4



Scale: 1mm

Figure 10 Joint Openings After Rapid Cooling for Run #4

Figures 9 and 10 show the final stress distributions and joint openings of the rapid cooling stage. The horizontal stresses after the rapid cooling in the rock mass at a few meters distance away from the drift wall reduce only 2 or 3 MPa. In the immediate vicinity of the drift walls, the horizontal stresses after the rapid cooling at roof and floor reduce significantly. The stresses at the corners of the roof and floor vanish while the rock blocks are further detached from the drift wall. In the drift pillars, the horizontal stresses vanish and the vertical stresses remain to support the weight of the overburden formations.

As shown in Figure 10, the openings of horizontal joints in the area within about 7 m radius from the drift wall are much larger than 1 mm. Outside this highly disturbed zone, all of the vertical joints remain closed, and the horizontal joint openings do not change significantly as compared to the results of the thermal loading simulation. The vertical drift closure measured through the center of the drift increases 4.2 mm, and the drift width measured through the center of the drift increases 0.8 mm during the rapid cooling stage. Rapid cooling subsides the top of the modelled rock mass 5.8 mm. Except at the corners of the roof and floor, the pressure arch in the rock mass immediately surrounding the drift remains and the drift is generally stable. However, the large joint opening immediately surrounding the drift suggests that small rockfall may be encountered after the rapid cooling.

As a conclusion to the numerical results of Run #4 (the case of 7 m drift diameter and 111 MTU/acre thermal load), drifts will be stable after excavation, and their structural integrity remains after thermal loading. The pressure arch around the drift wall is formed due to the removal of rock blocks during the excavation process. The high horizontal stresses at the drift roof connect rock blocks after thermal loading, causing them to react like a thick beam to support the weight of the overburden formations. Vertical joint apertures above and below the drift closure with the application of thermal loads are desirable from a stability viewpoint. However, the very high compressive tangential stress in the immediate area may induce rock spalling. The rapid cooling does not make the state of rock mass change back to what it was before thermal loading because the mechanical responses of rock mass around the drift are not reversible. In fact, much larger joint openings immediately surrounding the drift suggest that the drift is less stable. Some rock blocks at the corner of the roof and floor are unstressed and may fall into the drift. Since the heights and widths of rock blocks vary, the non-uniformly distributed thermal shrinkages of rock blocks cause concentrated (non-uniform) vertical stresses in the pillar to support the overburden rock mass.

The vertical drift closure, horizontal drift closure, the subsidence of the top of the mesh, the joint openings in the area immediately around the drift, the joint openings in the area away from the drift, the tangential stresses at roof/floor, and the horizontal stresses at areas away from the drift for all computer runs are listed in Table 3.

5. DISCUSSIONS

To evaluate the stability of the rock mass surrounding an excavation, the computed stresses in the rock mass must be compared with the established criterion for the rock mass strength. The following description of the criterion of the rock mass strength is quoted from the current version of the Reference Information Base (DOE,1994): "The criterion for the rock mass strength is used to predict the stability of the rock mass surrounding an excavation. The criterion, along with the predicted stress states of the rock mass, can be used to predict failure or overstressing as a result of compression or crushing, but it cannot be used to predict failure resulting from slippage or shearing along a dominant joint. Rock mass compressive strength is defined as the strength of a representative volume of intact and jointed rock matrix material. Estimates of the rock mass strength are based on the behavior of intact rock, the knowledge of joint characteristics, and the presence of applied stress or confinement. The rock mass strength criterion for TSw2 formation is $(\sigma_1)_{ult} = 16.0 + 10.2\sigma_3^{0.602}$ for $0 < \sigma_3 < 25$ MPa, where $(\sigma_1)_{ult}$ is the ultimate principal stress at rock mass failure (MPa) that is equivalent to strength of rock mass and σ_3 is the confining stress (MPa)."

Table 3. Results of Computer Runs

	Run #1	Run #2	Run #3	Run #4
Vertical Drift Closure (mm)				
by excavation	0.6	0.6	1.6	1.6
by thermal loading	-5.0	-8.3	-8.9	-13.4
by rapid cooling	0.7	1.7	2.9	4.2
Horizontal Drift Closure (mm)				
by excavation	0.1	0.1	0.3	0.3
by thermal loading	2.8	4.8	7.8	13.3
by rapid cooling	1.2	1.9	1.2	-0.8
Subsidence of the Top of Mesh (mm)				
by excavation	0.2	0.3	0.7	0.7
by thermal loading	-53.8	-83.9	-54.9	-83.2
by rapid cooling	4.3	6.9	7.2	5.8
Joint Openings Around Drift (mm)				
after excavation	< 0.01	< 0.01	0.1	0.1
after thermal loading	0.1	0.3	> 0.3	> 0.5
after rapid cooling	0.3	> 0.5	> 0.3	> 2.0
Joint Openings Away from Drift (mm)				
after excavation	0.0	0.0	0.0	0.0
after thermal loading	0.1	0.2	0.1	0.2
after rapid cooling	0.1	0.2	0.1	0.2
Tangential Stresses at Roof/Floor (MPa)				
after excavation	3.4	4.2	4.3	4.3
after thermal loading	73	109	58	97
after rapid cooling	30	62	28	57
Horizontal Stresses Away from Drift (MPa)				
after excavation	3.8	3.8	3.8	3.8
after thermal loading	38	53	36	55
after rapid cooling	36	51	35	53

In the DDA method, the stresses of rock blocks are computed. Although the rock block is equivalent to the intact rock, the rock block stresses can be considered as the local rock mass stresses if the level of stresses in the neighboring blocks is in the same level. For example, if a 10 x by 10 m rock mass confining hundreds of rock blocks is pressurized with 100 MPa confining pressure, the stresses in rock block will be close to 100 MPa. This analogy is also demonstrated by the result of the DDA analysis for the initial stress loading simulation shown in Section 4.

In the results of each computer run described in Section 3, the minor principal stresses at the area a few meters away from the drift are approximately equivalent to the vertical stresses (7 MPa for all stages, initial stresses, excavation, thermal loading, and rapid cooling simulations). The vertical stresses are the balance of the weight of the overburden rock mass. The temperature effects do not change the vertical stresses in the area a few meters away from the drift due to the allowable upward free thermal expansion in the vertical direction of the modelled rock mass. After inserting $\sigma_3 = 7$ MPa into the above equation for the criterion of the rock mass strength, the strength of rock mass is about 49 MPa.

As shown in Table 3, the horizontal stresses after thermal loading in the area a few meters away from the drift are 53 MPa for Run #2 and 55 MPa for Run #4, indicating that the rock mass in the near field of an emplacement drift is predicted to fail under compression generated by the 111 MTU/acre thermal load. This statement can be applied to either the 7.0 m or 4.3 m diameter drift. This DDA result about the high horizontal stresses generated by the thermal load can be validated with a simple one-dimensional analytic solution using the theory of linear elasticity, the theory of linear thermal expansion, and the boundary condition that the displacements at both ends of the one-dimensional material are fixed. The nonlinear thermal expansion coefficients of the rock in the range of 25 to 200°C (as shown in Figure 1) can be approximately taken as 10 parts per million (ppm); and the mean value of the Young's Modulus of the rock is 32,700 MPa (DOE,1994). As shown in Figures 3 and 4, the temperature in the area a few meters away from the drift may change from 25 to about 175°C after the 111 MTU/acre thermal load. Every meter of the one-dimensional rock bar will expand 0.0015 m if the ends of the bar are unrestrained. In order to compress the bar to its original length, 49 MPa compressive stress must be applied in the bar. If the 4 MPa in situ horizontal stress is superimposed in the temperature induced compressive stress, the stress value is 53 MPa, compared to the DDA results (53 MPa for Run #2 and 55 MPa for Run #4), suggesting that the thermal stresses modelled by the DDA method are validated. (Equivalent vertical stresses calculated before and after the thermal loading also serve to validate the DDA's thermal stress computation.)

The above statement about rock mass failure in the near field is only dependent on the validity of the criterion of rock mass strength and the assumption of fixed boundary displacement in the horizontal direction, as well as free boundary sliding in the vertical direction at both sides of the mesh (shown in Figure 2) for the thermal loading stage simulation. Such assumption is widely used for simulating the rock mass response of a mining layout with multiple parallel drifts. It is believed that this assumption is reasonable for simulating the rock mass response at the center part of the entire repository which consists of many parallel emplacement drifts. Therefore, the 111 MTU/acre thermal load appears to be unfeasible since the ground support for the entire near field appears to be unfeasible.

The horizontal stresses in the near field (the area a few meters away) of an emplacement drift for 83 MTU/acre thermal load appear to be below the rock mass strength described in the RIB. However, there are stress concentrations in the immediate area around the drift in addition to the thermally-generated near field stresses. As shown in Table 3, the tangential stresses around the drift are 73 MPa for Run #1 and 4 MPa for Run #3, which are far beyond the rock mass strength. Therefore, local rock spalling is expected to occur due to the overstressed rock mass immediately around the drift. This local rock mass instability is also suggested by the high joint openings shown in the immediate surrounding of the drift. It is possible that this local rock mass instability just around the drift may be stabilized by a tunnel support system.

It is concluded that the thermal stresses in the near field do not exceed the criterion of the rock mass strength for any thermal load less than 83 MTU/acre. The tangential stresses immediately around the drift, which govern the local rock mass instability (local rock spalling), are about proportional to the thermal loads. Therefore, there is much less rock mass stability and fewer local rock spalling problems if the 24 MTU/acre thermal load is adopted.

For all thermal loads and emplacement drift layouts considered for computer runs identified in Table 2, the temperatures at various points of the DDA model computed using Lingineni's thermal code at 200 years after emplacement have changed only a few degrees compared to the corresponding temperatures at 100 years after emplacement. Since the DDA model simulates a quasi-static mechanical behavior (i.e., the mechanical response doesn't change with time), the thermomechanical response generated by a few degrees change in temperature is negligible. However, this model consideration does not suggest that the emplacement drift at 200 years after the thermal loading remains stable. The drift stability at 200 years must be evaluated using the engineer's judgment about the deterioration of the rock mass and the tunnel support system during a very long period of time.

The function of the repository is to provide containment and isolation of the waste. Yet retrievability is a specific performance objective the repository must meet if necessary. The requirement for retrievability should not affect or unnecessarily complicate the design of the repository to the exclusion of, compromise with, or interference with the function of the repository. However, the method of retrieval planned should anticipate and identify all credible malfunctions or accidents to the emplacement system, the engineered barriers, and the host rock that could affect retrievability. The results of numerical analyses described in this report indicate that the effects to the host rock from thermal conditions which were not present at the time of emplacement could significantly influence the method and equipment employed to retrieve emplaced waste.

The design of the retrieval method and retrieval equipment for the repository should anticipate the deterioration and degradation of ambient emplacement conditions by the time retrieval may become necessary. These effects, which could increase the difficulty of retrieval, will likely increase in magnitude during the period retrievability must be maintained. By choosing a particular set of drift sizes and thermal loads, based on the results of the drift stability presented in this report, such difficulty can be minimized. If the unsupported emplacement drift appears to be unstable during the retrieval period, it will possibly create difficult and hostile operating conditions underground for both equipment and personnel. Therefore, the retrieval method should be developed in conjunction with the planned emplacement method.

6. CONCLUSIONS

The mechanical response of the rock mass around a potential repository drift has been analyzed to assess the stability of the drift and the rock mass surrounding it. The mechanical responses of the rock mass induced by excavation, thermal loading, and rapid cooling were analyzed using the DDA code. The effects on the stability caused by different drift sizes and different thermal loads were investigated.

The results of the numerical analyses for cases of 111 MTU/acre thermal load show that very high, major principal stresses and low, minor principal stresses are generated by the thermal load in the entire near field of an emplacement drift in the center of the repository. Comparing these results with the data of the rock mass strength described in the current RIB, it is predicted that the rock mass in the entire near field may fail or be overstressed as a result of compression or crushing generated by 111 MTU/acre thermal load. Therefore, the 111 MTU/acre thermal load appears to be unfeasible since the ground support for the entire near field in the center of the repository appears to be unfeasible. In the edge of the repository, the horizontal thermal stresses are less than those shown in the computer simulation due to the existence of a certain amount of lateral thermal expansion of rock mass. Therefore, less rock mass instability problems in the near field in the edge of the repository are expected.

In general, the drift will be stable after excavation and its structural integrity remains after thermal loading if the 83 MTU/acre thermal load is applied. In such cases, the emplacement drift with the smaller diameter appears to have better stability due to the smaller free surface allowing the rigid body motion of rock blocks separated by joints. However, a small amount of rock spalling or rockfall may still occur in the drift due to the very high, thermally- generated tangential stresses immediately around the drift. More rockfall may occur during the retrieval period due to the stress relaxation caused by the rapid cooling in the area immediately around the drift. Therefore, a tunnel support system is still required to prevent rockfall at the environments for the retrieval operation before and after the rapid cooling. In the edge of the repository, fewer horizontal thermal stresses are expected. However, these horizontal stresses are higher than the concentrated stresses induced by excavation only. Vertical joint apertures above and below the drift close with the application of thermal loads, which is desirable from a rock mass stability viewpoint. Since these horizontal stresses do not exceed the criterion of the rock mass strength, the drift and the near field of drifts in the edge of the repository are expected to be more stable than those in the center of the repository for the cases of 83 MTU/acre thermal loads.

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CEMENT-BASED GROUTS IN GEOLOGICAL DISPOSAL OF RADIOACTIVE WASTE

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ABSTRACT: The behaviour and performance of a specially developed high-performance cement-based grout has been studied through a combined laboratory and *in situ* research program conducted under the auspices of the Canadian Nuclear Fuel Waste Management Program (CNFWMP).
A new class of cement-based grouts - high-performance grouts - with the ability to penetrate and seal fine fractures was developed and investigated.
These high- erformance grouts, which were injected into fractures in the granitic rock at the Underground Research Laboratory (URL) in Canada, are shown to successfully reduce the hydraulic conductivity of the rock mass from $<10^{-7}$ m s^{-1} to 10^{-9} m s^{-1} and to penetrate fissures in the rock with apertures as small as 10 μm. Furthermore, the laboratory studies have shown that this high - performance grout has very low hydraulic conductivity and is highly leach resistant under repository conditions. Microcracks generated in this materials from shrinkage, overstressing or thermal loads are likely to self-seal.

The results of these studies suggest that the high-performance grouts can be considered as viable materials in disposal-vault sealing applications. Further work is needed to fully justify extrapolation of the results of the laboratory studies to time scales relevant to performance assessment.

1 INTRODUCTION

Cement-based materials constitute an important part of the Canadian and other nuclear waste management programs (Johnson et al. 1994). The advancement of disposal strategies has contributed to the development of a new generation of cement-based materials designed for fluidity, penetrability, impermeability, low heat of hydration, volumetric stability and resistance to chemical attack, as grouts to seal fractures and for use as massive seals.

In any engineered system, the materials and structures must be able to maintain their performance under the range of physical and chemical conditions to which they will be subjected over their design life. The required longevity of the seals to be used in underground disposal vaults for heat-generating radioactive waste is a unique demand placed on the performance of the materials. Depending of the nature of the waste form and the disposal concept, the required longevity may extend into periods of hundreds of thousands of years. Consequently, ensuring the longevity of the engineered barriers in a disposal vault will require the prediction of the long-term properties and performance of materials for periods longer than that during which humankind has been building towns, cities and all related structures. It was noted that insufficient information was available to allow for reasonable analyses of the performance of the sealing materials and the systems of which they form a part. On the assumption that unique seals may be needed in the design and construction of disposal vaults for heat-generating radioactive waste, coordinated *in situ* and laboratory investigations were undertaken to provide the needed information and to study the behaviour and performance of these new classes of cement-based grouts, the high-performance grouts. Generally all approaches taken to assess how these materials will perform in the disposal system require an understanding of the physical and chemical conditions to which sealing materials will be subjected in a disposal vault. These factors will be specific to the disposal vault site, the design of the disposal vault, the location and design of the sealing system within the disposal vault, and the required function of the seal.

The methods used to study the interaction between sealing materials, the rock and groundwater for a disposal vault located under the water table in saturated granite (such as that represented by the URL in Canada and the Stripa site in Sweden) after the disposal vault is closed may differ from those used to study material-performance factors while a repository is open. The significance of differences in hydraulic gradients, temperatures and other environmental factors needs to be quantified. Experience has shown that the physical conditions of the cement-based grouts injected into fractured rock depends on the properties of the grout material, the *in situ* rock conditions and the method used for injection. The investigations were required to assess the effects of these factors on grout longevity.

It is commonly assumed that the hydraulic performance of the cement-based materials used to seal a disposal vault would change as water passes through the pores in the material. Cement-based materials will dissolved in a saturated disposal vault, consequently the porosity and hydraulic conductivity of this materials will increase with time (Coons 1987). The rate of change would be affected by the immediate changes in the microstructure of the material and by the internal and external physicochemical processes on the surface of the grout. The rates at which these processes proceed in the high-performance sealing materials is uncertain. Few data are available on the relationships among the porosity, hydraulic conductivity and strength for this new class of cement-based materials. It is also known that there are other possible mechanisms that may cause cement-based sealing materials to lose their low hydraulic conductivity performance. These mechanism includes:

- microcracking under the excessively high loads imposed by the rock mass,
- microcracking from drying shrinkage,
- microcracking from differential thermal expansion, and
- thermodynamic instability of the phases, which may increase the porosity by solid phase transformation.

The presence of any defects (cracks, capillary pores) in a cement-based grout structure has the potential to provide a more rapid transport pathway for advection or diffusion of radionuclides. The presences of these defects could impair the performance of cement-based grouts as a barrier to radionuclide migration. It has been reported that cracks in concrete can self

- seal in the presence of moisture by a process known as "autogenous healing" (Wagner 1974, Munday et al. 1974, Dhir et al. 1973). The processes is now generally accepted to occur in concretes and cements. If self-sealing can be assured, particularly for the high-performance grouts, in the chemical conditions likely within an underground disposal vault, the cracks would be much less significant.

It is assumed that the cement-based materials selected for sealing will not have to play a primary structural role in supporting the rock. However, in order to prevent leakage, the sealing material will be required to maintain an effective bond with the host rock to minimize flow at the rock/grout interface.

High-performance cement-based grout materials and appropriate grouting methods were investigated in the laboratory and in the field trials at the URL. The major results from these investigations are presented in this paper.The short-term hydraulic properties, leaching behaviour and the ability of the high-performance grout to self seal are discussed.

2 CEMENT-BASED SEALING MATERIALS

Portland cements and cement-based concretes are used extensively in engineering practice. For conventional civil engineering practice, high-performance cement-based grouts containing superplasticizer and silica fume can be manufactured using any of the five commonly employed portland cement types (ASTM Type I to V). However, studies have indicated Type V (Sulphate Resistant Portland Cement-SRPC) was likely to prove to be the most desirable from the perspective of long-term performance (Hooton and Mukherjee, 1982). Other cement products designed for sulphate resistance, such as Class H (oil-well) cement and blast furnace slag cement (BFSC), may prove equally acceptable. Proprietary grouting products based on these latter two materials are available commercially. However, the generic nature of the research program and general lack of ability to obtain detailed information on proprietary products led to the selection of cement based on SRPC. A finely reground and well - characterized SRPC was selected.

The purpose of grouting the rock in which the disposal vault is built is to seal fractures that might contribute to the migration of the dissolved nuclear waste to the environment. Moreover, in view of the fact that the hazardous materials in nuclear fuel wastes are very-long lived, the fractures would have to be sealed for long time periods. To attain this objective, the grouts must be shown to have the following properties:

- acceptable hydraulic conductivity (e.g.,$< 10^{-12}$ m s^{-1}) or comparable to that of surrounding intact rock mas,
- an ability to penetrate very fine fissures within the host rock, and
- a physical and chemical compatibility with the host environment.

Other desirable characteristics are that the grout possess an ability to self-seal internal or interfacial fractures that could rise from physical disturbances (e.g., movement and stresses of the rock, drying and shrinkage), resist leaching, and not alter the groundwater chemistry significantly.

Reference cement-based grout formulations were selected for the laboratory and *in situ* investigations with these objectives and practical requirements of emplacement in mind.

A number of variations in the type of cement have been developed and are available commercially to meet specific requirements, these variations include rapid hardening, low heat of hydration and resistance to sulphate attack.

Three type of cements were considered in the investigations: sulphate-resistant portland cement (SRPC, Canadian Type 50, US Type V), expansive cement (Canadian Type K) and ALOFIX-MC (MC-500), a commercially available slag cement with an extremely fine particle size (Onofrei et al. 1993).

Type 50 cement was considered to be appropriate for use in Canadian granitic rock where some groundwaters contain sufficient quantities of sulphates to be considered aggressive. While portland cement grouts injected into saturated fractures zones should not shrink significantly during setting and hardening, the use of expanding cements (Canadian Type K) could be advantageous in sealing applications. They may improve the contact at the grout-rock interface and thereby limit groundwater flow.

Since the penetration of very fine fractures was of interest, both the type 50 and type K cements were investigated at their normal fineness and after regrinding. The MC-500 cement is extremely fine, and no regrinding was necessary. Figure 1 shows the particle size distributions of the cement investigated.

Both silica fume (a pozzolanic[1] material) and superplasticizer were incorporated in all mixes. Silica fume was used to minimize the amount of readily soluble residual lime (CH) in the hardened products. The CH is converted to less leachable calcium silicate hydrates (CSH). While the supposed prime benefit of this pozzolanic reaction is enhanced durability, the studies showed that grouts with silica fume also exhibited less bleeding[2] and segregation (Figure 2) (Al-Manaseer et al. 1991, Onofrei et al. 1993). Bleeding can occur after injection by simple settlement, or during the injection by consolidation of the solids under pressure gradients. Bleeding can cause poor bonding between grout and rock, as well as inhomogeneities in the chemical compositions and microstructure of the grout. Silica fume was obtained from SKW Canada, Becancour, Quebec, Canada.

Superplasticizer was incorporated to reduce the *W/CM*[3] ratio of the grout while achieving a viscosity that is low enough to permit injection into the rock (Aitcin et al. 1989). The superplasticizer used was a proprietary sodium salt of sulphonated naphthalene formaldehyde

1 Pozzolanas are silicious materials that, though not cementitious in themselves contain constituents that will combine with lime in the presence of water at ordinary temperatures to form compounds with a low solubility that possess cementing properties.

2 Bleeding is the separation of the solid particles from the liquid phases of a freshly mixed grout.

3 *W/CM* is the ratio of the mass of water to the combined mass of cement plus pozzolana (cementitious materials).

condensate (Na-SNFC) supplied by Handy Chemical Ltd., La Prairie, Quebec. Canada. The properties of this material are discussed by Aitcin et al. (1989) and by Onofrei et al. (1991). The use of low *W/CM* ratios in a grout will tend to maximize the density and consequently minimize the porosity. The laboratory studies showed that the amount of superplasticizer could be varied from 0.75 to 1.5% to achieve the desired viscosity that may suit many field applications without a significant impact on setting time (Onofrei et al. 1993). The superplasticizer acts as a retarder, delaying the setting time of the cement paste. However, it was found that the addition of silica fume can offset this effect to a certain degree. The grouts containing silica fume were found to be thixotropic; they retain a low viscosity if agitated or vibrated continuously (Onofrei et al. 1993).

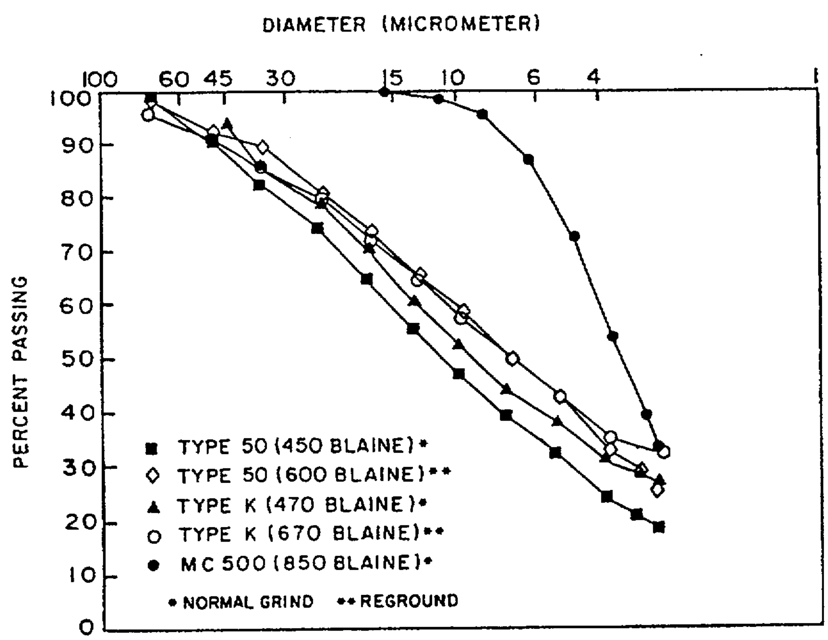

Figure 1. Particle size distribution of cements studied.

Laboratory studies showed that low-viscosity, non-segregating grouts could be prepared using any of the three cement types investigated. The amount of superplasticizer could be varied to achieve the desired viscosity without a significant impact on the setting time. However, the reground SRPC with 10 % silica fume appeared to require slightly less water than either Type K with 19% silica fume , or MC-500 for the equivalent viscosity. Furthermore, SRPC is a widely available material, with its properties well documented in the open literature.

Thus, the reference grout mixture adopted for use in the *in situ* experiments and in the laboratory for the longevity studies consisted of the following:

- 90%, by mass, SRPC (Canadian Type 50) reground to a Blaine fineness of 600 m^2 kg^{-1},
- 10% silica fume (by mass),
- 1% superplasticizer (Na-SNFC) by total mass of cement plus silica fume, and
- water (*W/CM* ratio = 0.4 and 0.6 by mass).

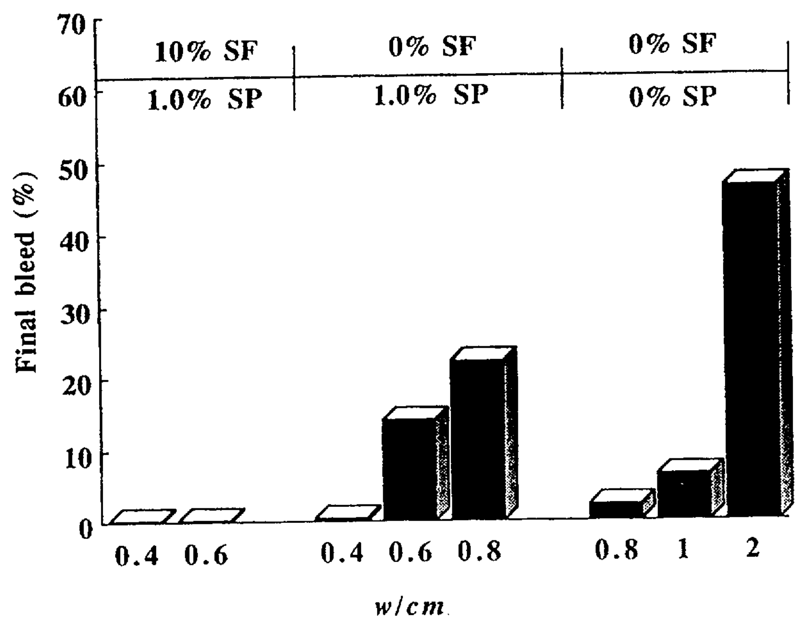

Figure 2. Final bleed observed for various grout mixes.

2 *IN SITU* INVESTIGATIONS

The main purpose of the cement-based grouts materials is to seal fractures in the rock for perhaps tens of thousands of years. Left unsealed, these fractures may contribute to the dispersal of dissolved nuclear waste to the environment. To seal the fractures, the sealing materials are required to have properties such as workability.[4] The effort required to place a cement mixture

4 The workability of cement and concrete is defined (ASTM C 125) as the property determining the effort required to manipulate a freshly mixed quantity of cement or concrete with minimum loss of homogeneity.

is determined largely by the overall work needed to initiate and maintain flow, which depends on the rheological properties of the cement paste. Workability is not a fundamental property of a grout. To be meaningful, it must be related to the type of application and the placement method. Thus, to be workable, the material specifications will vary according to the methodology used for placing the material. The criteria for workability will also vary with the location in the sealing system where the material is to be used. (i.e., shaft seals, fracture-zone seals or disturbed zone).

The process of grouting involves the injection of the grout paste into the rock, which limits any control of the flow direction and travel distances of the grout paste. In rock, the grout will encounter a wide variety of pathways, with possibly a wide range of hydraulic conductivities. While a grout paste should have low viscosity to be injected into fine fissures, and also to increase the distance of penetration into fractures, a high-viscosity cement paste will be preferred to fill wider fractures. Thus, the grout to be used must be selected on the basis of a "best" engineering judgement. Other factors included in this judgement include the requirement for the cement paste not to segregate or sediment once it has been injected. Thus, the development of a "stable" cohesive grout paste is required to limit the amount of bleed or sedimentation. Failure to limit the bleed may result in uncontrolled open channels within the grouted fracture. This will compromise the overall hydraulic barrier efficiency and quality of grouted rock mass.

The time required for the cement paste to achieve its initial set and to finally harden is important relative to its practical use in the field. A quick setting time may be desired when a grout is injected into fractures containing flowing water. However, setting time must be long enough to permit effective injection of a given batch before hardening.

Prior to the *in situ* trials at URL, trial mixing was performed using a full-scale high-shear colloidal mixer to confirm grout mixing and handling properties. The results indicated that the high-shear colloidal mixer was more efficient than the laboratory blenders. Specifically, grout mixed in an industrial blender with 0.75% superplasticizer was too thick or viscous to flow through the Marsh cone.[5] However, the same mix made in the high-shear colloidal mixer had a cone time of 150 s (Table 1).

Field measurements of hydraulic properties of granitic rock at the AECL Research' s URL indicated that grouting will be needed to reduce the hydraulic conductivity of zones of rock to values less than 10^{-7} m s^{-1}, commonly accepted as the limit of cement grouting in conventional engineering practice. The grouting trials took place at the 240 m level, which sits above a fracture zone in the host granitic rock body in which grout was injected. The fracture zone in which the grouting was performed exhibited an initial hydraulic conductivity that ranged from 10^{-5} m s^{-1} to 10^{-7} m s^{-1} and contained groundwater under a static head of roughly 2.0 MPa. The grout was injected through boreholes GH1, GH2, and HC9 (Figure 3).

The grouting plant used in the *in situ* trials at URL consisted of standard equipment used in the grouting industry. Specifically, the plant contained a single-drum high-shear colloidal

5 The Marsh cone "efflux" viscosimeter is a simple field device for measuring the time necessary for 1 L of a grout to pass a 5-mm-diameter orifice located at the bottom of a standard 30° cone

mixer, a holding tank with a paddle mixer and a pump. Full details of the field trials are presented by Gray and Keil (1989). No problems were encountered in the field trials in mixing, handling,

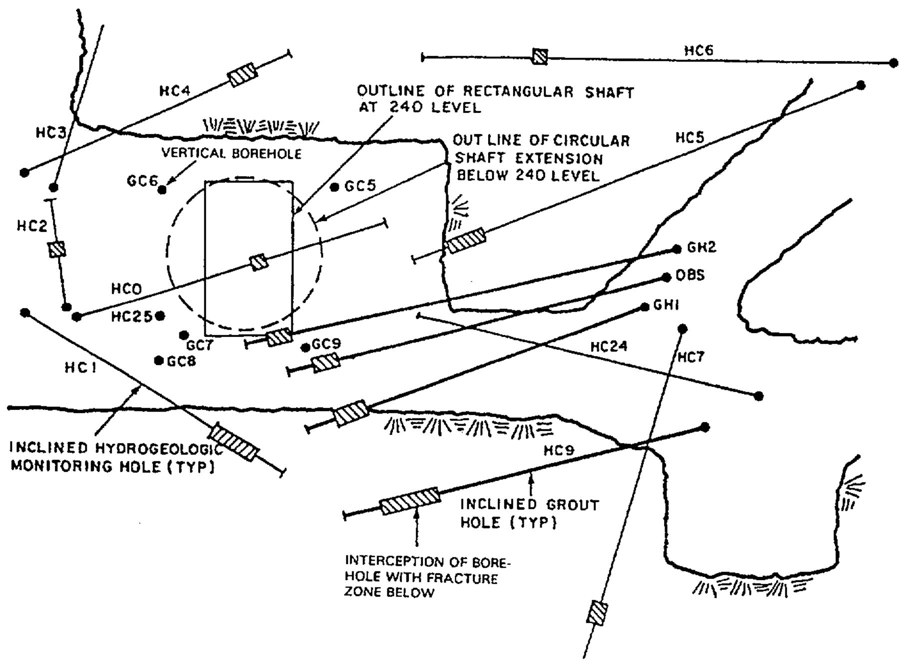

Figure 3. Plan of drill holes for URL grout trials.

or pumping of the grout. It was anticipated that the selected holes would accept only limited volumes of grout. Both boreholes GH1 and GH2 where grouted until no more grout could be injected into the holes ("refusal"). The fracture zone in the vicinity of borehole HC9 was known to be of higher hydraulic conductivity than at either GH1 and GH2, and correspondingly higher grout takes were anticipated. A total of nearly 700 L of grout was injected into the holes, "refusal" was never achieved.

Geochemical monitoring was performed throughout the grouting period. The results obtained were not totally conclusive since mine water, contaminated by cement, had been injected into the formation during the drilling associated with grouting work. However, it was clear that the geochemical signature of the grout injected into the rock was very limited both in lateral extent and in duration. For example, no rise in pH was detected in nearby boreholes except in those where the grout itself had migrated. This indicates that the injected grout exhibited no segregation or bleeding during or after the injection.

Table 1. Comparison of the Marsh cone for mixes performed in a laboratory industrial blender and full-scale high-shear colloidal mixer.

W/CM	Superplasticizer Content (% mass solids)	Marsh Cone (s)	
		Laboratory Industrial Blender	High-Shear Colloidal Mixer
0.6	1.25	42	38
	1.00	47	42
0.4	1.50	68	65
	1.25	84	73
	1.15	132	73
	1.00	200	73
	0.75	No Flow	150

Hydrogeological testing before and after grouting indicated that the hydraulic conductivity of the fracture zone was successfully reduced between one and two orders of magnitude. The hydraulic conductivity was reduced from $>10^{-7}$ m s^{-1} to $< 10^{-9}$ m s^{-1} (Gray and Keil, 1989).

The shaft wall containing the grouted fractures adjacent to grouted borehole GH2 was examined as the shaft was being excavated through the grouted region of the fracture zone. The examination revealed that the grout penetrated fractures with a wide range of apertures. Thin - section analysis of grouted specimens recovered from the excavation showed no evidence of voids or segregation within the grout. The grout was seen to conform totally to the irregular fracture morphology on the rock surface and to entirely incorporate the debris (i.e., crushed and altered rock) in the grouted fracture (Figure 4). Furthermore, the grouted rock samples were examined microscopicaly to determine the degree of penetration of the grout into fine fissures. This microscopic examination revealed that the grout had penetrated fractures with apertures as small as 10 μm (Onofrei et al. 1992).

One of the dominant degradation processes in the numerical modelling being developed for the long-term performance of cement-based grout sealing systems is diffusion, as opposed to mass flow, of dissolved constituents into the adjacent host rock matrix. In the matrix diffusion model, a thin layer of grout is assumed to be surrounded by a large reservoir of connected rock matrix porosity. It is assumed that the portlandite ($Ca(OH)_2$) dissolved instantaneously to its saturation limit, and that the resulting concentration gradient drives the transport of dissolved ions from the grout toward the rock. It is assumed that dissolution of portlandite leads directly to functionality of the seal being lost (Neretnieks, I. 1985, 1987, 1990, Atkinson and Titchell 1985, Alcorn et al). 1992).

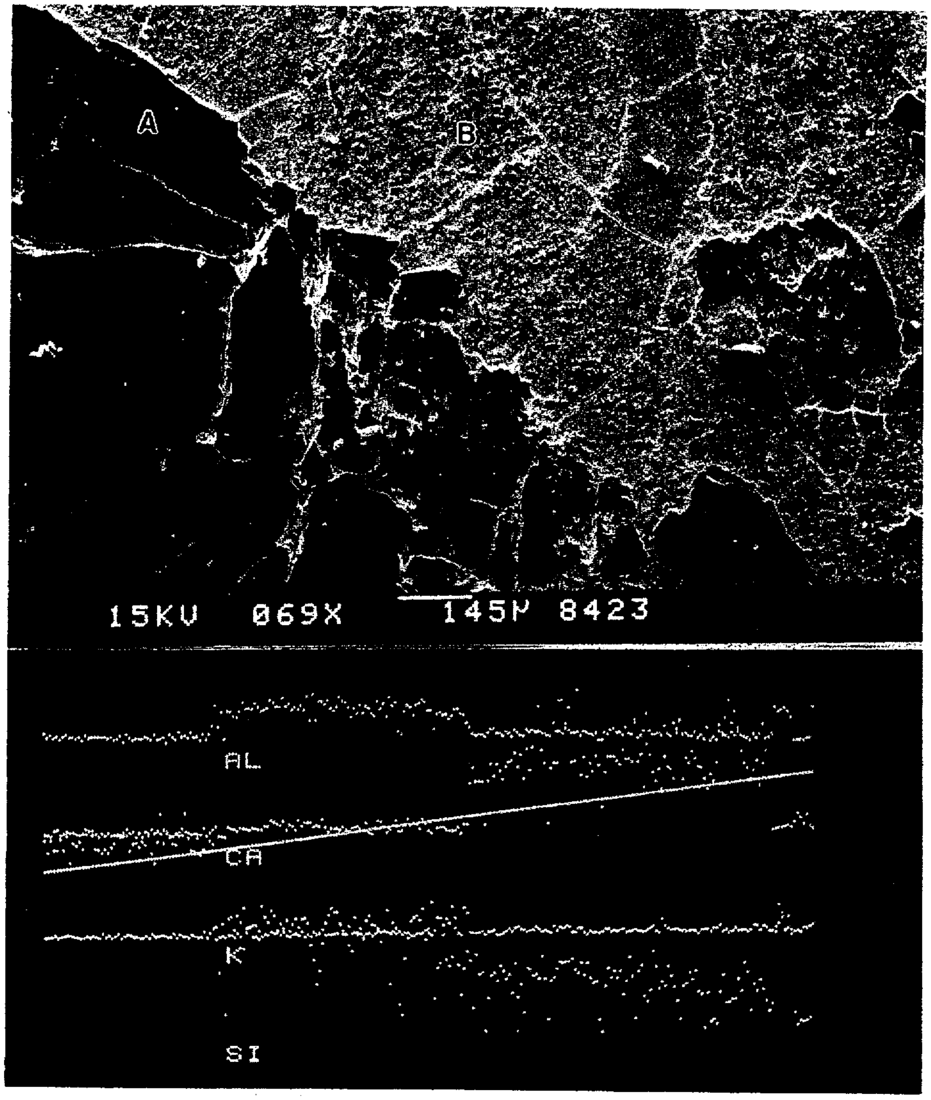

Figure 4. Photomicrograph of the rock/grout interface and X-ray line scans of Ca, Si, Al and K. (A) rock and (B) grout.

Microscopic examination of the granite/grout interface of grouted rock specimens (recovered during the shaft excavation through the grouted region of the fracture zone) more than one year old revealed that no diffusion of any elements had occurred during this interval of time. The spectra for each element (i.e., Ca, Si, Al, K) were collected along the line seen in the electron micrographs of the polished grouted rock specimens (Figure 4). The change in the intensity of the X-ray line scans is very clear at the grout/rock interface. The lack of diffusion during this period may be attributed to; a) the presence of a good bond between the rock and grout and therefore the absence of water at the interface, b) the fracture-filling material, the grout, has a very low hydraulic conductivity, and c) the absence or low amount of portlandite in the grout.

The matrix diffusion model developed as a mechanism for cement degradation employs assumptions that are very conservative. Water must first diffuse into the grout before the portlandite or other cement phases can be dissolved. In high - performance grouts containing both silica fume and superplasticizer, it is reasonable to assume a free calcium content (free calcium comprises calcium in the form of portlandite) of about 1 to 3% of the weight of the cement paste. However, the free calcium content will continue to decrease with the age of the grout, and if sufficient pozzolanic material exists the entire free calcium may be consumed. Furthermore, the bulk porosity of the rock is probably not all connected.

Based on these preliminary observations and the fact that a disposal vault will likely be designed and sited so that the groundwater flow is minimized, the grout could maintain an acceptable level of performance for a long time.

The field data strongly indicate that the high-performance cement-based grout can be injected successfully in the fractured rock using largely conventional grouting practice. To enhance confidence in this conclusion, a larger field experiment is planned to be carried out at the URL in Canada in early 1995.

3 LABORATORY INVESTIGATIONS

Because the most likely mechanism for waste migration is aqueous transport, most sealing programs are seeking sealing materials that have comparatively low hydraulic conductivities, and are chemically and physically compatible with the environment in which they are emplaced. The low hydraulic conductivity is required to restrict the access of the water to the waste and to contain dissolved waste near the emplacement area. Assigning this functional purpose for sealing materials, it is logical to assume that a chemical and physical compatibility does not necessarily require that the materials remain unchanged with time. Instead, they may be viewed as having properties that resist increases in hydraulic conductivity and that their interaction with groundwater and the host rock minerals will not increase the overall conductivity of the seal system. Because of the long-term hazardous nature of the radionuclides of concern, the seal must perform acceptably for thousands of years.

Given these concerns regarding the conductivity and the longevity of the sealing materials, laboratory investigations were undertaken to develop a conceptual model for the morphology and long-term performance parameters of high-performance grouts comprising reground sulphate-resisting portland cement (SRPC), silica fume superplasticizer and water. The hydraulic conductivity/porosity relationship for the material was determined to improve our understanding of the new high - performance grout and to provide information on the ability of the grouts to self seal if they become fractured in a disposal setting. Leach tests were carried out to provide details of solid/water reaction processes to improve our conceptual models of the material characteristics.

3.1 Hydraulic properties

It is generally known that the durability of hardened cement exposed to aggressive aqueous environments is related to permeability and thereby to the pore structure of the hardened cement. Permeability is an indicator of the ease of passage of liquids and gases in a grout. The permeability of hardened cement can be reduced by decreasing the porosity, the maximum pore size, and the extent of interconnectedness of the pores. Reducing the *W/CM* ratio is an effective means of decreasing the permeability. However, significant decreases in the *W/CM* ratio can result in difficulties with placing and consolidating the cement. Laboratory investigations showed that these difficulties can be overcome by using water reducers (such as superplasticizer) and adding pozzolanic materials (Onofrei et al. 1993). The addition of both superplasticizer and pozzolanic material (such as silica fume) alters the microstructure of the hardened grout that forms during the hydration.

The permeability coefficient of a liquid flowing through a porous solids can be expressed by the following form of the Dary equation:

$$Q = -\frac{kA}{\mu}\frac{dP}{dx} \tag{1}$$

where Q is the volumetric flow rate (m s^{-1}), A is the total cross-sectional area (m^2), μ is the viscosity of the liquid, P is the component of pressure capable of causing flow, and k (m s^{-1}) is the permeability coefficient, which is characteristic of the porous medium and independent of the fluid provided that the fluid flow is laminar.

When considering water flow in cement, it is common to calculate the hydraulic conductivity, K, from:

$$Q = -AK\frac{dh}{dx} \tag{2}$$

where dh/dx is hydraulic gradient across a specimen, and is dimensionless. For clarity, dh/dx is sometimes expressed as metres of water head per metres of distance (m/m).

The hydraulic conductivity (K) and the permeability of grouts based on the developed reference high - performance grout were determined for grout mixes with a *W/CM* ratio (by mass of the grout) between 0.4 and 0.8. The grout specimens were cast in 75-mm-diameter, 150-mm long plastic cylinder and hardened for 24 h before demoulding. The specimens were cured at room temperature in saturated lime-water for 49 d. After curing, the specimens were centrally and axially cored to produce a hollow cylinder with a wall 25 mm thick.

The hydraulic conductivity tests were carried out in a specially constructed constant-head hydraulic conductivity cell capable of sustaining hydraulic pressures of up to 10 MPa (Onofrei et al. 1992). The water was allowed to flow radially through the specimens either convergently (with the exterior pressure on the specimen higher than the interior) or divergently. Thus, K was measured with the grout specimens either in circumferential compression (convergent) or circumferential tension (divergent). The full description of the experimental design and methods has been given elsewhere (Onofrei et al. 1993).

Results from the compressive tests series for grouts with no silica fume and with 10% silica fume revealed that both grouts tend to store some water over the range of time and pressures investigated. At hydraulic pressure differences ≤ 4000 kPa (gradient (i) ≤ 16 500 m/m), specimens with no silica fume took up water from both their upstream and downstream faces. Similar behaviour was observed for specimens containing silica fume (Onofrei et al. 1993). Hydraulic pressure differences equal to or greater than 8 600 kPa (i > 35 000) were needed to obtain measurable through-flow water at the outflow end of the specimen containing silica fume (Figure 5). The results showed there was no measurable through-flow water from grouts containing silica fume at $W/CM = 0.4$ with the specimen in tension or when the specimen is subjected to a compressive radial flow. Only under very high hydraulic gradients, which could be tolerated under compressive stress conditions, did water tend to flow through the specimens.

The data show that adding silica fume decreases the apparent hydraulic conductivity of the grout. Investigations on the permeability of cement-based grouts have shown that the reference high-performance grouts containing 10 % silica fume have very low hydraulic conductivity ($<10^{-15}$ m s^{-1}) (Figure 6). Also, the data show that the grouts with W/CM of 0.4 and 0.6 are practically impermeable under hydraulic gradients (i) up to 36 000. In a sealed disposal vault, i may be as low as 10^{-2} (Chan 1989). Water did not pass through the grout specimens at these low gradients. Several factors were considered to be responsible for this resistance to the through-flow of water. These included entrained air in the pore spaces, morphological transformation of hydrated cement minerals and the lack of equilibrium flow conditions (Onofrei et al. 1993). Results from tests conducted under deaerated conditions revealed that entrapped air is not the reason for the resistance of the material to the through-flow water.

The examination of the microstructure of the hardened grout indicated that the addition of both silica fume and superplasticizer promoted the development of a dense homogeneous microstructure with extremely fine porosity. The examination revealed the absence of larger pores than 0.1 μm. The resistance to the through-flow of water was attributed to the absence of large pres (>0.1 μm) or interconnected porosity. The small pores do not make a significant contribution to permeability.

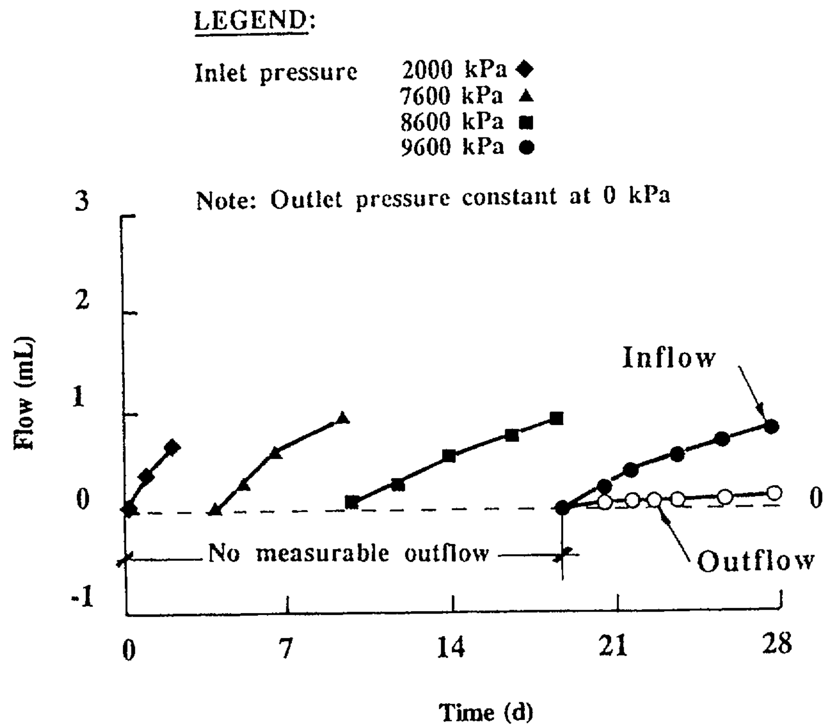

Figure 5. The effect of hydraulic pressure on time-dependent flow through grout under compression (*W/CM* = 0.4 and 10% silica fume).

The absence of such pores should have significant effects on the permeability and consequently on the durability of the developed grouts. In this respect, Mehta and Manmohan (1980) proposed a pore diameter of 1 μm as a somewhat arbitrary dividing point between "large" pores, which contribute most to permeability, and "small" pores, which are much less significant. The properties of the material are recognized to originate from its internal microstructure, and these properties can be modified by making suitable changes in the structure of the material. The low hydraulic conductivity ($<10^{-14}$ m s^{-1}) of the developed high-performance grout can be related to the grout's dense microstructure and the lack of pores with diameters larger than 0.1 μm. The results suggest that the pore size distribution is more important than the total porosity in determining permeability. Furthermore, the results indicated that the microstructural characteristics of the material changed while the material was in contact with water. The observed decreased in the hydraulic conductivity with time was associated with pore refinement caused by the increase in the volume of solids as the result of the increase in the degree of hydration (Onofrei et al. 1993).

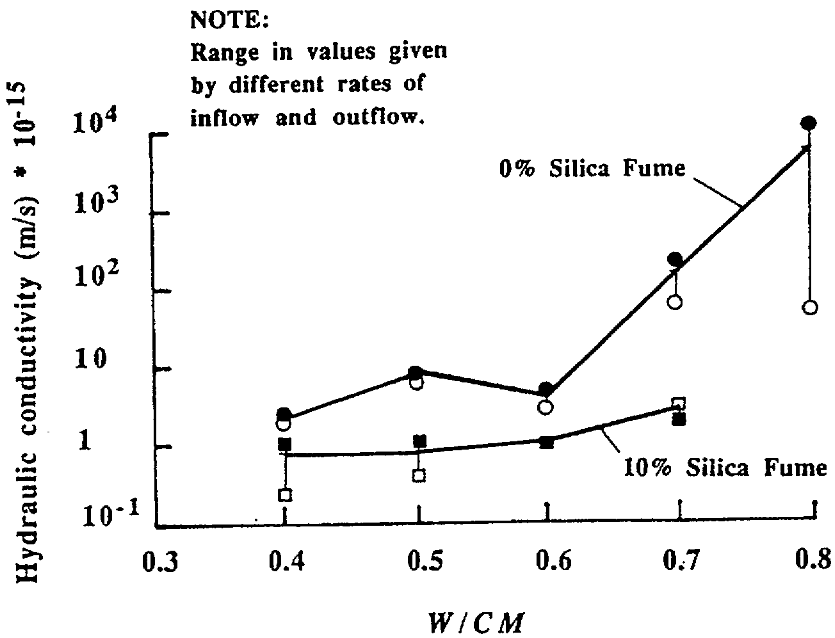

Figure 6. Effect of silica fume and *W/CM* ratio on the hydraulic conductivity of grouts (i ≥ 28 500 for grouts with 0% silica fume and i ≥ 35 000 for grouts with 10% silica fume).

3.2 LEACHING PROPERTIES

Leaching by water involves the penetration of the grout by water or aqueous solutions, the dissolution of soluble constituents of the hydrated cement paste, and transport of the dissolved species to the surrounding water. The depth of penetration of groundwater into cement matrix will be largely controlled by the permeability of the hardened cement matrix and the hydrostatic pressure of the water. The extent of leaching will depend on the chemistry of the water, the amount of soluble constituents of the hardened cement paste, the concentration gradients in the respective fluids, the flow rate of the groundwater and temperature. When the ground water flows, a dissolution equilibrium may not be attained and leaching can be a continuing process.

The cement-based grouts evaluated in this study were the reference high-performance grout mixed at *W/CM* ratios of 0.4 and 0.6. For the laboratory tests, the grout ingredients were mixed in a temperature-controlled room with an ambient temperature of about 20 °C. The mixes were prepared in a 20 L paddle mixer followed by mixing in a high-speed industrial blender. After mixing, the grouts were cast in cylindrical moulds with diameter and length of approximately 50 mm and 100 mm respectively. The specimens were struck from the mould after final setting had occurred, and the specimens were cured in a saturated lime ($Ca(OH)_2$) solution for appropriate periods.

Different types of leaching test methods were applied to the study of the cement-based grout materials (Table 2).

Table 2. Experimental parameters of tests methods.

Test Method	Leachant	Temperature (°C)	Flow-Rate (mL/d)	Leaching Time (d)
Static Test	DDW, SCSSS, WN-1	10-100		1-32
Continuous Low-Flow-Rate Test	DDW, WN-1	25-85	12	1-28
Continuous High-Flow-Rate Test	DDW, WN-1	25-100	240	1-28

Laboratory tests were carried out to evaluate the effects of these variables on the leaching mechanisms. The full description of the designs and methods has been given elsewhere (Onofrei et al. 1993). Since the grout itself is complex in structure and composition, distilled deionized water (DDW) was used as a reference leachant because it provides the simplest environment in which to study the grout leaching mechanisms. However, since a major goal of this investigation was to evaluate grout leaching performance in the realistic environment, other reference leachants other than the DDW were used including Canadian reference groundwaters, Whiteshell N-1 Groundwater (WN-1) and Standard Canadian Shield Saline Solution (SCSSS). They represent the range of groundwater compositions expected to be encountered at depths down to 1000 m in the Canadian Shield or similar granite bodies.

The dependence of the leaching behaviour of grouts on the extent of interaction with the leachant and on the resulting changes in leachant chemistry were evaluated by considering two extreme cases. In one approach, the grout was leached under static conditions, allowing the leachant in the grout/leachant system to approach saturation with the major grout elements such as Ca^{2+} and Si^{4+} leached out of the grout. The other approach involved dynamic conditions where the accumulation in the leachant of the species leached out of the grout is too small to alter the composition of the leachant in a sufficient measure to affect its reactivity toward the grout.

Particular attention was paid to the leaching of the Ca^{2+} and Si^{4+}. These elements were chosen since they reflect the leaching characteristics of two major phases in cement paste - the $Ca(OH)_2$ and calcium silicate hydrates (CSH)

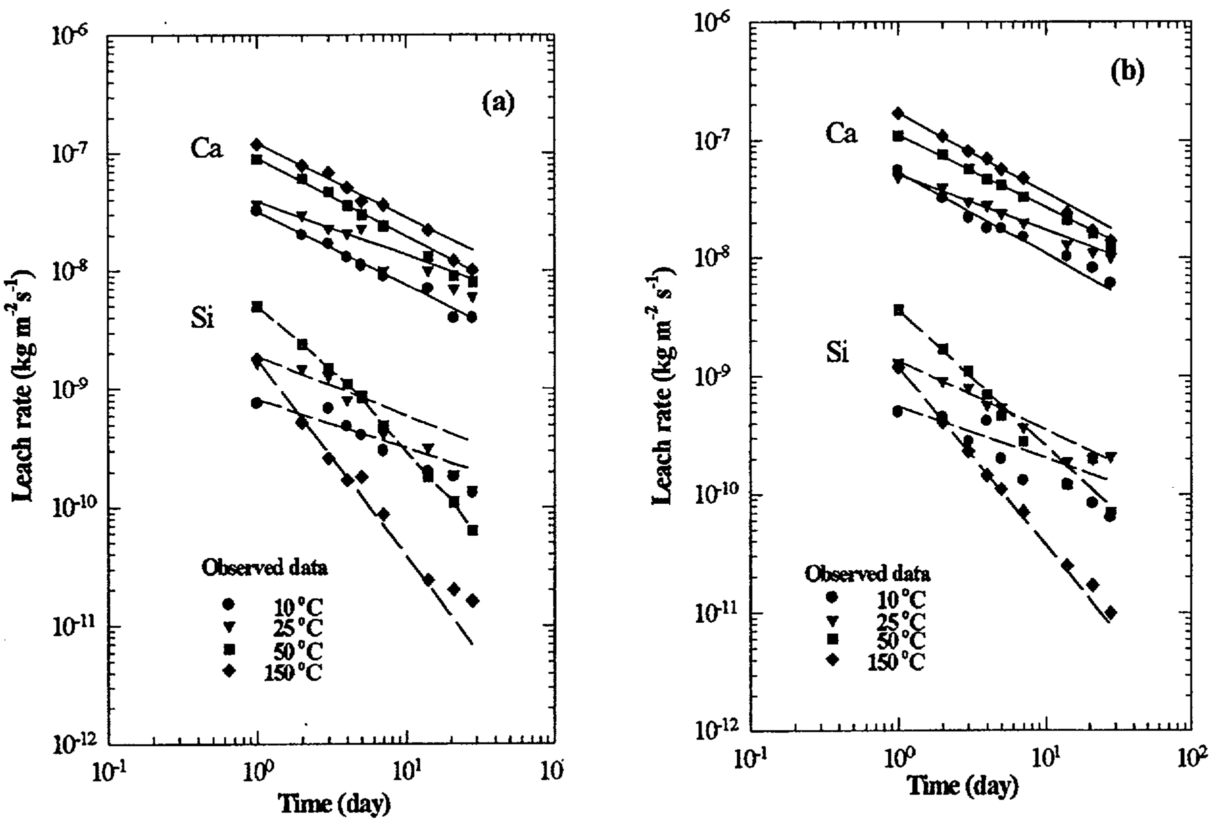

Figure7. Effect of time and temperature on the leach rate of Ca^{2+} for the reference grout based on Type 50 cement mixed at : a) W/CM = 0.4 and b) W/CM = 0.6 and leached in DDW under static conditions.

Figure 7 shows the rate at which Ca^{2+} and Si^{4+} were leached from the grout mixtures of the reference high-performance grout mixed at *W/CM* = 0.4 and 0.6 when reacted with DDW in static leach tests .

The curves in Figure 7 were obtained by fitting lines through the average values of the measured leached rates. The results show that there is an instantaneous release of both Ca^{2+} and Si^{4+} which increased directly with temperature. In all tests the initial leach rates of Ca^{2+} for the reference grout mixed at *W/CM* = 0.6 were higher than the leach rates of Ca^{2+} for the reference grout mixed at *W/CM* = 0.4 . It may be suggested that these results reflect an increase in the quantity of portlandite ($CaOH)_2$) in the grout as the *W/CM* ratio increases; this is the result of an increased degree of cement hydration. Furthermore, the *W/CM* ratio is an important factor in controlling the porosity of hydrated cement paste and thus the permeability. It is generally considered that, as a first approximation, the *W/CM* ratio and the porosity of hardened grout are directly related. The results show that the density and total porosity decreased and increased respectively over the investigated range of *W/CM* ratios as *W/CM* changed. As the *W/CM* ratio is increased (i.e. *W/CM* = 0.6), the porosity of hydrated grout increased, leading to the development of a more open network. The increased porosity is expected to increase the penetration of the grout by aqueous solutions and the rate of transport of dissolved species to the surrounding solution.

Differential thermal analysis (DTA) and mercury intrusion porosimetry (MIP) analysis performed on the samples before performing the leach tests provided support to the conclusions that a higher initial $Ca(OH)_2$ content and the presence of higher porosity contributed to the higher release of Ca^{2+} from the reference grout mixed at the higher *W/CM* = 0.6 ratio (Onofrei et al. 1993).

As expected, the leach rates were found to decrease with leaching time. The observed decreases in the leach rate may indicate that either the concentration of the element approaches the solution saturation limits or that an altered dissolution-limiting surface layer is formed. The calcium solubility limit in water in equilibrium with $Ca(OH)_2$ has been reported to be approximately 0.02 mol L^{-1} (Atkinson and Hearne, 1984). At a value of 10^{-7} mol L^{-1}, the bulk solution concentration level of calcium measured in static leach tests is significantly below this equilibrium value (Onofrei et al. 1991). Since the limited leach rates cannot be entirely attributed to the Ca^{2+} concentration approaching solubility limits, the lower values measured suggest the formation of a dissolution-limiting reaction layer. SEM/EDX examination of the leached grout surfaces confirmed the formation of the reaction layer at all temperatures. The XRD analysis indicated that the composition of the layer was a mixture of $Ca(OH)_2$(portlandite) and $CaCO_3$ (calcite) at low temperatures, becoming predominantly $CaCO_3$ at high temperatures (Onofrei et al. 1993). In addition to the solution concentration and the surface reaction layer, changes in the microstructural characteristics of the grout have been shown to contribute to the observed decreases in the leach rates. An MIP analysis of the grout specimen leached for various lengths of time show that the microstructural characteristics of the grout changed during leaching. Leaching was accompanied by decreases in the mean pore size and total pore volume (Figure 8).

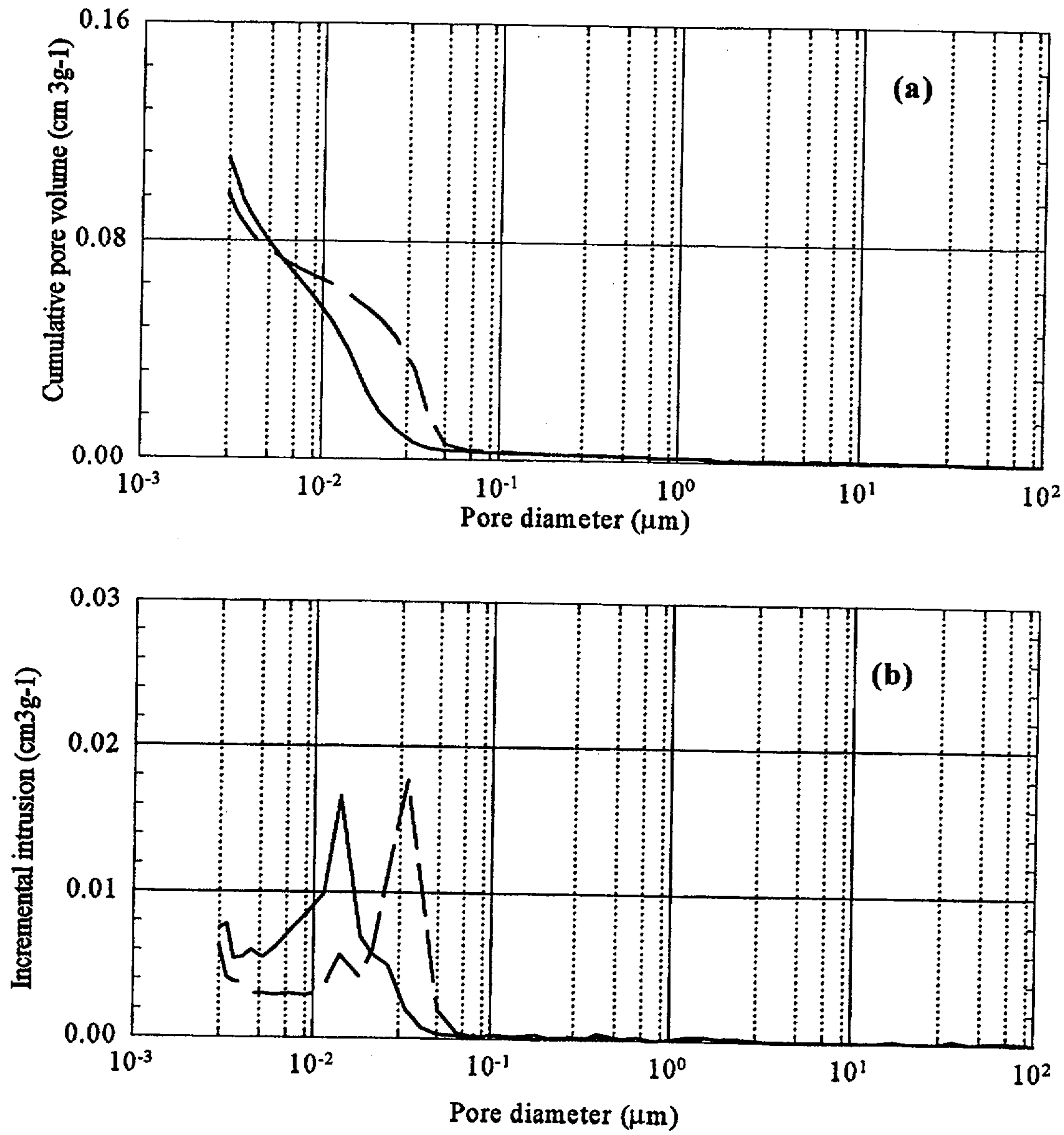

Figure 8. The effect of leaching time (dashed line - after 1 day, continuous line - after 32 days) on the pore size distribution for the reference grout mixed at W/CM = 0.4 and leached in DDW at 10 °C under static conditions.

The degree of change in the microstructural characteristics was found to be a function of the initial *W/CM* ratio, temperature, initial porosity and the type of cement. The porosity changes cause changes in the grout permeability and diffusivity, therefore altering the rate at which water flows through the grout and the dissolution of soluble constituents of the hydrated cement paste and the transport of the dissolved species to the surrounding water.

The leach rates of Ca^{2+} in synthetic groundwaters at temperatures of 10, 25, 50 and 85 °C are summarized in Figure 9. The values of the data points represent the average of six replicants. The changes in Ca^{2+} concentrations in solution were calculated by subtracting 4.85×10^{-2} mol L^{-1} or 3.5×10^{-1} mol L^{-1}, the initial concentrations of calcium in the original WN-1 and SCSSS groundwater respectively, from the measured concentration of Ca^{2+} in solution.

When the reference grout was mixed at low *W/CM* (0.4) and leached in WN-1 synthetic groundwater, the variations in leach rate of Ca^{2+} with time differed considerably from those that occurred when the grout was leached in DDW. Initially, calcium had negative leach rates (Figure 9a). The calcium present in the initial leachant (4.84×10^{-2} mol L^{-1}) was removed from solution and deposited on the bottom of the leaching cell and on the surface of the leached specimen.The calcium concentrations in the reference grout mixed at a high *W/CM* (0.6) ratio follow a similar trend, except for the system at 25 °C and 85 °C for which Ca^{2+} was initially leached from the specimens (Onofrei et al. 1993). The rate at which the Ca^{2+} was removed from solutions depended on the grout composition, temperature and time. However, the Ca^{2+}concentrations in leachants increased with time in all the leaching experiments and tend to approach the initial concentration value (4.85×10^{-2} mol L^{-1}) in the original WN-1 groundwater. The extent of Ca^{2+} leaching was less in WN-1 groundwater than in DDW.

The initial decrease in Ca^{2+} concentrations in solution are attributed to the precipitation of phases with which the groundwater is supersaturated (i.e., $Mg(OH)_2$, $CaCO_3$, $Ca(OH)_2$). Groundwater from granitic rock contains significant levels of dissolved sodium, calcium, silicon, magnesium and other ions. Solution equilibria calculated for groundwaters using the computer program SOLMNQ (Goodwin and Munday 1983) have shown typical groundwaters to be supersaturated with respect to several minerals such as calcite, portlandite, brucite, clays (sepiolite and illite) and gypsum. As soon as the hardened grouts are immersed in water, the surrounding solution becomes strongly alkaline (pH > 11.5). The high pH of the aqueous phase will act as a precipitating agent for the phases with which the groundwater is saturated. The XRD and SAM analyses of the precipitates on the surface of the leached specimens and on the bottom of the leaching cell provided evidence to support this hypothesis. The precipitate on the base of the leaching cell was identified by XRD as a mixture of $Ca(OH)_2$ and $CaCO_3$. The composition of the crystalline surface layer on the leached specimen consisted of brucite, $Mg(OH)_2$.

The low leach rates in WN-1 groundwater may be attributed to two possible causes. The precipitate layer formed at the surface of the leached specimens can act as a diffusion barrier. In addition to microstructural effects, the formation of the precipitate layer on the surface of the leached specimen can plug the pores on the cement surface, preventing and/or reducing the amount of water penetrating the hardened grout during leaching and therefore decreasing the leach rate. Alternatively, leaching involves the penetration of grout by aqueous solutions, and concomittent densification of the microstructure decreases the permeability of grouts, and so decreases the leach rate. The MIP analysis of the grout specimen leached for various length of time in WN-1 groundwater confirmed that leaching is accompanied by a densification of microstructure (Onofrei et al. 1993).

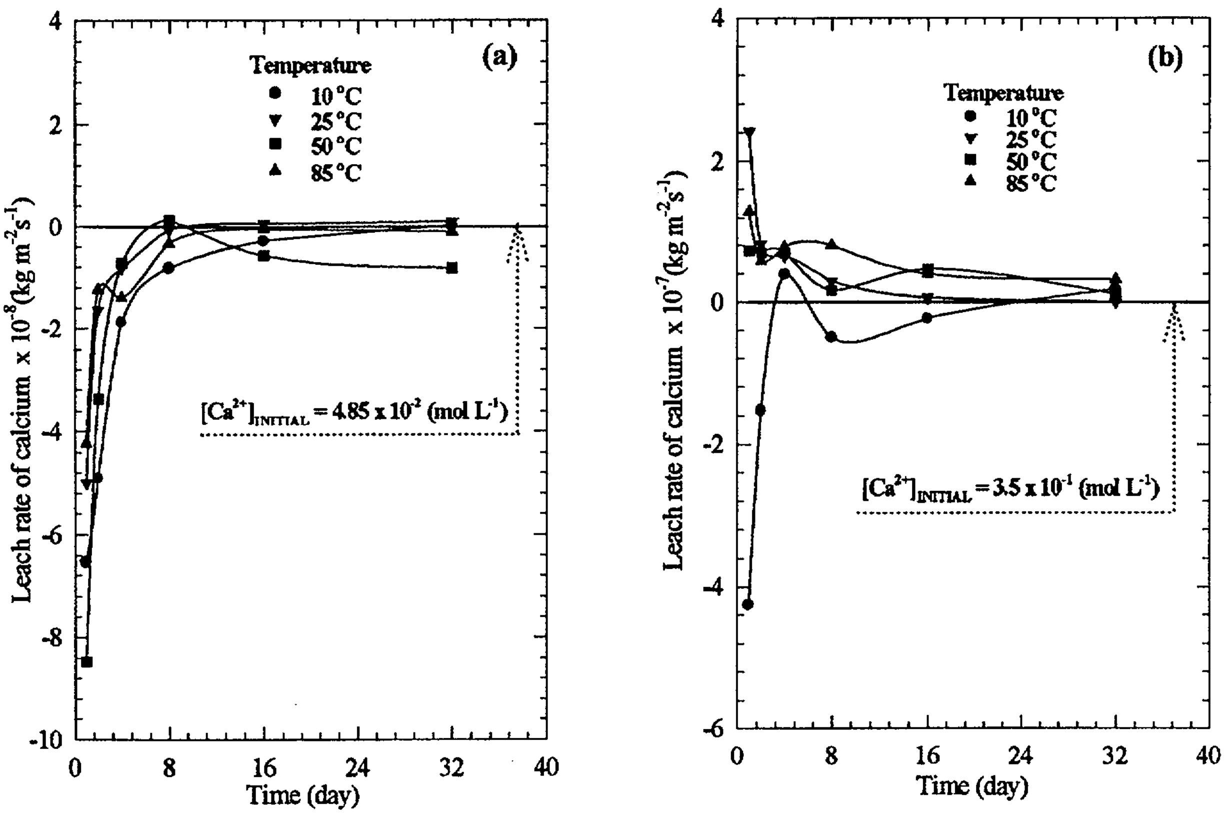

Figure 9. The effect of leaching time and temperature on the leach rate of Ca^{2+} for the reference grout mixed at *W/CM* = 0.4 and reacted with: a) WN-1 and b) SCSSS groundwaters under static conditions.

Initially, the SCSSS synthetic groundwater enhances Ca^{2+} removal from the grout at leach rates that exceed those occurring in DDW and WN-1 synthetic groundwater (Figure 9b). However, as in DDW and WN-1 solutions, the leached Ca^{2+} concentrations decrease with exposure time and approaches steady state after approximately 14 d of leaching. In the DDW it appears that the overall rate of grout reaction slows as the Ca^{2+} concentration in pore water in the grout approaches that of Ca^{2+} in solution. The situation is more complex in SCSSS groundwater, but it appears that a mechanism similar to that in DDW may be operational. The differences in chemical potential between Ca^{2+} in the unleached grout and the Ca^{2+} in solution may be controlled by an alteration phase, perhaps $CaCO_3$, $CaSO_4$ and CSH. This difference in chemical potential could lead to continued leaching of the grout. However, a decrease in the reaction rate of grouts with SCSSS with time was noted. SEM/EDX analyses of leached specimens revealed the formation of a surface precipitate layer identified as $Mg(OH)_2$. Alternatively, the actual concentrations of Ca^{2+} in solution may be controlled by the grout leachability (supply) and phase precipitation (consumption).

Any interpretation of the data from the tests involving groundwater under dynamic conditions is complicated by the presence of high concentrations of the elements of interest (Ca^{2+} and Si^{4+}) in the leachant, a continuous supply of these elements to the leach cell and their presence at the surface of the tests specimens, which makes it difficult to resolve those changes attributable solely to the grout. However, the results indicate that the diffusion across a surface layer controls the leaching processes, as in the static leach tests. The leach rates were found to be higher than in leach tests performed with similar leachants under static conditions. The reason for the apparent increase in leach rates can be related to the probable increase in the Ca^{2+}concentration gradient through the grout/solution interface, which may increase the driving forces for leaching from the grout. A full analysis of the results from the dynamic leach tests has been given elsewhere (Onofrei et al.1993).

4 AUTOGENOUS HEALING PROPERTIES OF HIGH-PERFORMANCE GROUTS

One of the possible functions of cement-based sealants is to seal fractures for long time periods if those fractures may contribute to the dispersal of the dissolved nuclear waste to the environment. To accomplish this objective, the sealant must have acceptable low hydraulic conductivity (e.g., $<10^{-10}$ m s^{-1}). It is evident that the presence of any defects (cracks, capillary pores) in the grout structure has the potential to provide a more rapid transport pathway for the advection or diffusion of radionuclides. The presence of these defects could impair the performance of grouts as a barrier to radionuclide migration. If self-sealing can be assured, particularly for the modified high-performance cement-based grouts containing silica fume and superplasticizer in the chemical conditions likely within an underground disposal vault, the cracks are of much less significance.

Laboratory investigations on the permeability of the high-performance grouts used in the laboratory and field trials showed that the high-performance grouts are practically impermeable ($k < 10^{-16}$ m s^{-1}) under hydraulic gradients up to 36 000 m m^{-1}. Moreover, the material's porosity was found to decrease with time due to continued hydration, precipitation and associated reactions.

The problem of assessing the ability of high-performance cement-based grouts to self-seal was investigated using two approaches: bulk grouts with imposed porosity (compacted granulated hardened grouts) and pre-cracked thin film of grouts (Onofrei et al. 1993). In both cases the self-sealing capabilities of the high-performance grouts were investigated with water flowing through the grout.

Autogenous sealing was studied by evaluating changes in crack sizes, pore structure (decrease in pore radius and volume of pores) and changes in the rate of water flow through the cement-based grouts. The full description of the experimental designs and methods has been given elsewhere (Onofrei et al. 1993).

For the first approach, the reference high-performance grout with W/CM = 0.4, 1% superplasticizer and 10% silica fume hardened for 28 d and 730 d was mechanically crushed and granulated. Cylindrical porous grout specimens were formed over a range of densities by statically compacting the granulated material directly in a permeability cell. The hydraulic conductivity was measured with the grout specimen at a constant head of 0.5 m. The water flowed upwards, vertically along the longitudinal axis of the specimen. This method provided specimens chemically identical to, but with higher porosity than, the reference high-performance grout used in the laboratory studies and in the *in situ* trials.

In the second approach, the self-sealing has been observed in terms of changes in the crack size of a pre-cracked thin film using the cone-in-cone apparatus. This procedure was intended to simulate cracking and separation of the grouts in fractures with thin apertures. Pusch et al. (1989) and Onofrei et al. (1989, 1993) describe the design of the experimental method.

Several series of permeability tests were made to evaluate the effects of such factors as percolating time, compacted density, curing time (age of the grout) and compacted size fraction on the hydraulic conductivity of grouts with imposed porosity.

The hydraulic conductivities, k, of granulated hardened reference grout compacted at densities, ρ, between 1.50 and 1.60 Mg m^{-3} are shown in Figures 10a and 10b. The results show considerable decreases in the hydraulic conductivity with permeation time.

The hydraulic conductivity of the recompacted grouts at the start of the tests ranged between 10^{-6} m s^{-1} and 10^{-7} m s^{-1}. The k was found to decrease two to three orders of magnitude after the grouts were in contact with percolating water for periods up to 230 days. The reason for the observed decreased in hydraulic conductivity were considered to be mostly the continued hydration and/or associated reactions.

It is suggested that, as a result of crushing the hardened grout, it is likely that microcracks are introduced in the hydrated layer which was formed during the hydration reaction around the cement grain. Thus, the residual unhydrated cement within the particles will be exposed to the permeating water. This led to renewed development of hydration processes. As the water moves through and reacts with the grout, the grout's overall porosity and pore structure are altered due to chemical reaction with percolating water. The porosity changes cause changes in grout conductivity, thereby altering the rate at which water flows through the grouts. With the increase in degree of hydation, the hydraulic conductivity decreases rapidly because the volume of the hydration products (i.e, CSH gel) is much larger than the volume of unhydrated cement. It was reported that the gross volume of the CSH gel is approximately 2.1 times the volume of unhydrated cement (Neville 1981). As the water moves through and reacts with the grout, the grout's overall porosity and pore structure are altered. The porosity changes cause changes in the grout's conductivity, thereby altering the rate at which water flows though the grout.

The hydraulic conductivity of the recompacted grouts was found to be affected by the properties of the starting granulated grout. It is clear that both the rate with which the k decreases with time and the final value of k were significantly affected by the age (curing time) of the grouts. The decreases are faster and more pronounced in younger grouts (i.e., 28 d curing time, Figure 9b). It required only ~ 160 d for the recompacted grout made from grout cured for 28 d to reach a k value in the range 5×10^{-10} m $s^{-1} < k < 7 \times 10^{-11}$ m s^{-1}. The k of the recompacted grout from older grouts (2-year curing time Figure 10a), for equivalent percolation time, was in the range of 2×10^{-8} m $s^{-1} < k < 1.2 \times 10^{-9}$ m s^{-1}.

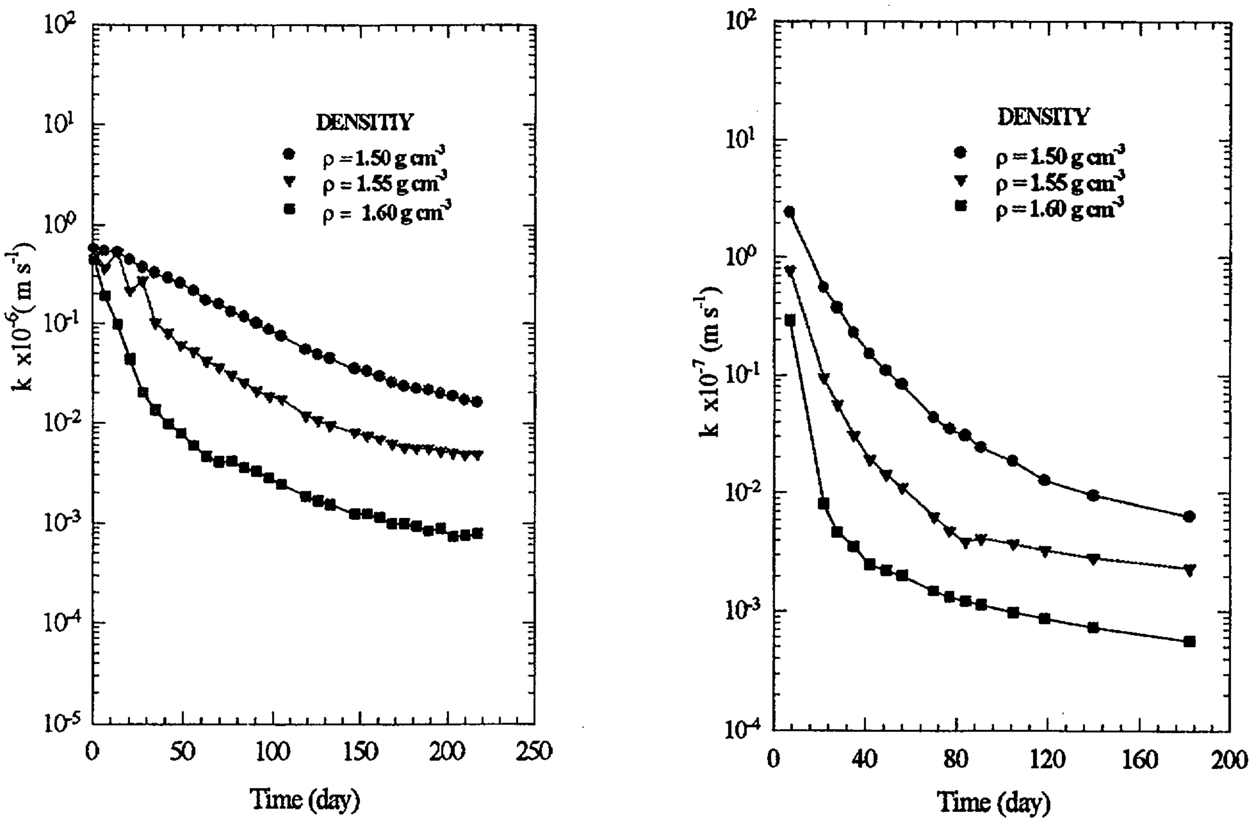

Figure 10. Changes in the hydraulic conductivity with time of recompacted reference high-performance grouts with *W/CM* = 0.4 made from 50% 1.18 mm and 50% 0.3 mm fractions from grout cured for: a) 730 d and b) 28 d.

An indication that the decreases in hydraulic conductivity is accompanied by changes in the pore structure is given by the results from MIP (Figure 11). The data show that, over the period of the test, the proportion of very large pores present (>1 μm) decreased and the proportion of the smaller pores (<1 μm) increased. In all tests, it was found that the pore structure became finer as the permeating time increased. Moreover, the compacted specimen with the largest decreases in hydraulic conductivity exhibited the largest changes in the pore size distribution (Figure 10, ρ=1.60 Mg m^{-3}). The differences in the volume of large pores in the compacted grout specimens, although difficult to relate directly with hydraulic characteristics of the compacted grout, are nevertheless reflected in the hydraulic conductivity of corresponding compacted grouts. The specimens with higher hydraulic conductivity tended to have higher volumes of large pores (>1 μm).

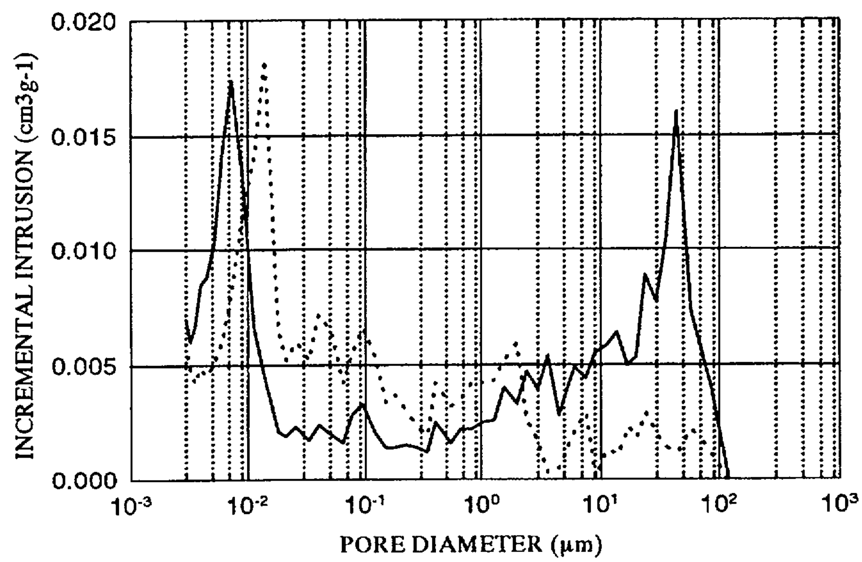

Figure 11. Pore size distributions of compacted granulated reference grout with *W/CM* = 0.4 and two particle sizes (Φ = 1.18 mm and Φ = 0.30) compacted at ρ = 1.60 Mg m^{-3}. (Continuous line-before tests. Dashline after the tests).

SEM analyses of the surfaces of fresh fractures in the specimens at the termination of the hydraulic conductivity tests revealed that a massive fibrous phase had formed on the surface of some grout grains and in the available pore spaces between the grout grains. EDX analyses of the fibrous phase showed it to contain Ca, Al, S and some Si. The material was identified with X-ray diffraction as ettringite ($C_3A.3CaSO_4.32H_2O$). The ettringite formation is the result of the reaction of C_3A with gypsum and water. The decreased hydraulic conductivity and the densification of the microstructure (reduction of the large pores >1 μm) can be also explained on the bases of ettringite formation.

Therefoere, both the increase in the quantity of the hydration reaction products and the formation of ettringite could contribute to the changes in the pore structure and consequently to the decrease in hydraulic conductivity of the grouts.

In all tests, the hydraulic conductivity did not appear to approach a steady state during the tests. The long time required to achieve equilibrium in the hydraulic conductivity tests may reflect a complex pore structure in the compacted granulated hardened grouts as well as the continuation of hydration and/or associated reactions (e.g., precipitation of ettringite).

From the hydraulic conductivity tests carried on the monolithic grout with high and low water content, analysis of the material observed to form in the pore space available revealed $Ca(OH)_2$ and amorphous CSH. These materials decreased the capillary porosity, decreased the total pore volume and increased the fraction of fine porosity. No pores larger than 0.1 μm remained after the test. Thus, a mechanism by which $Ca(OH)_2$ and microporous gels develop and grow into the larger pores, shifting the pore size distribution towards the finer pore size and completely blocking up some of the larger pores, can be inferred.

The material observed to form in the pore space available and to bond the grout grains in the compacted hardened grout specimens exposed to DDW in the hydraulic conductivity tests carried out in air was identified as ettringite. The observed densification of the microstructure and the decrease in the hydraulic conductivity of the compacted granulated hardened grouts was attributed to the ettringite formation.

Experimental work indicated that self-sealing also occurs in thin films of hardened grouts (Onofrei et al. 1993). Figure 12 shows a typical result from tests performed on a pre-cracked thin film (~1mm) of a grout consisting of a fine slag-cement (MC-500) mixed with 1% superplasticizer at W/CM = 0.7 and cured for 7 days in the cone-in-cone apparatus (Onofrei et al. 1993).

In this geometry the water was flowing at the surface of the grout and not through the crack itself. The crack was ~ 400 μm and the aperture ranged in places from 27.5 μm to 90 μm. The microscopic examination performed during the tests indicated that healing was initiated as soon as the grout came in contact with water. Most of the narrow areas of the crack self-seal completly after only 27 d in contact with flowing water. The crack was sectioned into a series of discontnuous smaller cracks with various apertures. The results showed that the rate of reduction in crack size appeared to be primarily dependent on the initial aperture.

The infilling material formed in the crack was identified as a mixture of CSH, $Ca(OH)_2$ and traces of calcite ($CaCO_3$). The infill material may have formed as a result of either continuous hydration reaction or dissolution of $Ca(OH)_2$ in the grout and increasing the concentration of Ca^{2+} in the crack and consequently recrystallization of $Ca(OH)_2$ and formation of $CaCO_3$.

The results indicate that self-sealing occurs in both thin-film and bulk-hardened grouts when they are in contact with water. More than one mechanism may be responsible for promoting self-sealing. These include the formation of ettringite and portlandite as well as calcite in the permeable connected porosity.

The results show that the reference grout (a modified grout) containing silica fume as pozzolanic material has the ability to self-seal. The present observation obviates the concern that modified grouts, containing silica fume or fly ash, may not seal due to a lower concentration of Ca^{2+} in the pore water. However, some aspects of mechanisms remain obscure and require further investigation. The results also indicate that the formation of calcite is not the only self-sealing process.

SUMMARY

The results from the *in situ* trials of high-performance grouts at the URL in Canada showed that no problems were encountered in mixing, handling or pumping the grouts in fractured rock. The injected grout produced only a very limited geochemical signature in the groundwater. This indicated that the grout exhibited no segregation or bleeding during or after the injection. In the *in situ* grouting trials at the URL, the apparent hydraulic conductivity of the fracture zone decreased from ~ 10^{-7} m s^{-1} to ~ 10^{-9} m s^{-1}. Microscopic examination of the grouted rock samples revealed that grout penetrated fractures with apertures as small as 10 μm.

Laboratory studies confirmed that decreasing the water content of the cement grouts by admixing superplasticizer (water reducing agent) increased strength and decreased the hydraulic conductivity of the hardened grouts. The hardened grouts have lower hydraulic conductivity (i.e., $<10^{-14}$ m s^{-1}) than intact granite (10^{-12} m s^{-1}).

Evaluation of the results from leaching studies showed that the leaching behaviour of grouts at a given temperature depends on the grout and leachant composition. The initial groundwater composition assumes a major role in grout leaching/dissolution by controlling the extent of leaching/dissolution required to produce solution saturation with leaching elements. The results indicated that leaching under both static and dynamic conditions is very complex, involving several processes such as incongruent dissolution, formation of alteration layers and precipitation. Microscopic examinations revealed that leaching/dissolution is accompanied by the formation of reaction layers, including precipitation and growth af an assemblage of secondary phases ($Mg(OH)_2$, $CaCO_3$, and $Ca(OH)_2$). The thermodynamic stability of these alteration/precipitate phases may influence the long-term performance of cement-based grouts, and in the view of very low hydraulic conuctivity, surface leaching is likely to be the major process by which the bulk high-performance cement-based grout will degrade.

It was shown that the high-performance grouts, containing silica fume as a pozzolanic material and superplasticizer as a water reducer, if mechanically disrupted have the ability to self seal.

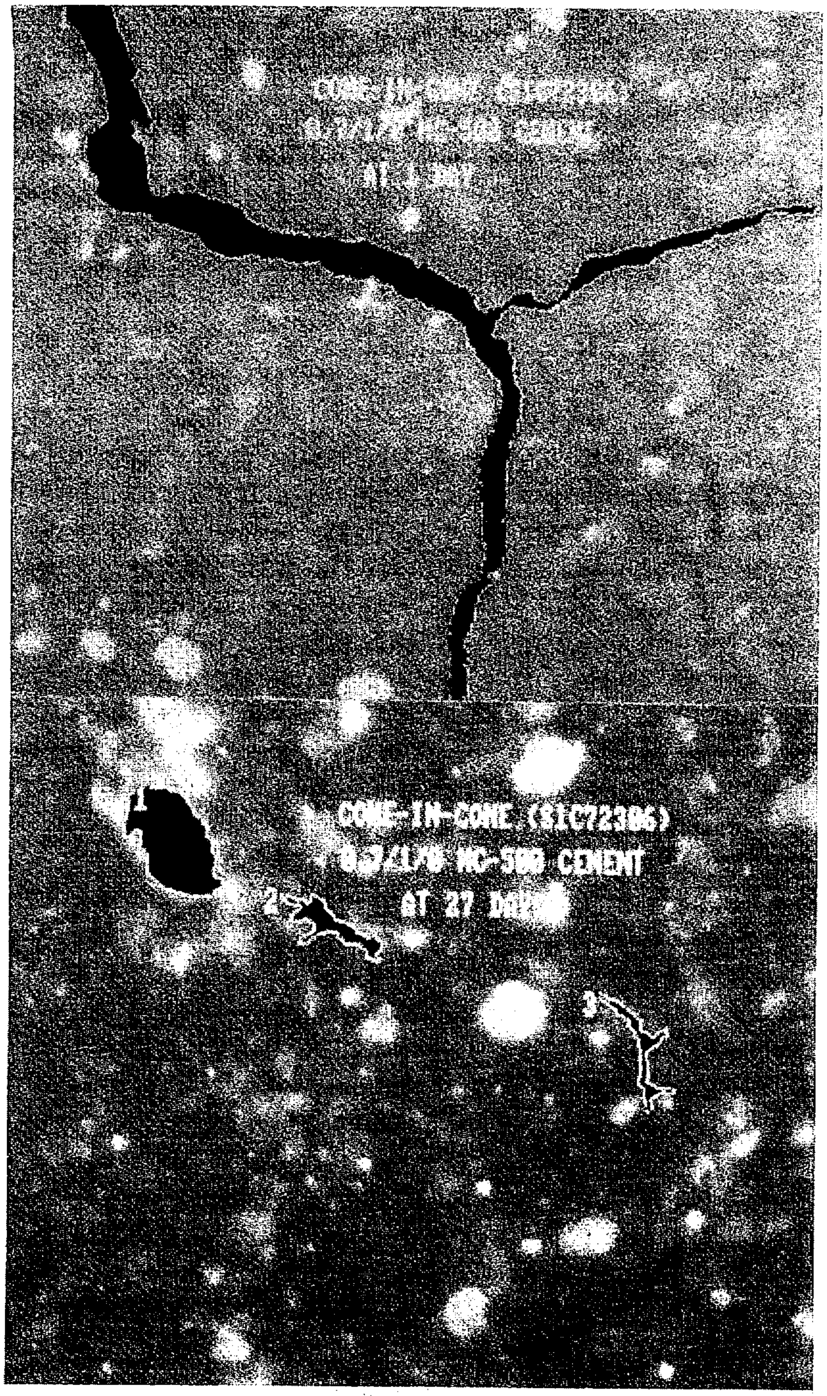

Figure 11. Autogenous healing of a crack in grout mix consisting of microfine slag cement (MC 500) with 1% superplasticizer and *W/CM = 0.7* after 27 days exposure to flowing water.

The results obviate the concern that modified grouts, containing pozzolanic materials may not heal. The ability of modified grouts to seal requires further investigation. It appears that the modified grouts can seal, certainly when they are relatively young (~2 years curing). Some uncertainty remains with much older grouts.

The results indicate that it is possible to manufacture high-performance cement-based grouts which can be injected into very fine fractures and divert water flow in granitic rock. These grouts are shown to have negligible hydraulic conductivity, associated with very low porosity, and to be highly leach resistant under repository conditions. Microcracks generated in these materials from shrinkage, overstressing or thermal loads are likely to self-seal. The results of the laboratory and in situ studies suggest that the high-performance grouts can be considered as viable materials in repository sealing applications.

ACKNOWLEDGEMENTS

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EXPERIMENTAL RESEARCH ON SEALING OF BOREHOLES, SHAFTS AND RAMPS IN WELDED TUFF

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ABSTRACT: Laboratory and in-situ experiments have been conducted to determine the mechanical and hydraulic performance of cement borehole seals in densely welded Apache Leap tuff. Test results indicate that under saturated conditions, commercial expansive cement can provide good bond strength and adequate hydraulic performance for borehole seal under changing stress conditions. The cement seal should be installed at the intact portion of the opening, and should have a length-to-diameter ratio greater than four. Drying increases borehole plug permeability and decreases mechanical and hydraulic bonds at the plug-rock interface. In-situ testing indicates that installation procedure may significantly affect the cement plug performance.

1 INTRODUCTION

Waste Isolation at the Yucca Mountain repository site may require that penetrations (e.g. boreholes, shafts and ramps) of the geological barrier be sealed. The primary function of seals or plugs is to prevent excessive flow of groundwater into the emplaced wastes and to retard the migration of the radionuclides to the accessible environment. Secondary functions include providing mechanical support for the surrounding rock, maintenance of their own physical integrity, minimization of water movement, and sorption/retardation of radionuclides. Cement is being considered as part of multi-component plugs for the repository due to its relatively low permeability, high strength, longevity and swelling capability. Cement or concrete has long been used as a hydrological barrier in underground mines (e.g. underground dams - Auld, 1983), and in the oil and gas industry (e.g. well cementing - Smith, 1976). Insufficient data exists about the mechanical and hydrological performance of cement plugs in rock under variation of in-situ stresses and borehole pressures, and in particular, about their long-term sealing effectiveness.

The primary objective of the present research is to determine experimentally the mechanical and hydraulic performance of cement borehole plugs in welded tuff. The work involves laboratory and in-situ flow testing of cement borehole plugs, and push-out testing. Densely welded Apache Leap tuff has been used as rock specimens. Although the experiments are primarily conducted on borehole scale, most of the results are applicable to sealing in larger underground excavations, e.g. shafts, ramps, tunnels and adits. Specific purposes of this paper include (i) determination of the permeabilities of cement borehole seals, tuff and their interface under various confining and borehole pressures, (ii) determination of the axial strength of cement borehole seals, and (iii) determination of the mechanical and physical properties of the tuff and cement. The present paper provides descriptions of rock specimens, experimental methods and results. Applications of the results on design of seals at the proposed repository site are discussed. Remaining knowledge gaps and future research needs are identified.

2 TEST MATERIALS

Rock specimens have been prepared from the densely welded Apache Leap tuff collected from Superior, Arizona. The location from which the tuff specimens are obtained is also the site for testing the hydraulic performance of cement seals in in-situ boreholes. Fuenkajorn and Daemen (1991) give the mechanical, petrographical and chemical properties of the Apache Leap tuff, and the location from which it was obtained. Table 1 compares the mechanical properties between the Apache Leap tuff and the Topopah Spring tuff. Table 2 compares their chemical composition. A common characteristic between the two rock types is the high intrinsic variability of the properties. Figure 1 shows variation of the uniaxial compressive strength with density of the Apache Leap tuff. The strength increases from 43 MPa at density of 2.32 g/cc to 145 MPa at density of 2.54 g/cc.

The cement tested here is commercial Type I/II Portland cement mixed with anti-foam and expansive agents. Mixing procedure follows the American Petroleum Institute (API, 1986). Prior to testing, the cement has been cured under distilled water for 7 - 10 days. The uniaxial compressive strength is averaged as 18.8 MPa. Young's modulus and Poisson ratio are 5.25 GPa and 0.22, respectively. Figure 2 shows the increase of the swelling pressure of the cement cured in steel pipes with different diameters. Within the first year of curing, the swelling pressures rapidly increase and tend to remain constant with time. Results from related experiments conducted by Fuenkajorn and Daemen (1986, 1987) suggest that swelling pressures of cement plug are also governed by the restraint of the boundary (Figure 3). For example, cement plug cured in a stiff rock (high elastic modulus) tends to have a greater swelling pressure than that cured in a softer rock.

3 LABORATORY PERFORMANCE OF CEMENT BOREHOLE PLUGS

3.1 Radial Permeameter Tests

Radial permeameter testing (South and Daemen, 1986) has been conducted to investigate the sealing performance of cement borehole plugs under a variety of stress conditions. Figure 4 gives the configuration of the radial permeameter cell. Changing the stress conditions sequentially makes it possible to impose severe conditions on the plug-rock interface. Application of external and internal pressures to the specimens allows simulating in-situ stress conditions. The applied stressfield makes it feasible to operate at high differential pressures across the borehole plug.

To evaluate borehole plug performance, water flow through a plugged borehole in tuff is compared with flow through rock itself. Rock cores 15 cm in diameter and 30 cm long have

Table 1. Comparison of Mechanical Properties (dry and intact) between Apache Leap Tuff and Topopah Spring Tuff.

	Apache Leap Tuff		Topopah Spring Tuff	
Uniaxial Compressive Strength (MPa)	97.5	(25.1)	134.4	(29.3)
Elastic Modulus (GPa)	22.6	(5.7)	26.7	(7.7)
Poisson's Ratio	0.20	(0.03)	0.14	(0.05)
Brazilian Tensile Strength (MPa)	5.72	(1.2)	12.8	(3.5)
P-Wave Velocity (km/s)	6.4	(1.5)	4.1	
Internal Friction Angle (degree)	43		47	
Cohesion (MPa)	15.9		27.7	
Density (g/cc)	2.37	(0.42)	2.22 1.32	(welded) (non-welded)
Porosity (%)	5 - 10		6 - 20	

Note: Value in parenthesis indicates standard variation.

Table 2. Chemical Composition of Apache Leap Tuff and Topopah Spring Tuff.

	Apache Leap Tuff (%)	Topopah Spring Tuff (%)
SiO_2	71.7	76.8
Na_2O	3.8	4.1
K_2O	2.5	3.8
CaO	2.5	0.3
MgO	0.85	0.11
Al_2O_3	14.8	12.4
Fe_2O_3	3.3	1.0
FeO	0.1	0.05

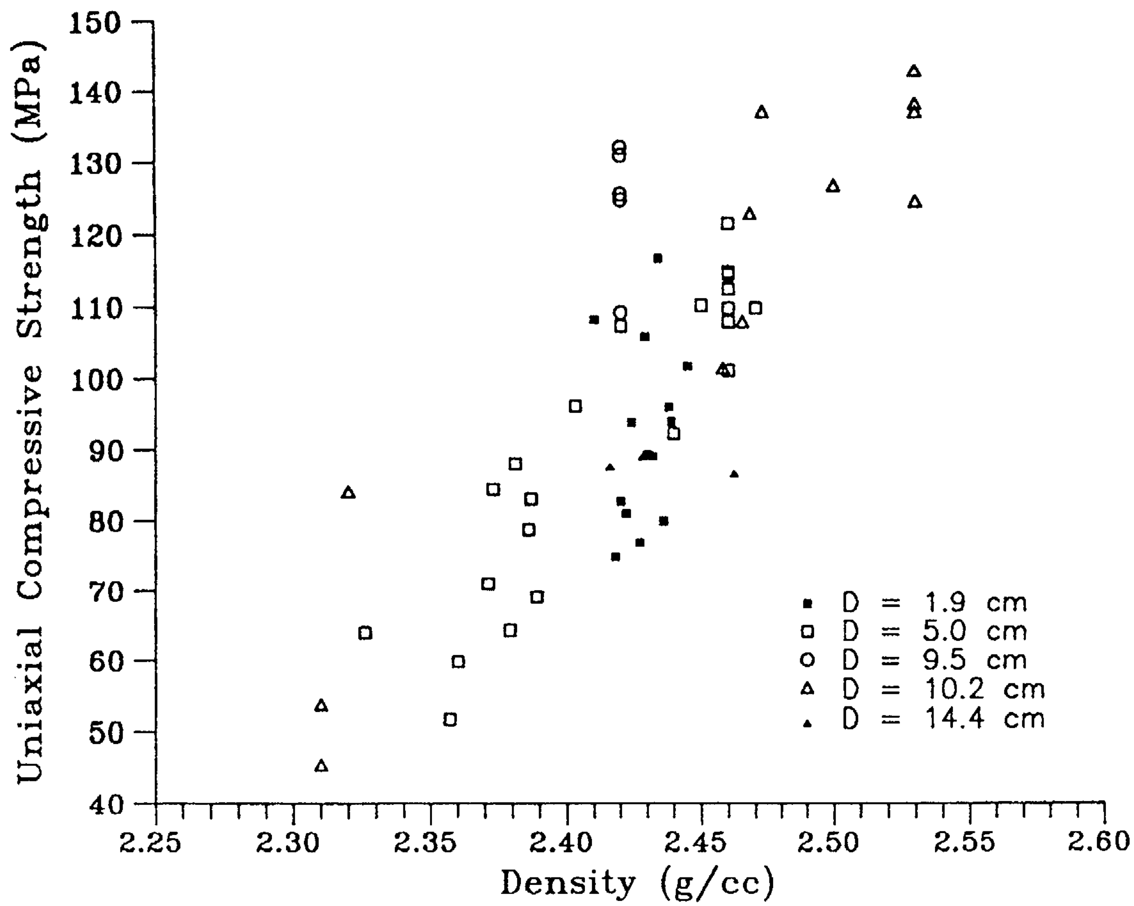


Figure 1. Uniaxial compressive strength of densely welded Apache Leap tuff.

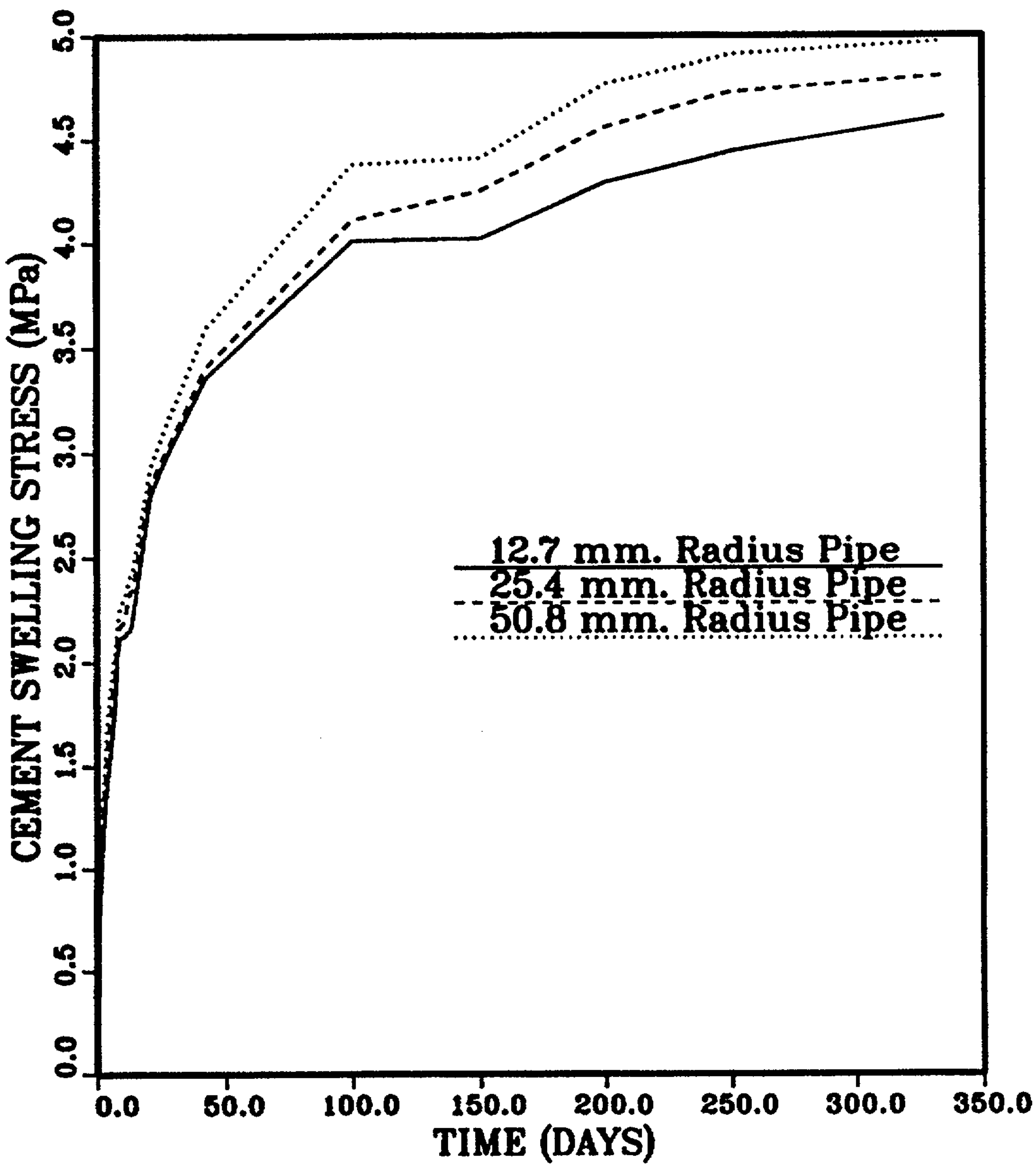


Figure 2. Expansive stress of cement as a function of time cured under distilled water.

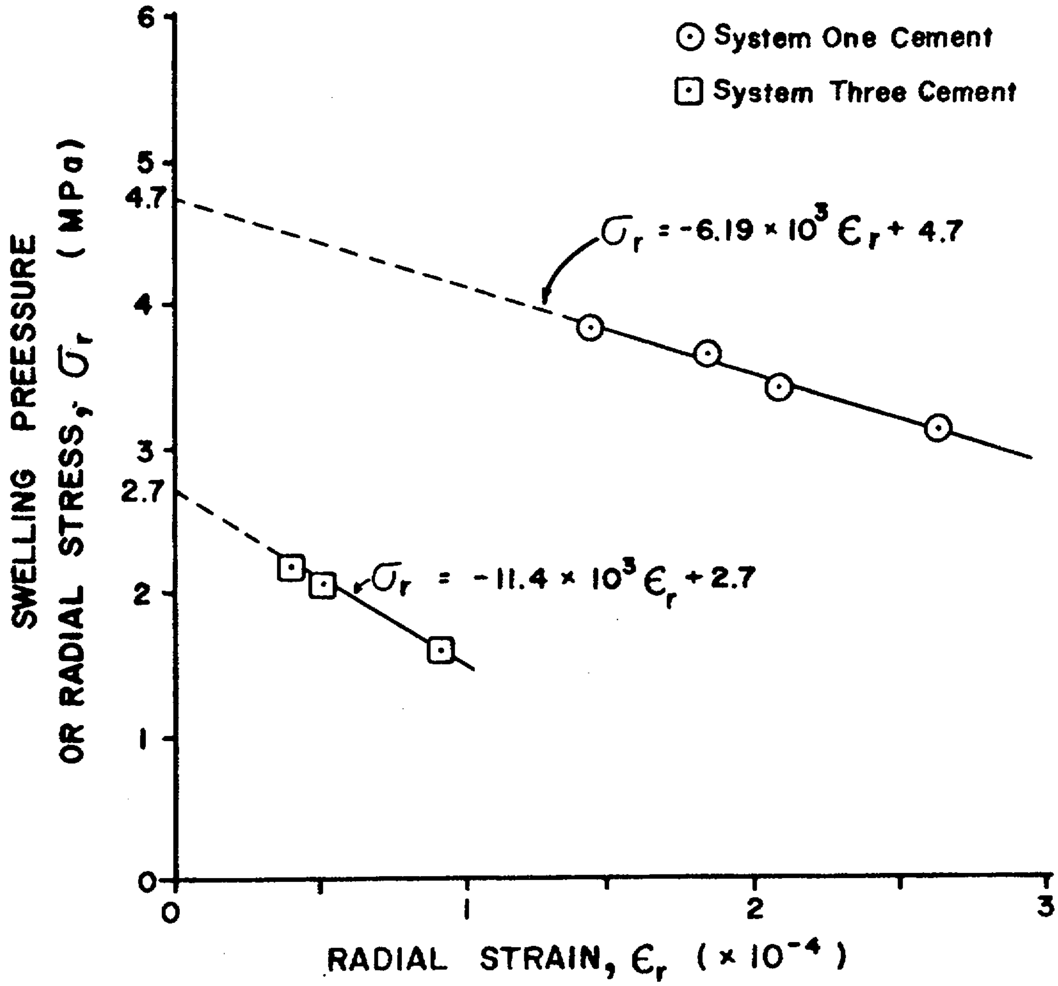


Figure 3. Radial expansive stress of cement as a function of radial strain at 7 day curing period.

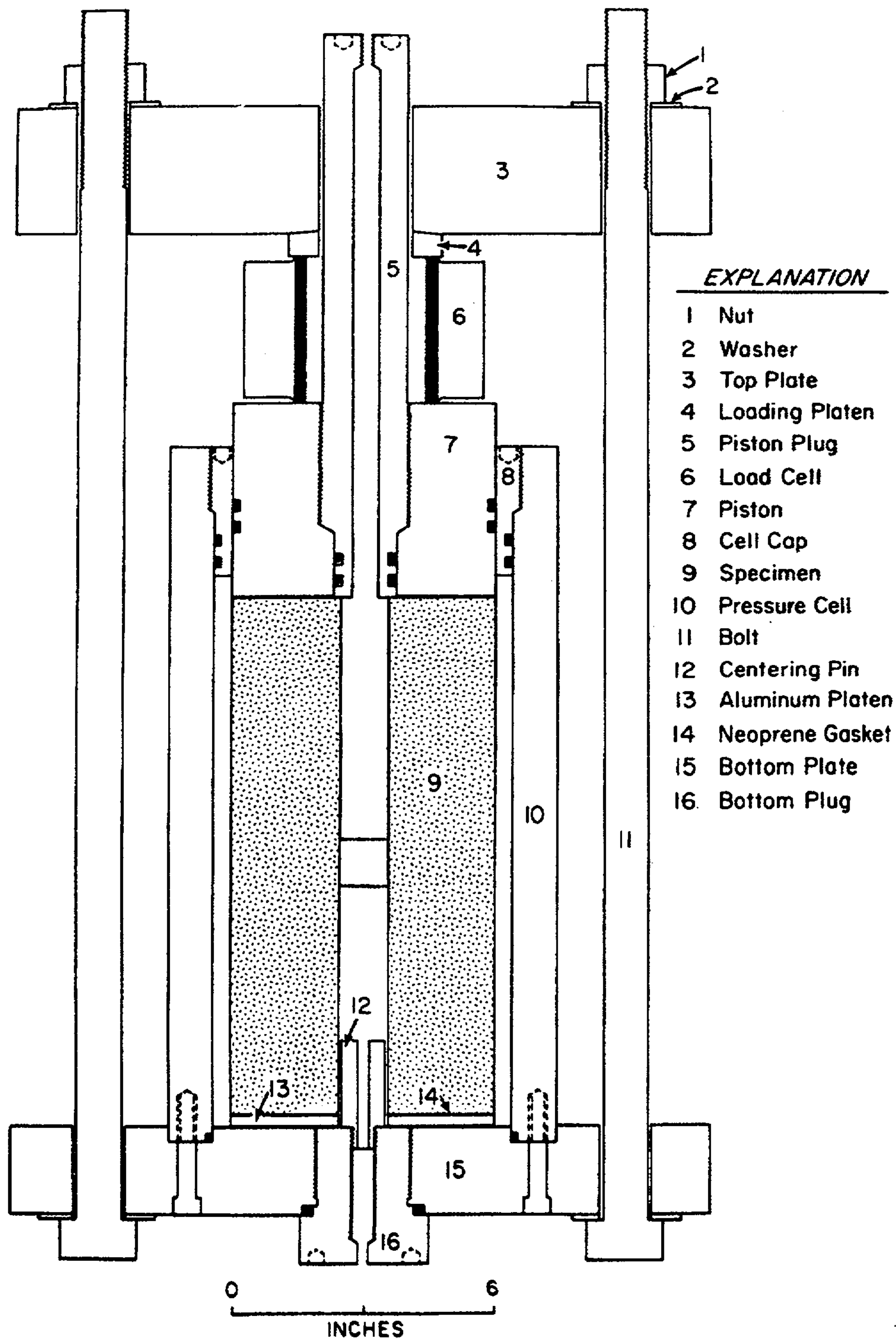


Figure 4. Radial permeameter cell.

2.54 cm diameter coaxial hole drilled from each end, leaving a 2.54 cm long rock bridge in the center of the specimen (Figure 5). Laboratory arrangement is shown in Figure 6. Prior to flow testing, the rock bridge specimens have been saturated by injecting a water pressure of 1 MPa into the bottom hole and applying a vacuum pressure of -200 kPa to the top hole. The saturation process is terminated after water flow into the top hole. For flow testing, the constant confining pressures vary from 0 to 12 MPa and the injection (top hole) pressures from 0.5 to 8 MPa. For each set of pressures, the test is run until a constant outflow rate has been observed. The constant confining pressure is increased progressively while the injection pressure is usually kept to less than 75% of the confining pressure. Identical sequences of applied pressures are used for both rock bridge testing and plug testing.

3.2 Flow Calculation - Finite Element Analysis

Finite element analyses have been performed using the program FREESURF (Neuman and Witherspoon, 1970) to determine the hydraulic conductivities of the tuff cylinder, rock bridge and plug from the measured flow rates, to estimate the amount and path of water flow through the specimen, and to determine the effect of anisotropic rock permeability on the flow. The analysis is made in axisymmetry, assuming that the specimen is homogeneous and fully saturated, and that Darcy's law is valid. By performing a parametric analysis the flow rates measured from the radial permeameter testing can be used to determine the hydraulic conductivities of the tuff cylinder and the cement plug. Figure 7 shows some of the results of the finite element simulation, in term of flow path (flow net) through the specimens with different plug-to-rock permeability ratios. Fuenkajorn and Daemen (1991) give details of the analysis.

Three radial permeameter specimens had been subjected to constant injection pressures for over two years. Rock bridge test results show the hydraulic conductivities of Apache Leap tuff ranging from 10^{-11} cm/s under unconfined condition to 10^{-12} cm/s under a confining pressure of 8 MPa. The conductivities remain at 10^{-12} cm/s when the confining pressure is increased to 12 MPa. Comparison of these results with those obtained from related experiment by Sharpe and Daemen (1991) suggests that Apache Leap tuff may be hydrologically anisotropic. The vertical permeability (normal to flow layer) could be up to an order of magnitude lower than the horizontal one. This is also supported by the results from petrographic analysis (Fuenkajorn and Daemen, 1991).

The results from cement plug flow testing indicate that the hydraulic conductivity of the plug ranges from 10^{-11} to 10^{-10} cm/s (Figure 8). The plug permeability tends to decrease with increasing confining pressure. Under low confining pressures (less than 2 MPa), the plug permeability decreases with increasing injection pressure. The effect of injection pressure becomes insignificant under confining pressures larger than 8 MPa. The variation of plug permeability with changes of injection pressures is less than one order of magnitude. The effect of test period on the permeability has not been observed. This implies that the cement plugs can maintain their sealing performance adequately for over a year.

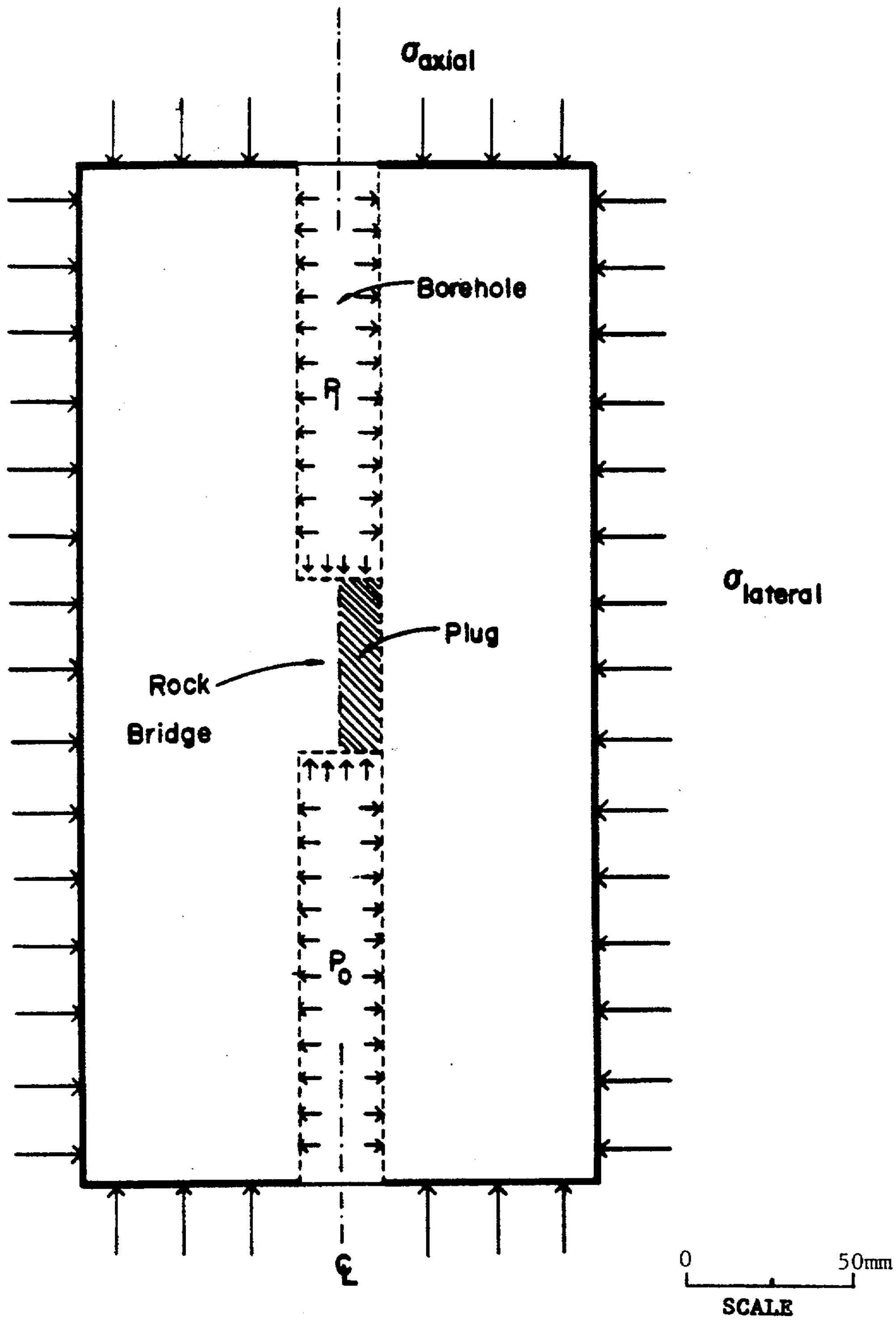


Figure 5. Loading configuration on radial permeameter specimen.

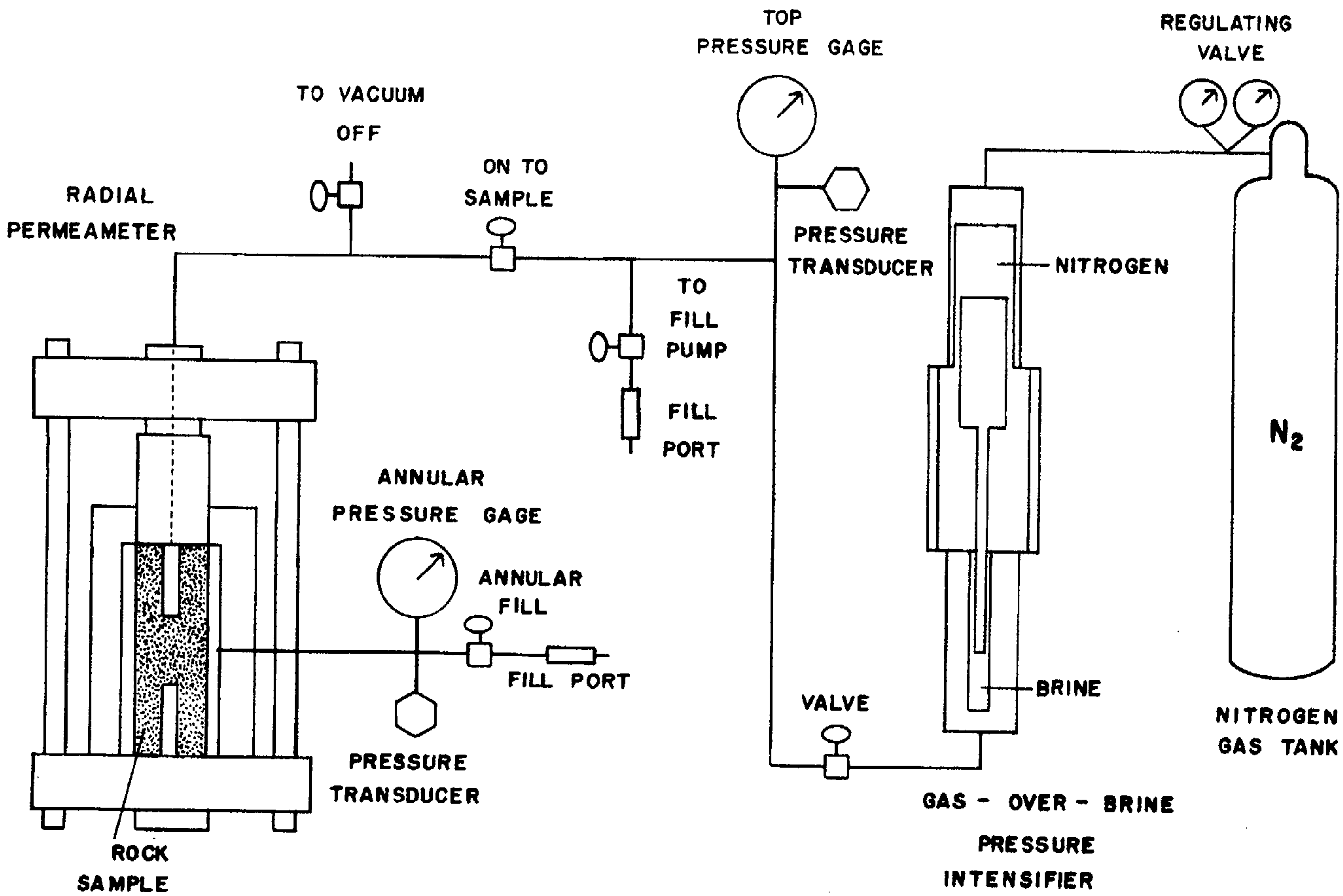


Figure 6. Laboratory arrangement for radial permeameter testing.

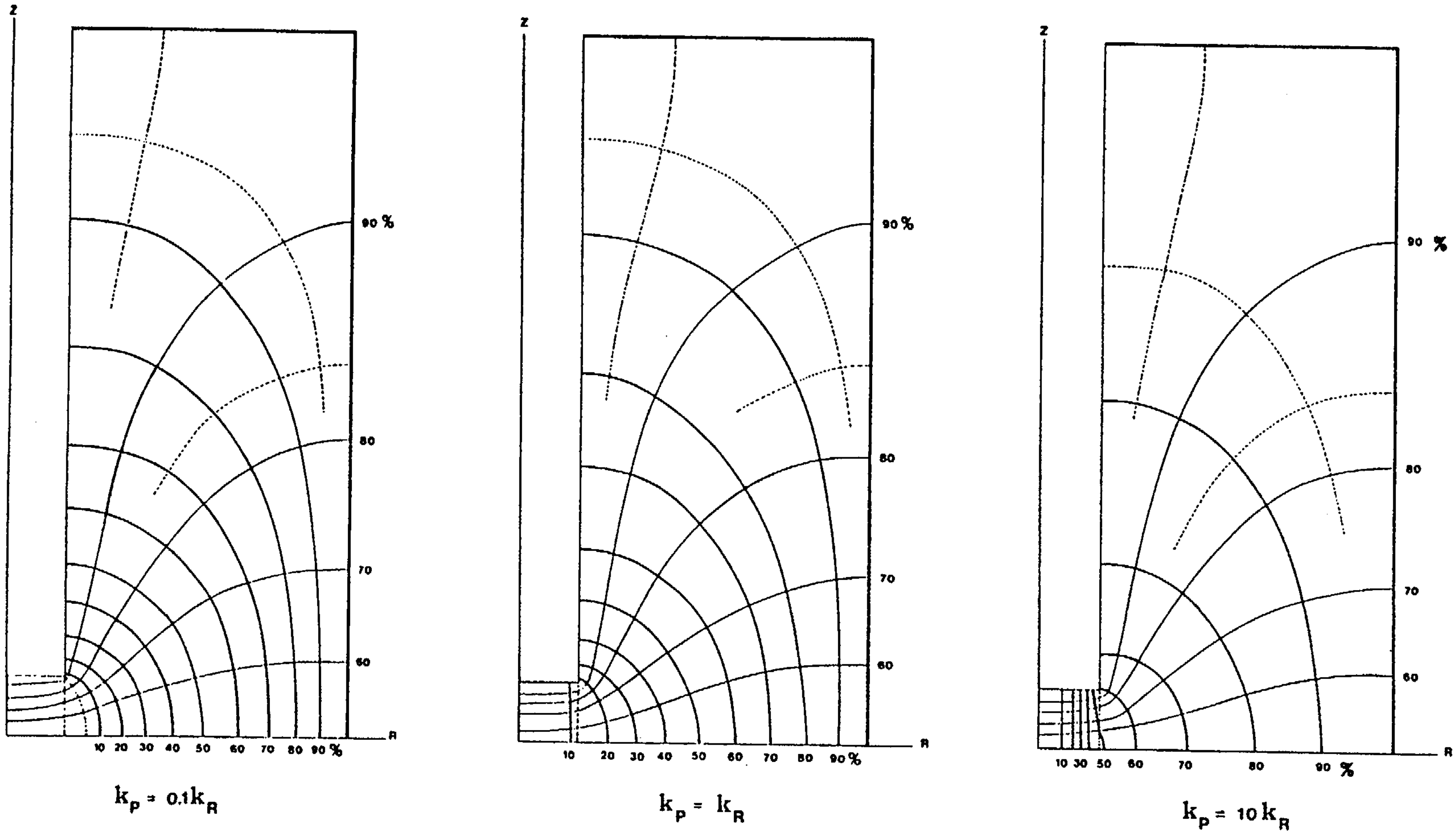


Figure 7. Flow nets in radial permeameter specimens calculated by program FREESURF.
k_P: plug permeability, k_R: rock permeability

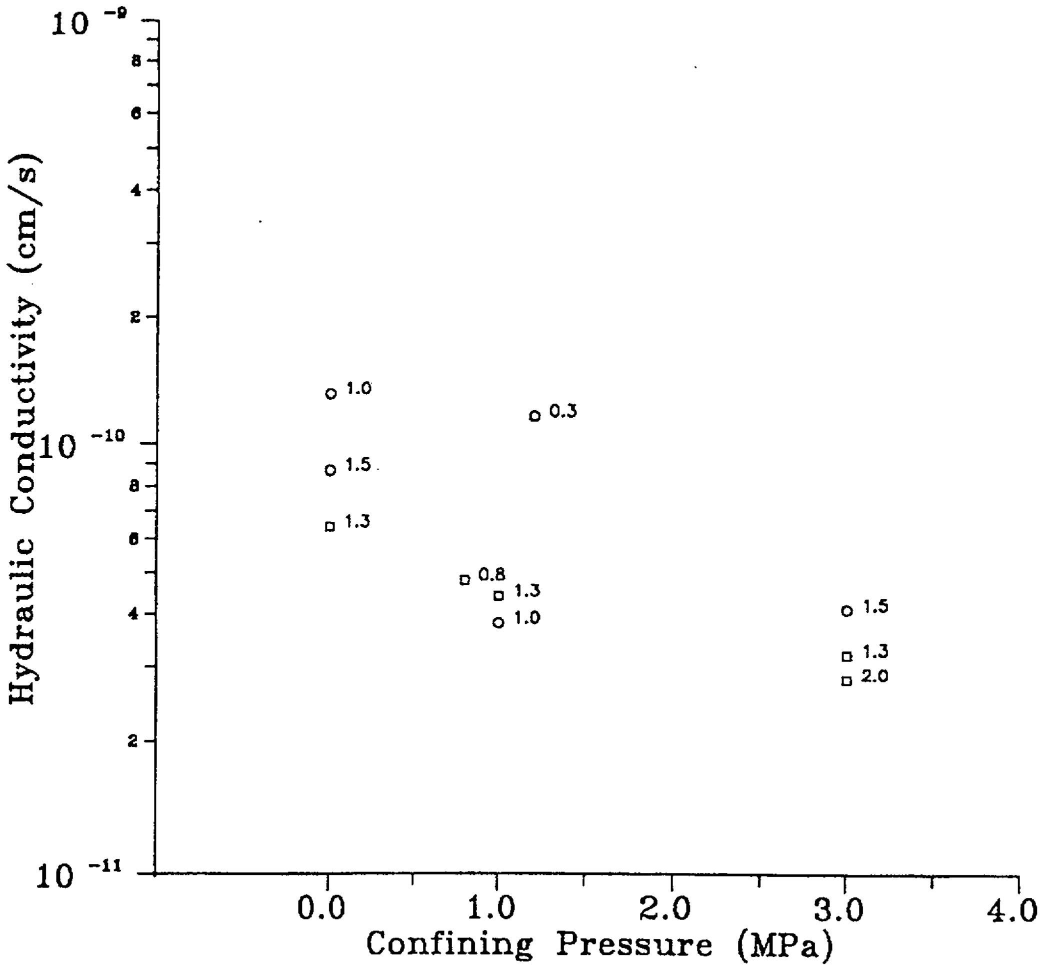


Figure 8. Hydraulic conductivities of cement borehole plugs. Numerical value at each data point indicates injection pressure in MPa.

3.3 Stress Analysis - Finite Element Method

The program PLANE2D-FE (Desai and Abel, 1979) is used to determine tangential and radial stresses in the permeameter specimen under a variety of confining and borehole pressures. This is part of a preliminary investigation on the effect of the applied pressures on the flow path and permeability of the tuff and cement borehole plug. The analysis is made in axisymmetry and assumes that the materials are linearly elastic, homogeneous and isotropic, and that the rock-cement interface is a welded surface.

One of the objective is to identify tension zones at the plug-rock interface. The tension zone is of concern because it may induce a separation at the interface, particularly near the upstream end of the plug. Figures 9 and 10 give some of the finite element results. The analysis indicated that the tension zone will not be induced at the interface if the injection pressure is less than 75% of the confining pressure. Akgun and Daemen (1991) give a detailed stress analysis for cement borehole plugs in welded tuff subjected to a variety of applied pressures.

4 IN-SITU PERFORMANCE OF CEMENT BOREHOLE PLUGS

The primary objective of the in-situ experiment is to determine the hydraulic conductivities of cement borehole plugs in welded tuff as affected by field installation and in-situ environment. Three 15 cm diameter vertical test holes were drilled in the densely welded Apache Leap tuff, near Superior, Arizona. Inclined holes (5 cm diameter) were drilled at 55° from the surface to intersect near the bottom of the vertical holes. The objective of the inclined hole is to provide an access to beneath the plug (Figure 11). The vertical boreholes were hydraulically tested using a straddle packer arrangement to locate and determine the hydraulic conductivity of an intact rock zone for emplacing of the seal in the vertical borehole. The hydraulic conductivity of the intact tuff was about 10^{-11} cm/s, and 10^{-4} - 10^{-6} cm/s for the fractured tuff.

Prior to installing a plug in the field, a full-scale laboratory model was constructed to aid in determining the optimum cement grout plug installation techniques. Plugs were placed with a custom built dumping bailer both underwater and without water present. These trials indicated that the installation of cement seals with standing water in the borehole can cause significant piping channels along the plug and borehole wall interface, creating pathway for preferential flow. The piping effect was recognized for seals installed with any level of standing water present. In addition to piping, significant mixing of the cement with the water was evident. This mixing can noticeably increase the porosity and permeability of the plug and can weaken the plug. Based on the laboratory simulation, the plug in the field was installed in a dry hole. The cement was lowered into the hole with a bailer to minimize disturbance of the mixture and released above a previously installed injection stand cemented into the bottom of a test hole (Figure 12). The inclined hole allows for access to both ends of the plug (top and bottom). This allows injection and collection of fluid on either side of the plug. The two-sided

Tangential and Radial Stresses in Permeameter Test Specimen
Confined Conditions: $P_i = 0.5\ \sigma_3$

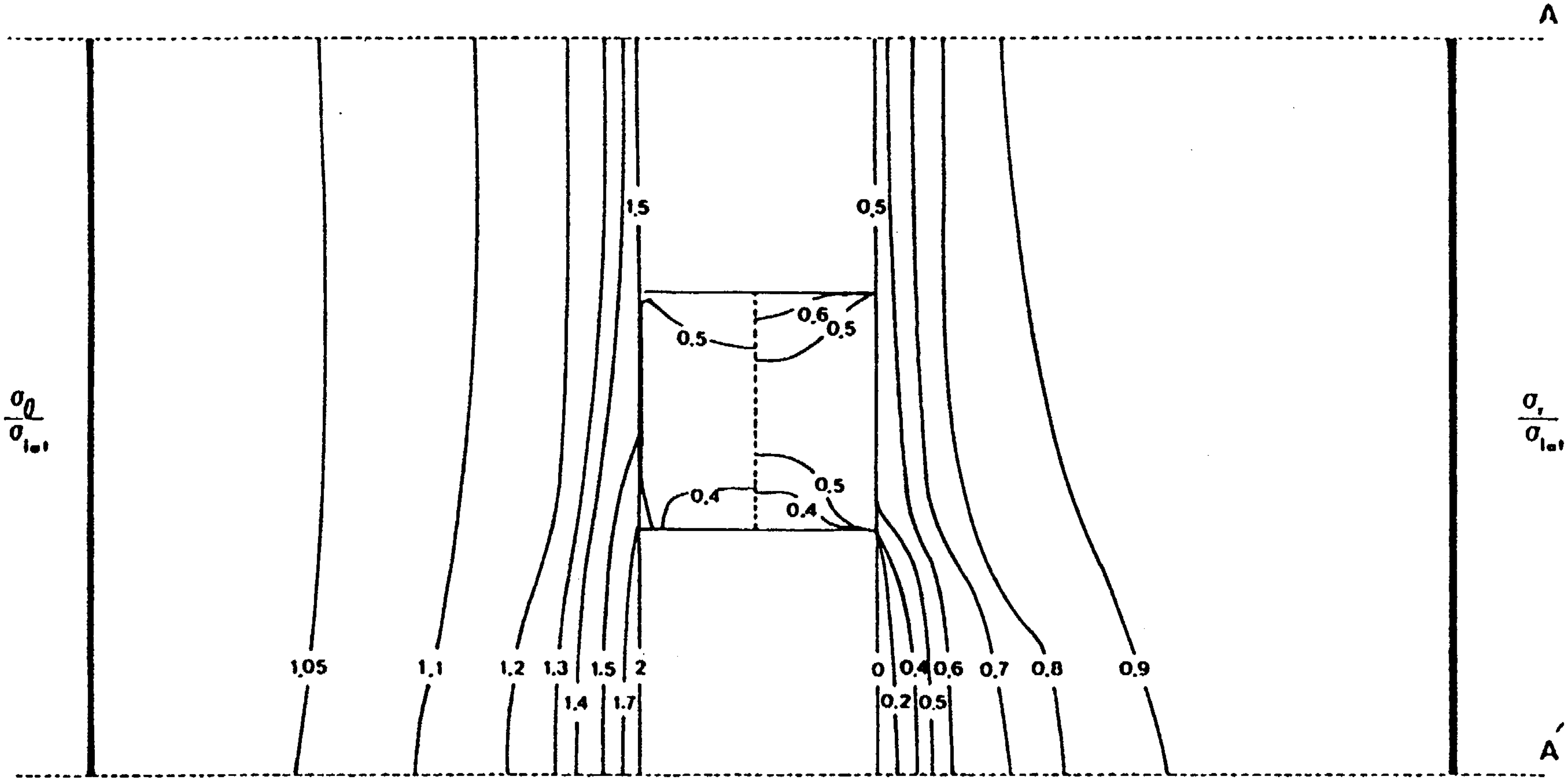


Figure 9. Tangential and radial stress distribution at the mid-section of radial permeameter specimen. injection pressure = 0.5 confining pressure.

Tangential and Radial Stresses in Permeameter Test Specimen
Unconfined Conditions: $P_i = 1, \ \sigma_3 = 0$

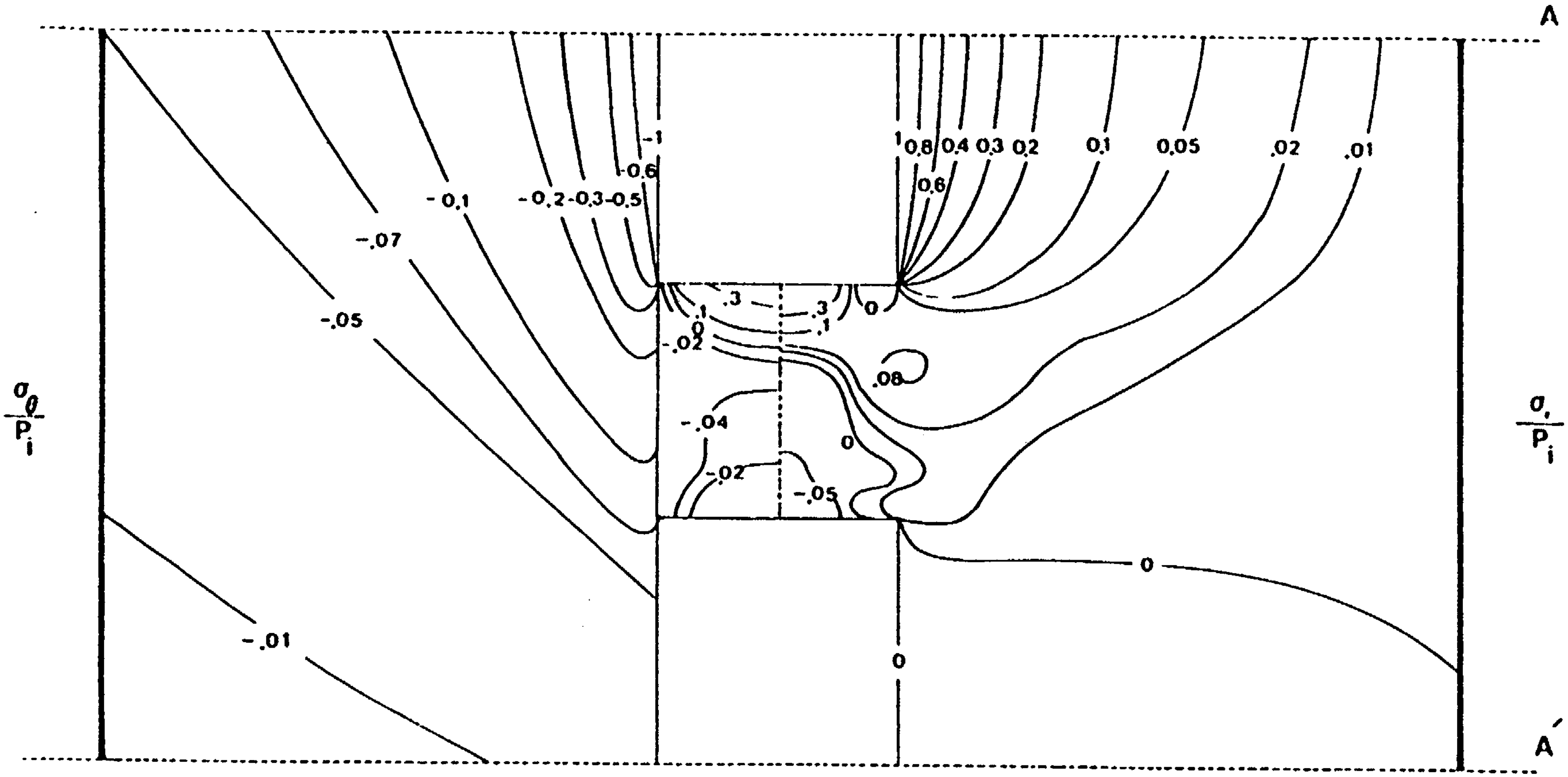


Figure 10. Tangential and radial stress distribution at the mid-section of radial permeameter specimen. Unconfined condition: injection pressure = 1, confining pressure = 0

In-Situ Flow Test:
Borehole Preparation

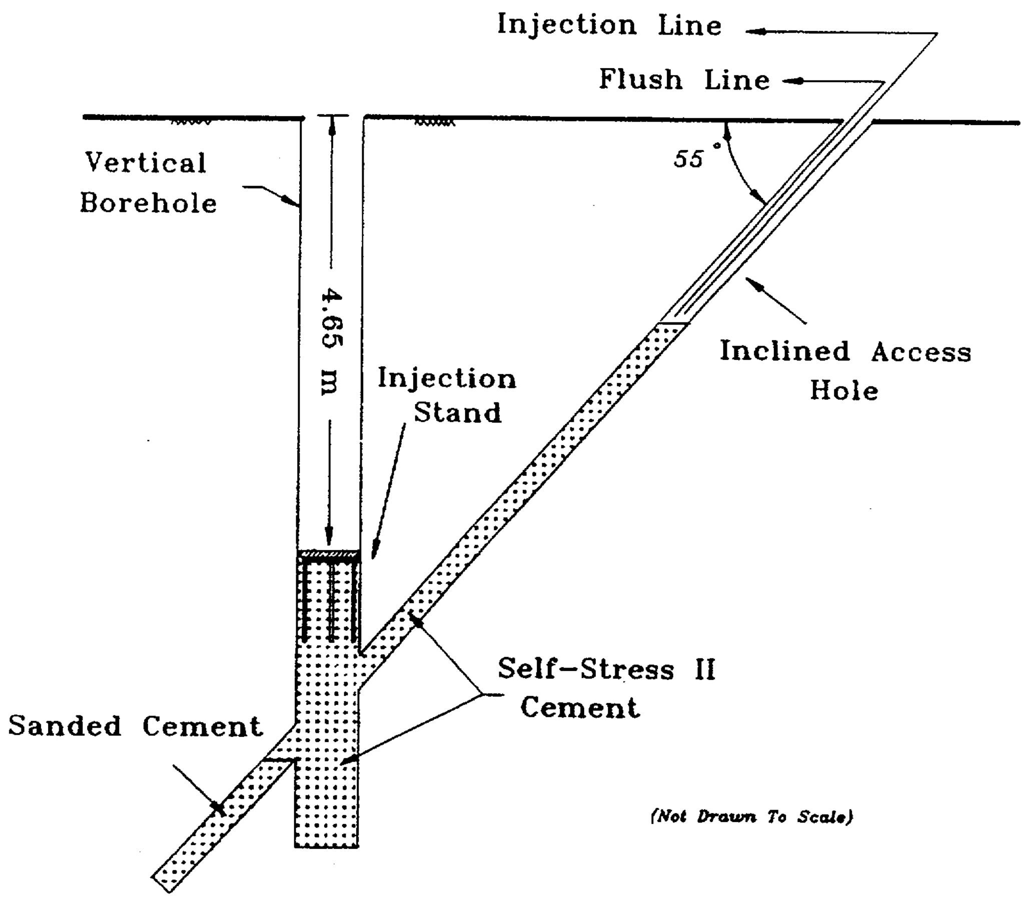


Figure 11. Cross-sectional test holes prepared for in-situ flow testing on cement borehole seals

In-Situ Flow Test: Cement Borehole Plug

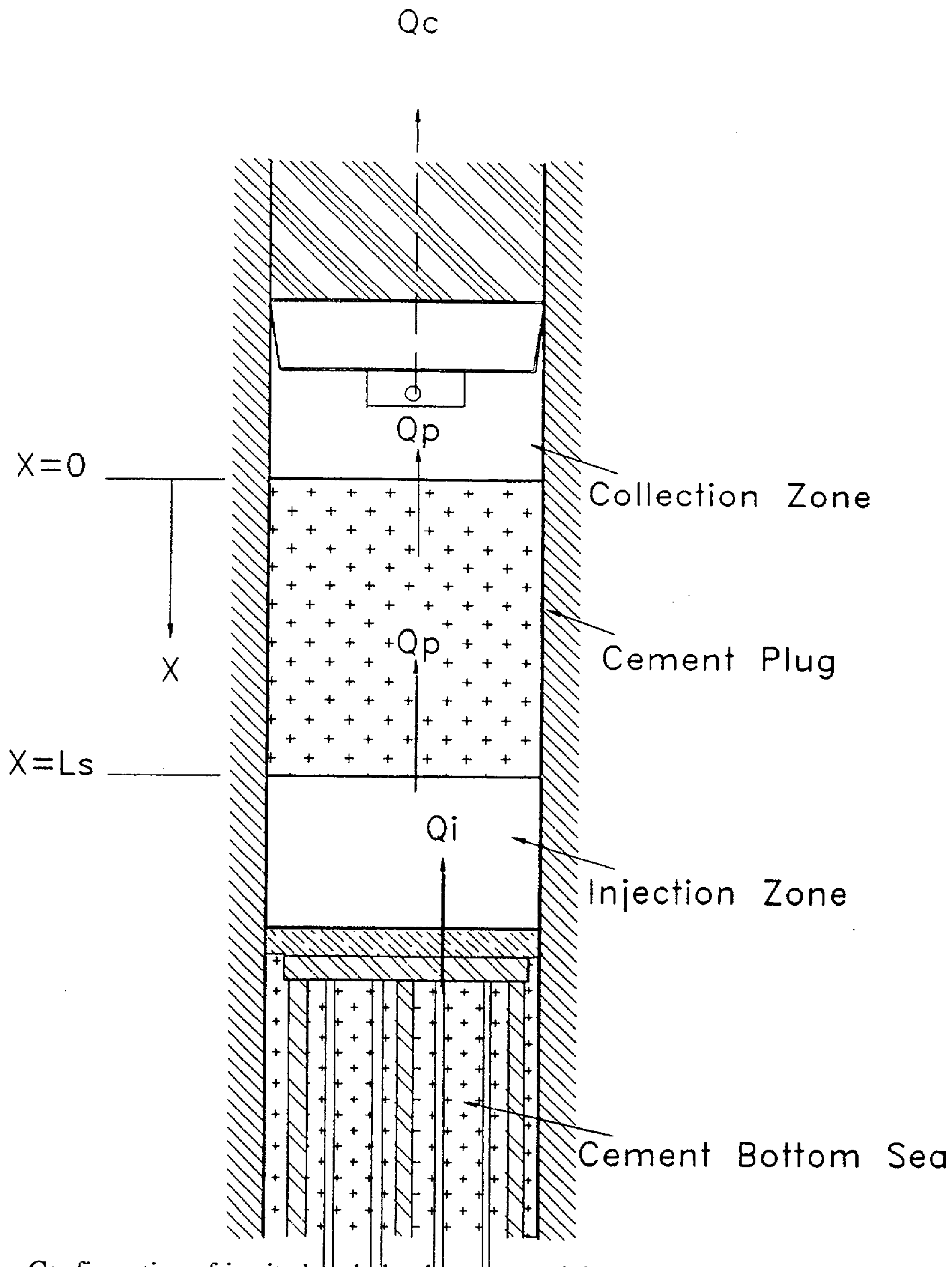


Figure 12. Configuration of in-situ borehole plug prepared for steady state and transient flow testing.

configuration allows verification that measured flow rates, through a balance check of the injection and collection of flow rates, are not erroneous.

A series of one dimensional constant head steady-state and transient injection tests were performed over a two years period. The transient tests include a head buildup test and a transient constant head test. Crouthamel (1991) describes detailed test method and analysis. The hydraulic conductivity of the cement seal was confirmed through four steady-state and three transient tests to be in the order of 10^{-10} cm/s.

5 AXIAL STRENGTH OF CEMENT BOREHOLE PLUGS

Push-out testing is designed to determine the shear strength at he cement plug-rock interface. Figure 13 shows the test arrangement. A cylinder steel rod applies an axial load to a neat cement grout plug installed in a rock cylinder. The LVDT and dial gage that measure the vertical displacement of the top of the plug are mounted on horizontal arms connected to the loading rod. The top LVDT and dial gage displacement monitoring points rest on horizontal brackets clamped to fixed vertical reference bars. The steel platen underneath the specimen has a slid on one side to allow the downward movement of the horizontal arm of the bottom vertical displacement monitoring assembly. A vertical rod, screwed into the bottom of the cement plug is connected to the horizontal arm which supports the bottom LVDT and dial gage monitoring points for bottom plug displacement measurements. The bottom LVDT and dial gage are clamped to fixed vertical reference bars.

The tuff cylinders tested had inside radii of 6.4, 13, 25 and 51 mm, outside radii ranging from 38 to 94 mm, and lengths ranging from 102 to 178 mm. The tuff cores were plugged with nearly centered Self-Stress II cement grout plugs having length-to-radius ranging from 2 to 8. The cement grouts of the push-out specimens were initially loaded to 4450 N. The load was kept approximately constant and incremented 4450 N every 5 minutes until the plug failed. The load and displacements were recorded every 30 seconds upon failure. Akgun and Daemen (1991) give detailed experiment method, analysis and results. Figure 14 shows a typical axial stress-displacement curve of the push-out test on the cement grout plugged in tuff specimen. The results are analyzed by assuming that the shear stress was uniformly distributed along the plug length and by using finite element method. Figure 15 shows the axial and shear stress distribution along the plug length at the interface resulting from the finite element method. The average shear strength at the plug-rock interface varies from 3.9 to 11.0 MPa.

The peak shear stress along plug-rock interface increases with decreasing Young's modulus ratio of plug and rock, and with decreasing plug length-to-diameter ratio. The axial stresses and interface shear stresses do not affect the entire lengths of plugs if the length-to-diameter ratio is greater than two.

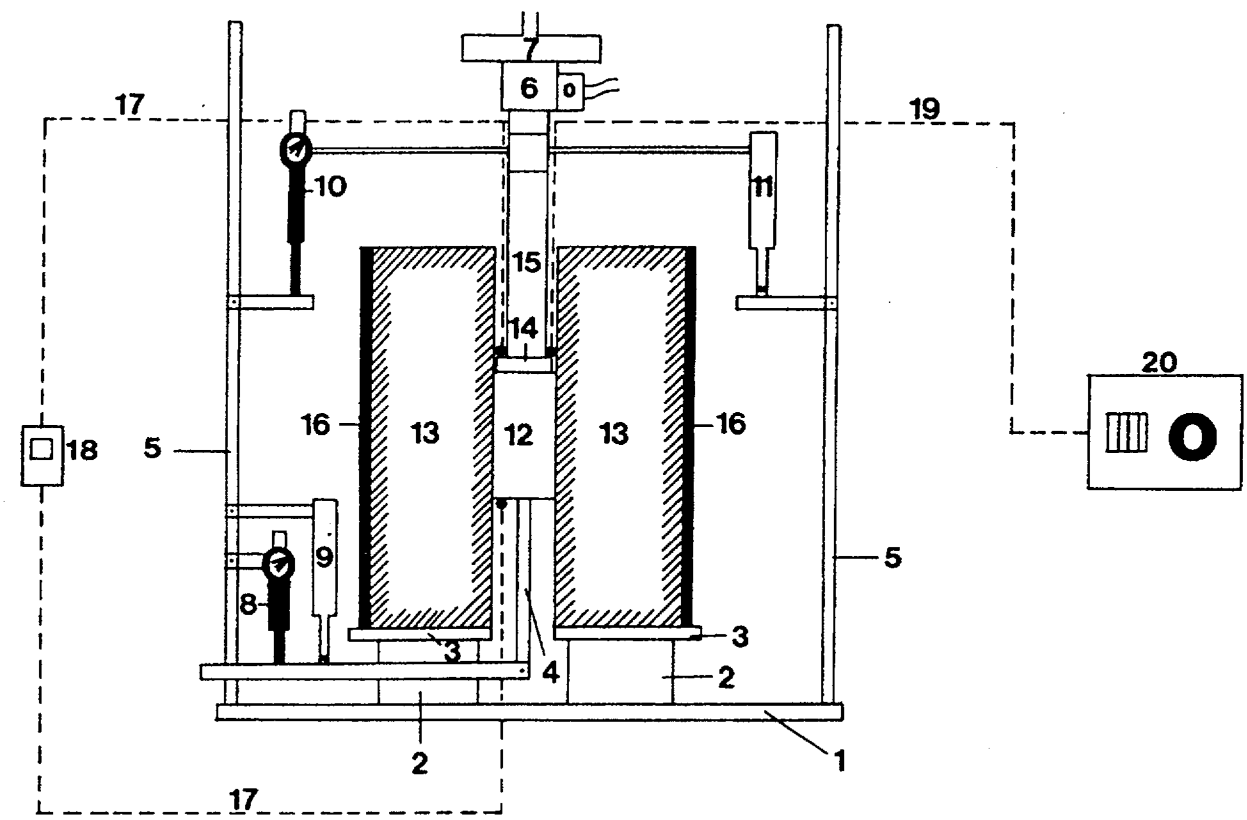


Figure 13. Laboratory arrangement for push-out testing.

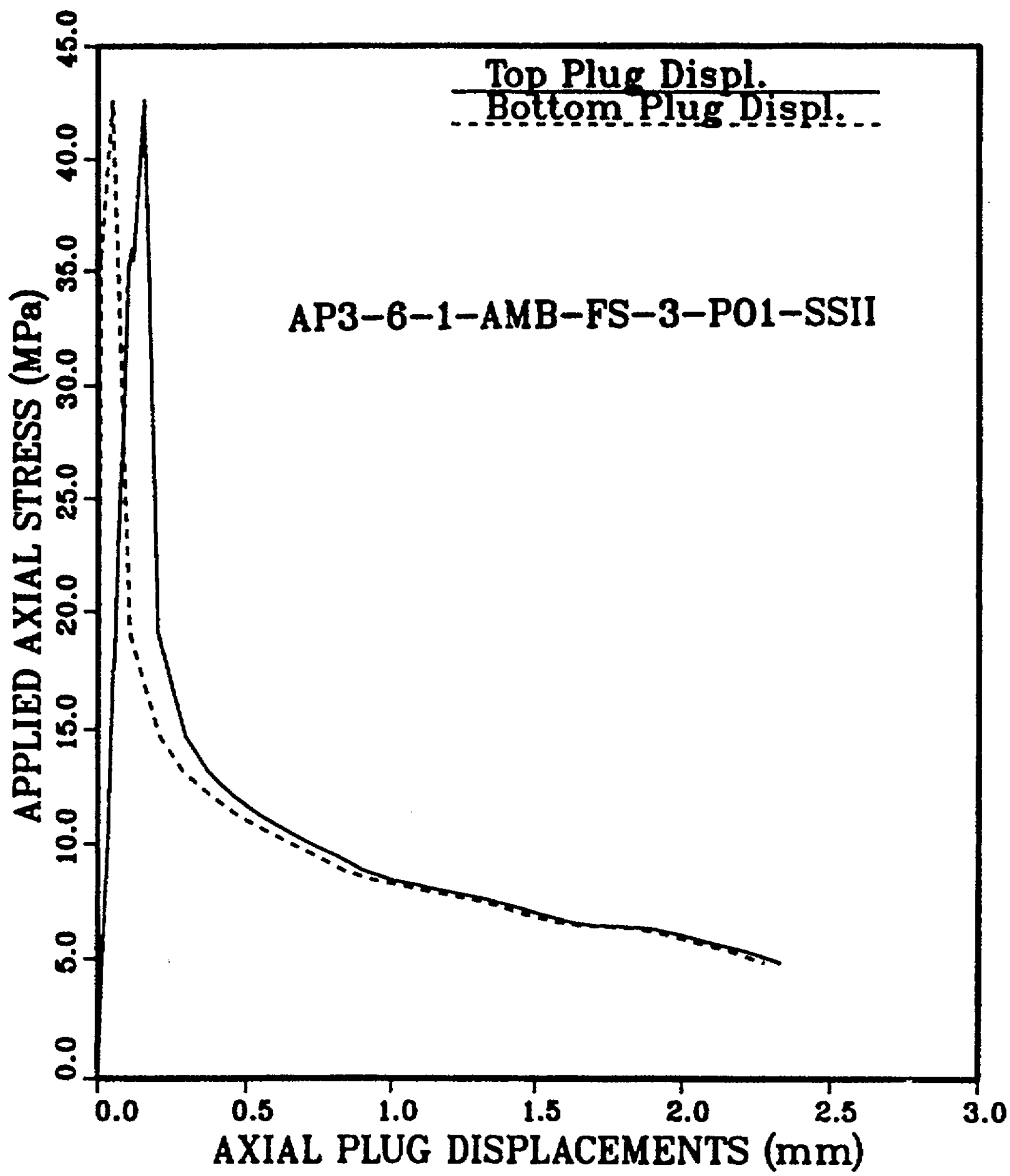
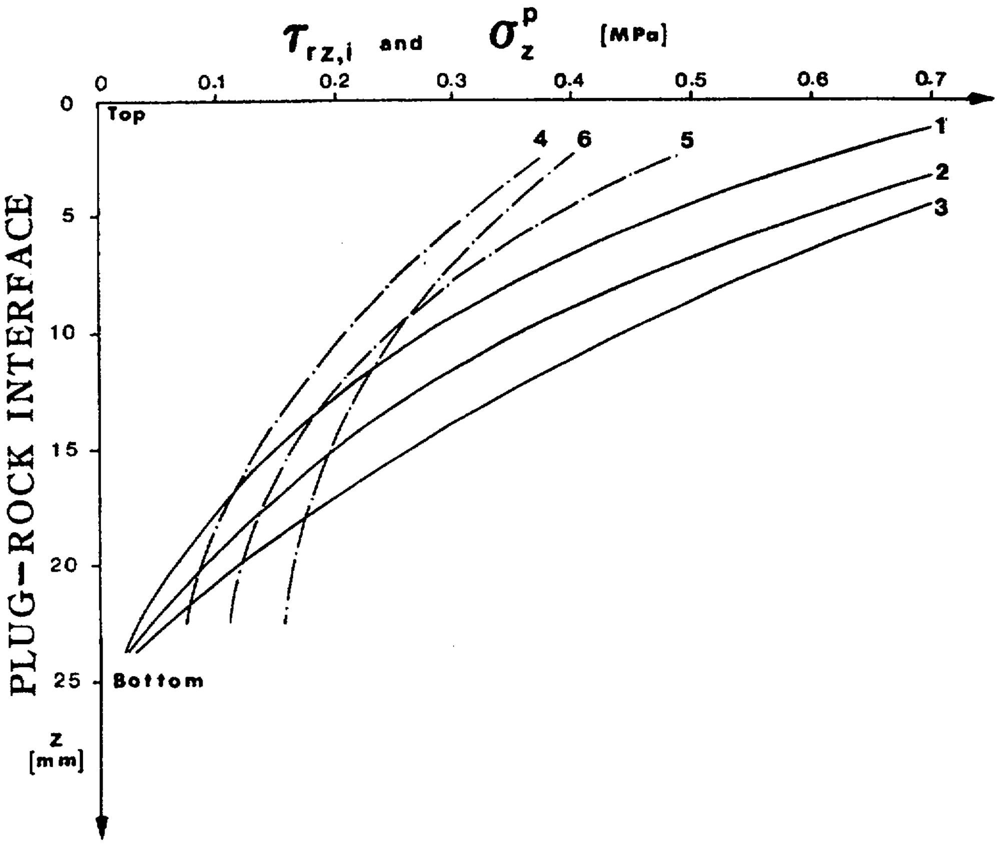


Figure 14. Typical results of push-out testing of cement plug in Apache Leap tuff.



Solid lines (1,2,3): normal stress (σ_z^p)
Broken lines (4,5,6): interface shear stress ($\tau_{rz,i}$)

Figure 15. Distribution of normal and shear stresses along the interface between cement borehole plug and welded tuff, calculated by finite element method.

6 DISCUSSIONS

The cement plug permeability obtained from radial permeameter testing is about an order of magnitude lower than those from in-situ flow testing. The discrepancy is probably due to a combination of the following factors.

(i) Installation procedure. This is primarily related to cement mixing duration, temperature and humidity during plug installation, which could affect the swelling capability and porosity of the plug.

(ii) Conditions of the borehole wall. The borehole wall of the radial permeameter sample is clean, smooth and free of fractures. This probably results in a high interaction pressure at the plug-rock interface. This pressure, induced by cement expansion, lead to a good hydraulic bond at the interface. Such conditions are unlikely for an in-situ borehole. Installation of cement plug in a rough borehole wall and near fractures may result in a lower interaction pressure and a poorer hydraulic bond at the interface.

(iii) Injection pressure (hydraulic gradient). Evidence from radial permeameter testing indicated that increasing the injection pressure results in a decrease in hydraulic conductivity for the cement plug. The injection pressure used in the in-situ flow test (less than 0.3 MPa) is significantly lower than those used in the laboratory.

The push-out tests performed in this study had a narrow range of test geometries and specimen stiffness. The strength results should be extrapolated only with extreme caution. In this study the tuff cylinders were not confined. It can be assumed that the strengths measures are lower bounds. Confined push-out testing should also provide better insight into the relative contribution of the adhesive and frictional interface strengths. The tests reported here are short-term tests (minutes to hours) and are performed on neat cement grouts cured for a short time, typically eight days. Performing long-term quasi-static loading would aid in understanding any stress corrosion effects, creep, and true long-term strength. Push-out tests on cement grout plugs mixed with crushed tuff or on concrete should be performed to simulate plugs in larger excavations.

7 SUMMARY AND RECOMMENDATIONS

Tables 3 and 4 summarize the test results obtained from the experimental work presented here and from other related experiments conducted at the University of Arizona for the U.S. Nuclear Regulatory Commission. The implications of the research results on the design considerations for sealing of the boreholes at the Yucca Mountain repository site can be summarized as follows.

(i) Available expansive cements seem adequate to provide good performance for borehole seals under saturated condition.

Table 3. Summary of Test Results on Apache Leap Tuff Permeabilities.

	Hydraulic Conductivity (cm/s)	Test Conditions
Intact Rock:	10^{-10} - 10^{-9}	confining pressure = 0 MPa
	10^{-12} - 10^{-10}	confining pressures = 8 - 12 MPa
	10^{-11}	4.3 m deep, in-situ boreholes
Natural Fractures:	10^{-2} - 10^{-4}	normal stress = 0 MPa
	10^{-3} - 10^{-6}	normal stresses = 4 - 10 MPa
Artificial Fractures (tension induced):		
	10^{-1} - 10^{-3}	normal stress = 0 MPa
	10^{-4} - 10^{-5}	normal stresses = 4 - 10 MPa

Table 4. Summary of Test Results for Cement Borehole Seal Performance.

Saturated Condition:			
Hydraulic conductivities	=	10^{-10} cm/s	(unconfined condition)
	=	10^{-11} cm/s	(confining pressures = 3 - 8 MPa)
	=	10^{-10} - 10^{-9} cm/s	(in-situ borehole plug)
Interface shear strength	=	3.9 MPa	(assumed uniform shear distribution)
(L/D = 2)	=	17 MPa	(peak from exponential distribution)
	=	3.4 MPa	(residual from exponential distribution)
Drying Conditions:			
Hydraulic conductivities	=	10^{-6} - 10^{-5} cm/s	(dried out plug, unconfined)
	=	10^{-8} - 10^{-7} cm/s	(re-saturated plugs, unconfined)

(ii) Cement plug or seal should be installed at the intact portion of the opening.

(iii) Plug length-to-diameter ratio should be greater than four.

(iv) The cement plug performance is not sensitive to the conventional drilling methods used to produce the borehole.

(v) The expansive capability of the cement provides hydraulic and mechanical bonds at the plug-rock interface, and hence increase the shear strength at the interface.

(vi) Drying increases cement plug permeability and decreases mechanical and hydraulic bonds at the interface. Performance does not fully recovered upon re-saturation.

(vii) Installation procedure significantly affects plug performance.

Since the wastes are planned to be emplaced in the unsaturated zone above ground water table, the performance of the cement seal under saturated condition which was not part of this research effort, should be investigated. Outlined below are some of the recommendations for future research. They are proposed to investigate and answer the remaining knowledge gaps and uncertainties with regard to sealing of boreholes at the proposed repository site.

(i) To investigate the size effect, cement borehole plug with diameter greater than 15 cm should be tested.

(ii) Gas permeability testing should be performed to determine the cement permeability under unsaturated conditions, and to determine the minimum moisture content of the surrounding rock mass required to maintain acceptable cement performance.

(iii) Cycles of wetting and drying may be imposed to assess the long-term performance and deterioration of the cement seals.

(iv) Interface shear strength of the plug should be measured by applying hydrostatic loading (using gas or fluid) on one end of the plug. This will provide a more realistic loading configuration than the push-out testing.

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Panel Discussion on Rock Mechanics Issues in Repository Design

Panel Moderator:	Z. T. Bieniawski, Pennsylvania State University
Panel Member:	K. S. Kim, Columbia University
	M. Nataraja, U.S. Nuclear Regulatory Commission (NRC)
	M. Voegele, Science Application International Corporation (SAIC)
	E. J. Cording, Nuclear Waste Technical Review Board (NWTRB)
	H. N. Kalia, Los Alamos National Laboratories (LANL)

Z.T. (Richard) Bieniawski:

I am pleased to tell you that we have assembled an excellent panel of experts all very much experienced in the area that we are discussing. I would like to thank the panel in advance for agreeing to participate and being so cooperative when I called them to attend as well as to prepare their remarks.

I am going to introduce the panel members to you, after which each panel member will deliver about a 10-minute statement about their views on aspects of rock mechanics integration in repository design. Each one of them will finish off with identifying a few issues for discussion. I will begin with giving you a list of the issues that I have compiled so that between all the discussion here and our issues from the panel, you (the audience) will judge which of the issues you think are important or not important, and we then will ask you to elaborate, ask questions, contribute to discussion, so that we can help the workshop organizers in identifying the rock mechanics issues for repository design.

I am going to introduce to you the panel members in the order in which they will speak. Some of them have already been introduced to you so I will only refresh your memory.

Professor Kun-Soo Kim from the Henry Krumb School of Mines, Columbia University, is a specialist in the area of rock mechanics testing. He appears here before us bringing his experience of directing the program on rock mechanics testing during the Basalt Waste Isolation Project (BWIP).

You have already met Dr. Mysore Nataraja who has been with the NRC since 1982 and today is the senior project manager.

Dr. Michael Voegele does not require much introduction. You know him well and he has been very vocal already during the day asking questions and making contributions to the discussion. He is the project manager for Science Application International Corporation (SAIC) on the Yucca Mountain Project. He has over 17 years of experience in salt, basalt, crystalline, and tuff repository programs. Within the Yucca Mountain Project, he worked in the areas of regulatory compliance, environmental assessment, and site characterization plan development.

Professor Edward Cording is very well known and we are particularly delighted to have him here. Dr. Cording is a professor of civil engineering at the University of Illinois, and he is a past chairman of the U.S. National Committee on Tunneling Technology. He is a member of the National Academy of Engineering, and most of all he is a member of the Nuclear Waste Technical Review Board (NWTRB).

Finally, Dr. Hemendra Kalia, who has been employed with Los Alamos National Laboratories as project leader and technical support assistant manager for engineering and field operations. He has a special position coordinating the activities of various testing programs for the Yucca Mountain site and will explain to us some aspects of interaction of the various programs as well as what is happening at the moment at Yucca Mountain.

With these words, I would like to begin by briefly giving you my own opinion as far as the various issues are concerned. Based on the presentation I have given to you today and also on my own experience, I have compiled a list of ten rock mechanics issues. I will briefly elaborate on them and then ask you to please consider them in the spirit in which they are given: to stimulate discussion. So, starting with my "Top Ten List" of rock mechanics issues, first of all, I believe that we do need to review the specific design needs which should be provided by M&O (Management & Operating Contractor) so that the rock mechanics community would know which parameters or which aspects they consider important for design.

Secondly, we have already talked about thermal loading and panel members will address it. It is a rock mechanics issue as a result of consequences of in-drift emplacement instead of borehole emplacement. The concept of rock mass strength needs to be examined, as does the question of what techniques to use and what assumptions to make in the determination of *in situ* stresses. There is still a lot of controversy concerning seismic loading. Issue number seven is redesign of the core test area which apparently already is being done, and also the matter of planning of test alcoves for test purposes. This would then involve the need for a prioritized program of *in situ* tests in the core area. Item number 9 are the consequences of using machine excavations instead of drill and blasting. Finally, as I indicated earlier, I believe that we need some integrating overall design methodology which should consider design principles and emphasize the concept of moving from objectives through design into construction.

With these words, I would like to introduce Professor Kun-Soo Kim who is going to tell us about BWIP and his views on rock mechanics issues.

K.S. Kim:

I am very glad that I have a second opportunity to mention my experience with the BWIP. I hope that I do not give an impression that I am trying to beat a dead horse again and again and again. That is not my intention. We learn from a bad experience or good experience because we are intelligent human beings. I just want to give a brief walk through the testing and design exercises we have gone through during our work for BWIP. I want to show you the testing we considered

"Top Ten List" of Rock Mechanics Issues for Repository Design	
1.	Review of the specific design needs (to be provided by DOE/M&O) concerning new thermal loading strategies, tunnel dimensions and layout, retrievability period, repository capacity and design alternatives (e.g. multilevel repository).
2.	Effects of different thermal loading options on rock mechanics data needs.
3.	Consequences of in-drift emplacement instead of borehole emplacement.
4.	Concepts for rock mass strength in design using the criteria of Hoek-Brown, Yudhbir-Bieniawski and Mohr-Coulomb and incorporating TBM effects; *in situ* confirmation tests.
5.	*In situ* stresses, techniques to be used, assumptions to be made (e.g. is the intermediate principal stress neglected) and design values.
6.	Seismic loading: current probabilistic seismic hazard analyses may be defective for the most important earthquakes, i.e. those with M>5 on the Richter scale.
7.	Redesign (simplify the layout) of the core test area, including the possibility of a multilevel repository, reducing the number of underground offices, store rooms, shops and warehouses, and using full-face TBM excavation. Test alcoves should not be excavated by drilling and blasting.
8.	Prioritized program of *in situ* test in the ESF core test area, complete with specifications for the configurations of the test alcoves.
9.	Machine excavation effects on rock mass classifications and geologic data collection.
10.	Incorporation of all the above into the Systems Design Methodology using the six design principles for rock engineering, with emphasis on "design for constructability."

at the Near Surface Test Facility (NSTF) to support design activities. We did, of course, launch a very elaborate site characterization study. We did the joint mapping and geologic mapping, came up with RMR, Q rating, and so on. We planned full-scale heater tests 1 and 2, block test, time scale test, room scale test and supportive technology development test. We developed a testing logic. We wanted to characterize the site first. From the knowledge we gained from the site characterization study, we prepared a block test to measure rock mass properties such as rock mass deformation properties as a function of pressure and temperature, and rock mass thermal properties as a function of a pressure and temperature. Those numbers we obtained from the test or the functional relationship were input into the design analysis, such finite element analysis or whatever analysis method we were using. Then, we conducted a heater test, of course using that numerical model and the numerical scheme we developed, to simulate the emplacement of a heater in the underground repository condition and see how the rock deforms and whether we can predict the deformation and response of the rock to the thermal loading. We then went planning one scale bigger to a heated room scale test, where we could heat up the whole room and see whether the room collapses or not. Of course, that is an exaggeration. The next step was to emplace actual nuclear waste, vitrified waste, 5-year, and 10-year old spent fuels. We made an elaborate plan to conduct all those tests in the NSTF and finally, as Dr. Nataraja mentioned earlier, we mobilized the drilling rig. The huge drilling rig moved through the town and everyone of us working on the program held our breadth and we had to hold our breadth for some 5, 6, 7 years. Finally, with Congressional help, we didn't have to hold the breadth any longer.

I would like to spend a few more minutes to talk about jointed block test. This jointed block test that we conducted is somewhat similar to the block test that the crystalline rock people did, and the one that Mike Voegele was involved in. We learned from their experience quite a bit and also our experience was shared with the G-tunnel people. This block test consisted of a block we cut out of the rock to which we applied mechanical and thermal loading. Those of you who are involved in the design of it know how complicated it is. I just want to remind you of the complicated nature of the design, execution, analysis of the results, and the extraction of the results to a useful form for the actual design. Originally, we planned to conduct some heater tests on the floor and in the wall of the NSTF. Because of the budget constraints, we finally conducted one block test, not because we believed in it; that was the best we could do. Even one test was really difficult. Because of the geology, the columnar basalt, we had to cut the rock and conduct the test in the wall, so that we could characterize the anisotropic behavior of the basalt block. We cut a single slot and then conducted pressurized jacking tests. We elevated the temperature up to 100°C and we got stress-strain curves which were pretty reasonable with hysteresis loops. As the cycles continued, as temperatures were raised, the hysteresis loops became a little bit more pronounced which was anticipated. We were pleased with the observations and we cut three other slots and installed the instrumentation. Some of our overview panel members expressed a concern about us conducting a "swiss cheese" test, and the concern about the space the boreholes were occupying, was a real concern. However, we had to install instrumentation and there was no other way than using penetrating boreholes. We had to minimize the number of the boreholes and as a result, came up with an ingenious idea of measuring deformation properties of the rock mass using an optical device called the BDMS (basalt deformation measurement system). The BDMS itself costed a lot to design, fabricate, and test. However, it was a very key instrument. Some of the data, however, we were not able to interpret and did not make much sense at all. Michael Hardy and his colleagues carried out a very elaborate numerical analysis. They found that the stress distribution in the block was no really uniform. However, the data analysis was done under the assumption that everything is uniform but that is apparently not the real situation. When we analyzed the deformation data from BDMS, we assumed that the block was experiencing a uniform loading.

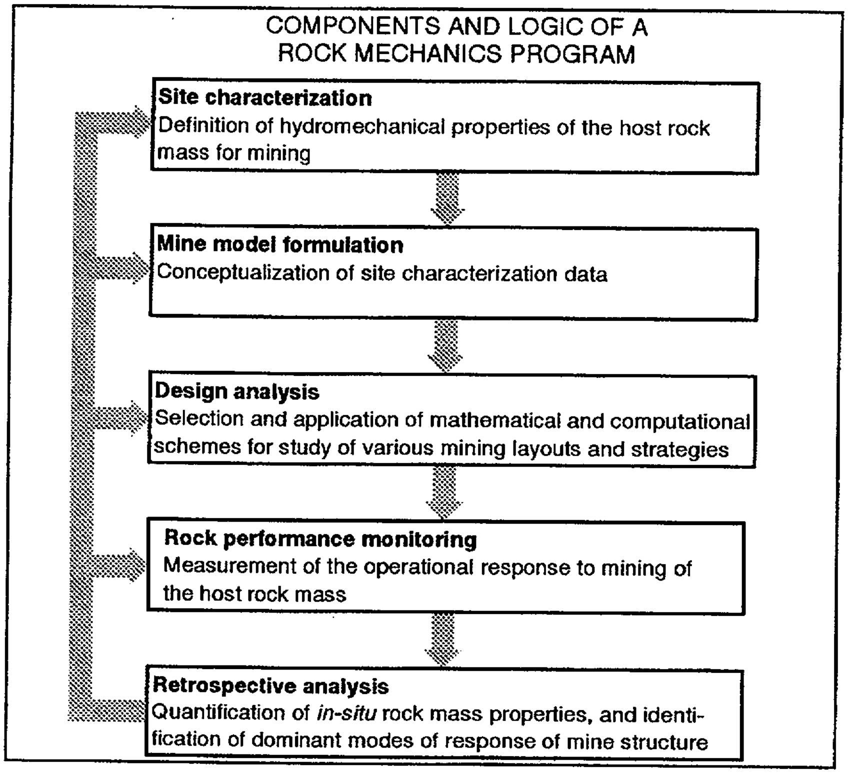
Why am I discussing all these matters after the fact? This is a forensic analysis. What did I learn from this exercise? The importance of the planning effort. This program is a very schedule driven program. I talked about a regulatory driven program. The regulatory driven process is important, but when it comes to conducting the tests, it is very schedule oriented. We have very little time to plan appropriately, but unless we plan properly the results will not be good. We all know that in the test design and numerical modeling we can carry out only one test, maybe two, in some cases. How can we validate the results? My philosophy is that we have to design the test in such a way that we can come up with some mathematical relationship between rock mass parameters and the component parameters. Through field testing, we attempt to validate it, but we also do a very elaborate numerical testing. Numerical testing is less affected by the schedule and cost. Ron (Price) talked about the lack of budget earlier and I agree with him. We experienced the same problems. But in retrospect, I feel that we should allocate a significant portion of our budget to do the numerical modeling. Numerical modeling is just our intellectual exercise. We have the surplus of supercomputers all over the nation. This powerful parallel computation technique is available for us, and we have to take advantage of it. We have also the defense workers, the high caliber people in every corner of the national labs. We should utilize their expertise as well as the hardware available. Also, we felt that the understanding of hardware, testing of hardware including flat jacks, tender anchor system, MPBX, and BDMS is important. We really did not have a complete grasp of the performance characteristics of those instruments. This is the thing that we have to do now. We should not spare money because of whatever reasons. This includes other supportive tests, such as geophysical tests and equally important the QA program and management system. I know you have a superb management system and so on, but we cannot emphasize the experienced managers with the related expertise and good technicians and expert rock mechanics engineers. That is what I learned and I would be happy to respond to any questions you may have.

Z.T. Bieniawski:

We will do that later. Thank you very much. Our next speaker will be Dr. Mysore Nataraja from the Nuclear Regulatory Commission.

M. Nataraja:

I have already talked about what I thought were the important technical issues. I am not going to elaborate one more time about technical issues, but I will discuss a chart which many of you might have seen in the book by Brady and Brown. It starts off with the site characterization process and goes on to model formulation, design analysis, performance monitoring, performance confirmation, and/or design modification. Basically, this logic appeals to me very much because Dick (Bieniawski) was talking about the design methodology. I think that is probably one of the biggest issues in this program. We have to have a design methodology based on certain broad principles. I believe that this is as good as any other set of principles that you can have. If you look at 10 CFR Part 60, it is basically reflecting many of these things. I am surprised, in retrospect, looking at all the people who are involved in the different disciplines and the background they came from. Many of these concepts are reflected in the development of those rules, because it says that we have to have site characterization, where you go ahead and describe the geology, the hydrology, the geochemistry, and the rock mechanics and many other aspects of the site that are important for design. Based on that information, you conceptualize the site, the behavior of the site, the behavior of the rocks and so forth. You then use that particular conceptual model to perform your design and analyses using more sophisticated techniques such as analytical techniques (finite element, finite difference, etc.) to do some numerical analyses/calculations using some empirical equations. You can then go ahead and do the design. After you do the design, you go ahead and build it and pray. Subsequently, you monitor the



performance and see if it behaves the way you predicted it. If it does not, there is something wrong either in your understanding, the models, or in the data, so you go back and reexamine it. If your performance does not conform your predictions, you basically go back and modify the design as you go. This seems like a very logical approach, especially for underground construction where you have surprises after surprises. This is one thing I wanted to share with you.

The other thing that I want to share with you is a design methodology that we, in the NRC, developed looking at one very small aspect of the regulation, namely 10 CFR 60.133(i). This regulation is a design requirement for the underground facility which talks about designing the underground facility in such a manner that the performance objectives are met. It is a fairly difficult thing to accomplish when you sit down and think about how this would be done. Performance objectives, in plural, talk about all the performance objectives, preclosure and postclosure performance objectives. You have got to go back and design the underground facility in such a way that you meet the performance objectives. It is a sequence of operations that one performs. I am not claiming that this is the only way to do that. This is a way that we thought was acceptable to the staff, and Mike (Voegele) objected to me, saying that "you people (NRC) should be giving us (DOE) guidance at much higher level than going to such a lower level." I agree with him. We need to give DOE guidance and various other things, but this is one that we accomplished so I thought that I would share it with you. Basically, what we do is start off with asking a question. "How do you meet the performance objectives taking into account the thermal mechanical behavior of the rock?" The first question that comes to our mind is that "At a given site, do we have sufficient understanding and/or experience to make a finding that 10 CFR Part 60 performance objectives are insensitive to thermal loading?" It is quite possible that thermal loading may have no bearing on the performance objectives in which case we are really home free. Then, if the answer is yes, one can consider the underground facility design as independent of thermal loading. This may happen if your thermal loading is very small, for instance. Obviously, the answer is going to be no, so you go the next step and ask the question "Do we have predictive models to quantify the sensitivity of these performance objectives to thermal loading?" If the answer is yes, then again we have a fairly easy job. We have existing models and can use them and go ahead and do the designs. If we don't, then we go the next step where you conduct an examination and understand the thermally induced phenomena. This is where the first technical issue that I presented this morning, the TMHC interaction technical issue, comes into play. We have to develop sufficient understanding based on which we develop some predictive models. This is because there isn't a single test that would give you the answers for hundreds of years so essentially you have to make predictions based on predictive models. Next we have got design criteria and goals that we have developed based on regulations and any other needs of the designer. We go to the next step and use the predictive models and design the underground facility and then check to see if we meet the design criteria and the design goals. If we meet them, fine, we go to the next step. If we do not, you keep iterating until you meet the design criteria. We then go back and check if the performance objectives are met. Take for example, whether given a thermal loading, can we meet the retrievability performance objective. If the answer is yes, fine. If it is not, then you go back and iterate your design until you can meet your performance objectives. And mind you, we have to show this by analysis because many times you may not be able to actually demonstrate that at the time of licensing, or you may be able to partially demonstrate it and not a complete situation where you can demonstrate hundreds of thousands of packages of retrieval. Sometimes even after you do all these things, you may still not be able to meet the performance objectives. In this case, you have to examine whether this has anything to do with the underground facility design at all. It might be another problem. However good your design might be, you may not be able to meet the performance objectives because of the nature of the site or some other problem, in which case you have to determine that this is independent of this and go about looking at another site or whatever. In any case, when you go about this kind of logical iterative process of looking at the design criteria and the performance objectives and start thinking about those things right from step one. You do not think about meeting performance objectives after completing your design. You have to do this in parallel, keep all the requirements in mind before you start your design. Since it is such a complex animal, I think that is the only way we can accomplish our goals. When I say only way, that is the only way we could think about. There are obviously alternative ways of accomplishing the same goals in which case they (DOE) will demonstrate to us how to accomplish this goal.

In summing up, there are 10 issues that Dick (Bieniawski) put up. Out of these, I could find two issues that we had in my morning presentation; they are the thermal loads and the seismic loading. The other two issues I had were the retrievability and long-term performance of seals. All those four issues in my mind are still important issues that we have to keep in mind as far as the rock mechanics program is concerned. The most important one is the question of design integration. I think, if we keep all the technical issues in mind and do not do a good job of design integration, we will not be able to meet our goals. When I talk about design integration, at this stage, I specifically mean the integration of the ESF design with the repository conceptual design. There is a lot of confusion when we talk about this issue with the Department of Energy. We use the phrase "design control process" which causes a lot of confusion and people go back and develop more procedures. I hope that's not what will happen. What we would like to see is not development of more procedures but implementation of good solid procedures which already exist and demonstration to other parties, the third parties, that such procedures do exist and are being implemented. That is all I have to say and we will probably discuss further this issue. Thank you very much.

Z.T. Bieniawski:

Thank you very much. Our next speaker will be Dr. Mike Voegele.

M. Voegele:

Well, I notice Raj (Nataraja) that you added three things to your speech to try to preclude me from making my comments. But you didn't box me out completely!

I think where I would like to start is to try to talk a little bit about what happened in the early 1980s up through the period of 1987 within the Department of Energy as a whole. I could recall from Kun-Soo (Kim) all of the field activities that were going on at the near surface test facility in Hanford. We had comparable programs in mind and underway at the Yucca Mountain program. I think the observation I would like to make about what happened in that timeframe has two dimensions to it. First of all, the NRC had reviewed a draft site characterization report from the basalt program and had been very critical of the way in which the basalt program expressed the types of programs and activities that would be undertaken in the site characterization program. The DOE turned its attention very seriously to that criticism and tried to develop a methodology for explaining why we will do certain tests. The methodology of necessity derives from the regulatory requirements which you will find in Part 60 and the subtlety, the second part of this that I'd like to comment, it is very easy to argue that everything that Kun-Soo (Kim) had discussed was in fact justifiable in a site characterization program, and in fact it goes one step farther. It is very easy to convince yourself that the NRC is demanding that you do all of those things. The result of falling into that cycle is that your characterization program just keeps getting bigger and bigger and bigger. I will use the word "without focus." Not to say that everything that we can come up with, any principal investigator can come up with, is useful information. I am sure it is all useful but the argument keeps coming back to whether or not you really have to do that to move the program forward. You might find me talking in and out of different topics as I step into this.

One thing we did with the NRC, and this is a negotiated agreement with the NRC, is that we agreed that we would develop some particular types of tables for our site characterization plan. This is the strategy that was used in the site characterization plan and I intend to argue that there is a basis in place for integrating this program and I will also accept the criticism that we are not doing a very good job of it.

The concept of coming to grips with resolving all the questions that you have to truly answer for the NRC really is one of figuring out what the requirements are that you must address and what kind of system that you are going to use to address them. From that you can develop strategies for dealing with the NRC for resolving questions and that results basically in performance measures. That is an important word "performance measures." Those performance measures are associated with specific licensing strategies. What drives the site characterization program theoretically at Yucca Mountain is not the study plans. It is regulatory requirements and the information that you need to address those regulatory requirements which can then be broken down into parameters that one can measure and one can use as a basis for calculations. That can further be broken down into the basis for your characterization programs. And so the intention in the site characterization plan for Yucca Mountain was in fact to have a regulatory, design, performance assessment driven methodology which would result in a definition of the important test that you would need to do to characterize the site. I will argue that is a credible basis for integration of the program and we will see why it hasn't worked very well. I wanted to stick something else in there up front to start with that.

With respect to geotechnical issues, Raj (Nataraja) pointed out quite carefully this morning that there are some additional design criteria for the underground facility. These additional design criteria as was pointed out do roll up into helping you understand how you address the performance objectives and demonstrate compliance with your site. The ones that really have a lot to do with geomechanics address containment and isolation, site specific conditions in the rock, reducing potential for rock movement and limiting preferential pathways, designing your engineer barrier system to assist the geologic setting in meeting the performance objectives, and addressing the thermomechanical response in a way that shows that you can allow compliance with the performance objectives. Those are, at least at a high level, and of course ceiling fits in this as well, what the geomechanics aspects are of the repository design. When we were developing the system description and when we were developing the performance methodologies, the design methodologies that are embodied in the site characterization plan, this is the part that dealt with geomechanics.

From a site specific perspective, the things that you will find in our site characterization plan which we are doing testing to address with respect to contributing the isolation and containment, we are looking at how to identify the areas that are unfavorable that you might have to skip or you might have to isolate, how can you adjust thermal loads for local conditions and of course water also comes in there as well. Within the SCP with respect to limiting excavation and induce permeability changes, we were worried about control blasting, we were worried about subsidence, and we were worried about how we could use backfilling effectively to help us control those. Thermal mechanical response from our perspective, we were very worried about temperature changes in selected barriers, particularly if those barriers contained minerals which could undergo geochemical change in response to temperature. We were worried about continuous joint slip; we set that up as an original design goal to try to see if we could develop a repository that had limited continuous joint slip in the borehole walls above the boiling temperatures from a different perspective. Some geomechanics issues related to the seals have to be considered so that the boreholes do not become pathways that compromise performance. You have to select materials and place them in a way that reduces those pathways for groundwater movement and of course there are geomechanics aspects to that as well.

The reason I chose this particular set of materials to present is that this is very different from the way the DOE went forth originally with the basalt site characterization report in terms of organizing how they might derive a site characterization program. Although there are a lot of people in the program who tried to pretend that they did not have anything to do with this performance allocation process, it is one that the DOE has never done a good job of explaining to people. It was not a very effective tool for us in the site characterization plan review because of the way the NRC elected to break the SCP apart for review purposes. We have had a lot of dialogues on this as well and I think the NRC has found that we were trying to tell them things that they did not find because they were looking in different parts of the SCP. We found we did not do a very good job of anticipating how a document like that would be reviewed so we could have helped the NRC review it. We have gotten smarter over time, I think with respect to that.

I wanted to talk a little bit about where, in terms of the performance objectives, the kinds of things that we think we have the highest uncertainties with and this is from a tester's perspective. This is from the perspective of how are you going to test these sorts of things that you have to address in the additional design criteria so that you can convince the NRC that your performance objectives are met. Of course, one of the 10 CFR 60.113 performance objectives is the waste package life and this is kind of a systems, multiple systems approach. We tried to break down our uncertainties into these six different categories, waste form, mineralogy, geochemistry, hydrogeology, and geomechanics. We looked at the system itself from the repository through the engineered barriers down through the natural barriers at Topopah Springs down into the groundwater unit. We find, at least from a geomechanics perspective, the waste package life and the geomechanics issues are really at the repository level. Those have to do with boreholes. If we had stayed with the borehole emplacement, they might have to do with the borehole collapsing around the waste package. If we go to much larger drift emplacement, and incidently, I think I would like to address two comments that were made today. In drift emplacement of a multipurpose canister is really not that much different technically from what we had before. In one of our alternatives which was in the sidewall long boreholes, the NRC had questions about retrievability for that particular configuration. We have those same questions today if we are going to have an unshielded multipurpose canister pushed off into a drift which we are going to seal at both ends. How are we going to monitor that? How are we going to demonstrate that we can actually retrieve that material should it become necessary?

The release rate calculations also have some geomechanics aspects of it at the repository horizon that have to do with the opening and closing of fractures and how that interacts with the controlling of the water that might access the waste package itself and contribute to the corrosion. In the groundwater travel time, we find that the uncertainty in the geomechanics issues comes to importance at the repository horizon level, the Topopah Springs, the Calico Hills, on down to the groundwater system. That is simply because of the way the disturbed zone definition is working its way around. The disturbed zone is one of the pieces of part 60 that you have to ask yourself how it came out to be exactly the way it is and whether anybody ever intended there be so much argument over the disturbed zone. That particular piece of the regulation is ostensibly the part which somehow looks at the performance of the natural barriers independent of there being a repository there. On the other hand, you must look at the effects of the repository to calculate the extent of a disturbed zone which is the point at which you start the calculation of this groundwater travel time. I do not want to suggest that anything that is said today about the disturbed zone is either right or wrong, only that it is an enigma to some people how it got in there in the way, shape, and form that it is in there.

With the total system I think everything that we've talked about sort of rolls up into that outcome. This is really just a synthesis. Everything that is important for waste package life and release rate and travel time is also important for the total system performance calculation release.

What are some of those uncertainties in detail? I will summarize this kind of quickly. Starting from the pre-waste emplacement groundwater travel time, the geomechanical aspects have to do with construction induced effects and thermally created fractures and opening and closing of existing fractures. In the total system performance, we have borehole stability issues, creating new fractures, opening or closing existing fractures and the aspects of useable area, and flexibility to accommodate rock conditions. Within the waste package life, again we have a borehole stability and even though that says borehole stability, imagine that an in-drift emplacement of a multipurpose canister is not conceptually different from a borehole stability issue. If you have a rock that is big enough to fall down and puncture that canister that is an issue. You can see the role of thermal loading coming through in these as well how critical that is. Again mechanical effects and thermal effects are really, as you would expect, where the geomechanics issues are. Perhaps DOE is able to demonstrate, in fact, that these things that I have called "uncertainties" are in fact not significant issues. It may be possible that you cannot get a big enough piece of rock falling out of the roof at Yucca Mountain to cause damage to a transporter. Maybe the credible range of thermal loading that you would ever consider in a repository at Yucca Mountain will not open or close fractures to the extent that it will impact for compliance with the performance objective. Please do not lose sight of that. I know that when you are dealing with solutions that tend to have 1/R and $1/R^2$ floating around, you are always going to be able to calculate effects. The important issue is whether or not those effects prevent you from complying with the performance objectives or prevent you from presenting an integrated argument that your site will meet the overall performance objectives. Part 60 itself has the flexibility for you to go to the Commission and propose alternate performance objectives as well. I haven't mentioned yet as a geotechnical aspect of this, whether particular pieces of the repository could end up on what we would call the Q List. The Q List is that list of items which are important to radiological health and safety, or important to waste isolation and containment. If you can succeed in getting things off the Q List, the QA program controls that you apply to them are very different. I'm not saying that QA will go away, since DOE itself has requirements that you have to do certain levels of quality assurance, but the NRC's interest in a particular item becomes less important if it is not a Q-type item.

The way the site characterization plan intended to integrate the geotechnical design activities, is that we had postclosure design activities and postclosure performance assessment activities, preclosure performance assessment and preclosure design activities. Within the site characterization plan, you can find strategies for dealing with these various topics of design, postclosure waste package design, configuration of the underground facilities, seal characteristics, and postclosure aspects of performance assessment (e.g., containment, EBS release, 40 CFR Part 191 individual protection standard, groundwater travel time, etc.). The idea that we (DOE) had was that as the design was to be controlled as a preclosure design (given additional constraints and design criteria from the postclosure aspect of design), we envision an integrated design which was being driven by preclosure issues such as waste package production, health and safety, and waste retrievability. Regarding additional design criteria that should be driving your design in an iterative fashion, you should be coming up with system concepts, checking them out against this and going forward just as Raj (Nataraja) has suggested. The postclosure aspect would also be integrated and fed over to the preclosure issues. The way this worked in concept, was in fact an iterative design process, no different from what Raj (Nataraja) has laid out for one specific aspect of the design.

I am here to tell you that we are not doing a very good job of this. We need to find a way to focus the program more on the overall program design. I believe that the NRC needs to help the DOE, not by writing a NRC position on groundwater travel time even some subcomponent of the disturbed zone. I think that if you have trouble with the way the DOE is coming to grips with its overall design philosophy, we ought to be focusing on this for NRC staff guidance. If we (DOE) are not doing it right, if you (NRC) have problems with this concept and see ways to help us better implement it, I think it would be a good idea for the program if we could have interactions on that basis. My only comment to Raj (Nataraja) was simply that I think that we need to put some effort at a much higher level in the program, but really how do you make this work, how do you make this integrate a program?

For the record, I am not speaking for the DOE.

Z.T. Bieniawski:

Our next speaker is Ed Cording.

E.J. Cording:

I want to state up front, I came here today because I love rock mechanics. This is my area of background and it has been a fascinating field in which be involved. I think that if you would recognize that is where my interests lie and why I came here, I would then like to present some information on the way I've been looking at things. Again, I am not here to present the NWTRB's official position, although some of the things I will present may have been things the NWTRB has formerly stated. However, I'm thinking more in terms of my background in rock mechanics and the things that I see in terms of characterizing the site and designing the repository at Yucca Mountain. I hope you will keep that in mind when I talk about what I think are the critical issues, because I do not think it is rock mechanics.

The most important issue to me is site suitability. I am talking about site suitability as related to flow of radionuclides, the containment aspect, and the hydraulic characteristics of the underground in terms of the matrix and fractures that will control radionuclide flow. I think there has been more and more of an understanding in the past few years of the thermal effects on the driving forces for transport of vapor and fluid. I think that the thermal-hydraulic aspects are as key as anything in terms of site suitability. It seems to me that there are a lot of other issues that are very important ones, but I think that they are really design issues as how to design the site in such a way so as to accomplish the other requirements of making this a satisfactory design for a repository.

In terms of exploration to characterize the underground facility, it seems to me that the key issues are to be able to get this sort of thermal and hydraulic characterization done and do it as soon as possibly feasible. In doing the exploration, two of the very key points are to be able to accommodate the testing that has to be done to evaluate hydrologic conditions and to be able to begin the thermal tests. The last thermal testing done on this project was in 1989, and it needs to get started again regardless of when we think the results will be available. It needs to be started again so that results will be available as soon as possible for use in further decisions that are to be made. To me the exploration needs to focus on getting this sort of thing accomplished. One of the main concerns I think has been being able to do the exploration in a timely manner, cost effectively, and to meet schedules. Right now I think there is quite a bit of effort that is going on to try to make the schedule more efficient and to reduce cost. It has been a very costly environment. I think you put factors on it and the factors (estimates) turn out to be much lower than what it is actually coming out to be. I think there is a lot of money being spent on the program to excavate the size of the facility that is there. Just to excavate that sort of facility would be a several hundred million dollar project, and the way the cost has been going at this point, it is going turn out to be much more than that to do what is has been planned. The other part has to do with the design and repository, where I think that particularly in terms of the rock mechanics it is really crucial that we understand behavior. Some of you in the meeting have talked about the importance of looking at the behavior. This is one of the things that models do not always do very well, particularly in the area of fracture, failure of rock, interaction of the tunnel and the tunnel support system under various loading conditions that can take place (e.g., body forces, thermal effects, and seismic effects). I think that one of the things that can be done here, and I think Mike (Voegele) mentioned this before either in some of the general discussions or in his presentation, is to focus on the importance of seeing what happens during construction. I think that is one of the key parts of the rock mechanics program, to be able to not only log the conditions that are present but to observe the behavior. The other part of the behavior that will be helpful to be able to observe is the thermal portion and looking at the thermal-hydraulic testing that needs to be done. Of course Sandia has been making plans for testing thermal mechanical processes, and I think that those programs obviously have to be put together to make them cost effective and efficient. I think that there is much that can be learned by looking at the thermal mechanical behavior in terms of the long-term behavior of the support system. I think that some of those effects are not going to be extremely pronounced, but are present. Some of them will be subtle effects and it is hard to predict those conditions without actually doing tests. I think that we should look at not only the rock behavior, but also at the rock lining behavior. Thus, if we are going to be putting up shotcrete as part of a lining, we should look at various ways of putting in the shotcrete so that it does the best job of being able to work over the life of the facility.

The exploration, I think, has a tremendous amount of opportunity to make measurements of movements as the tunnels progress. I think that to a large extent, we have to treat it as well-planned targets of opportunity here because again, the key thing in terms of schedule at this point is related to this other aspect. In looking at the initial ground support and the design of the facility itself, it certainly has to be compatible with any potential use of the Yucca Mountain site as a repository site. But the principal thing is that certainly one has to design and construct a facility that is safe and stable as one excavates it. I think it is extremely important that the support system is being used for an exploratory facility be integrated in such a way that the construction process can be efficient. Again it is not because one wants to set tunnelling record, it is because you want to get down there and do the work that needs to be done in the short time available. Regardless of what schedules there are, there is a need to get on with it and there is a need to minimize the very large cost that exists because of having mobilized such a large infrastructure to both carry out the construction and to evaluate the results. I think the interest of the DOE has been to set things up so that you don't cover up the rock, but you allow the rock to be viewed. This would be one of the areas where there is an effort to do that as much as possible within the constraint of number one.

Finally the other part is to make it fit into the repository, and I think that we can talk about several different items on minimizing things that could affect the potential repository. It is not that one has to build anything that is used or will be in place as part of the repository, the point is not to affect it. I think that it is hard in some cases in the planning at this point or in the actual design to separate out, what do we really need here for the exploration, making sure that it does not affect the repository as opposed to saying do we have to put in something that will last or provide support over say a potential life of the preclosure period. One of the things we had commented on early in my participation with the Nuclear Waste Technical Review Board, was to try to minimize things that would also affect the testing. This had to do, for example, with the introduction of water where one could not tell if you were looking at perched water or water introduced from the drilling. Another concern related to the mapping of fracture conditions and being able to tell that they were natural fractures and not getting perhaps overwhelmed by a lot of blast fracture damage. I think that the boring machine excavation is a big help, in fact, for both of those objectives.

Certainly, the introduction of materials that are potentially deleterious, such as organics that could contribute to colloid transport, is a concern. To minimize rock loosening and damage around the opening, I think that the construction techniques with boring machines will tend to do that in early support behind the machine. But at the same time, there will be places where overbreak can be expected to occur, things will loosen up, that you may have to place additional support in some areas. I think that one should recognize that these things are going to happen and that this is something that you have to be able to accommodate without it being a surprise to the point that you really stall the whole program just to deal with something that is going to happen. That is, I think, an important part of this. The fact that you have caused some fracturing and some loosening around the opening, may impact extent of your seals in the postclosure period. Certainly, the construction could change permeability immediately around the openings, but it seems to me to be a design issue and you have to recognize that that is going to occur. Certainly this is not a site suitability issue, you work to achieve this but have procedures that you can take care of the other aspect of it. The other thing is that certain drifts that go in and are not going to be used as part of a permanent facility. It could be that one gets down to a certain level and drives even a main drift and you say it is really not at the right place. Having approaches where one can abandon drifts should be something that, I think, is part of the program. This should be able to be done without having to affect the suitability site as the repository, or something that is acceptable for that condition.

Looking at the tunnel boring machine that is going into the portal area, it has some really very excellent capabilities in terms of limiting overbreak to benefit seal performance as well and covers over the hydraulic systems to limit the loss of fluid into the tunnel floor. If you consider the section back behind the tail of the front end of the machine, there are grippers to expand vertically, as well as grippers that can expand horizontally at that location to obtain almost a full perimeter grip against the rock. As such, if one is dealing with very bad ground (e.g., squeezing ground) conditions this machine has some real benefits. The Yucca Mountain site, in general, will be much better than that. The support system can go in with an erection system back behind or within the protection of the tail section of the machine, such that it is possible then to place a circular steel support in poor quality ground or to go in with some sort of a standard rock bolt pattern, with perhaps some mesh and straps to hold in the top. I think they (DOE) are going to be putting some slots so that they can do that under the protection of that tail shield. This does occur some distance back from the front end of the machine, and in general there could be some small amount of loosening if rock is able to do that in terms of the presence of the fractures in joint system. It should be quite feasible to design for example a support system that can keep up with this. There is some concern that if the machine gets up to higher rates of advance that one cannot keep up with it in terms of being able to put in the support system (i.e., a permanent support system, or a support system that would have a life of 100 years). The most important thing is to get it in place for the life of the repository and I think that a lot can be done there to come back in and support it if it ever becomes a repository at that time. I think that there's some advantage to being able to evaluate conditions in the project before you come back in and come up with your final design.

Looking at just a couple of other items, in terms of the repository preclosure aspects certainly the size and shape of openings have a lot to do with stability. The excavation method and the initial support are of interest as well as the long-term stability for both existing and new excavations related to things like corrosion resistance and fully encapsulating bolts and grout and the thermal effects on both the rock and the lining. Regarding, maintenance over the preclosure period and possible recovery, it needs to be assumed that those sorts of things will occur and need to be able to be taken care of. I had looked at some of the stress effects and because the rock is of relatively high strength, at least at repository level, with respect to the stress level in the ground you are not going to see the fracturing or the slabbing conditions that we saw, for example, in the tunnel beds in the Nevada Test Site. However, adding the thermal effects to it, you will see higher stresses developing with time because of the thermal effects and I think that the effect of that is something that needs to be carefully looked at. I think the anticipated stress levels are going to be at such a level that they would not create really severe stress effects (e.g., not as severe in terms of the ratio of strength of the rock to the stress level as we have seen in the Nevada Test Site in some of the G-tunnels and the tunnel beds within Ranier Mesa). I think that some of the observations that we have had in those other projects studying stress effects, even though they have not been thermal, give some sort of perspective on what we can expect at Yucca Mountain. There were stress slabs that existed in a 120-foot diameter cavern that was built in G-tunnel many years ago, that formed on the perimeter. They were basically extension type fractures forming parallel to the major compressive stresses in which the direction of excavation took place. Even though there was blasting there and some blast damage, I think principally what we observed was stress induced effects because the strength of the rock was on the order of 10 MPa and the actual stress in the ground at that depth was about 7 MPa. Based on elastic analysis and taking into account stress concentration factors, the stresses would have exceeded the strength of the rock. Because the rock was not a very stiff material and did not store a lot of energy, cracking and popping could be heard when these failures occurred but not a major release of energy. Looking at other scales (e.g., tunnels), we would see tendency for stress slabs to occur on the sidewalls. I would like to briefly mention one other project (the Drackensburg pump storage project) in South Africa, where the facility in mudstone was about 500 feet deep with horizontal stresses about 2 or 3 times the vertical. There we noticed that instead of the fracturing being on the sides of the boreholes, we could see fracturing taking place basically at the top and the bottom of the boreholes, and the phenomenon was time dependent.

The condition we saw in the roof of the tail race was that we could see some fracturing in the shotcrete itself because they continued some excavation below in the bottom which resulted in a little bit more deformation. In addition, we noticed fracturing right down in the roof of the tail race. The sort of thing I think we need to look at is what long-term effects could occur on the tunnel lining. Again, I think it is not a site suitability issue but a design issue that needs to be investigated further during the exploration program.

Z.T. Bieniawski:

The final speaker of the day is Hemi Kalia.

H.N. Kalia:

I think it is an advantage to be the last speaker. I would like to commend the organizers for such a good workshop. I think we can learn an awful lot from what is being presented. The only plea for the future, is to keep it small because normally this can grow into such large conference that it loses its purpose. I think it is a very timely workshop. As Ed (Cording) mentioned just now, a TBM is poised to get rolling. We know a lot of frustrations, we know of a lot of slips in the schedule, we know a lot of problems, since the beginning (1979-1980) when the whole program started. There have been a combination of problems, changes in priorities, changes in goals and objectives, and changes in needs and resource concerns. We have three programs, as we heard before, and it will take billions and billions of dollars to do these things. A two-week delay from August 8 to September, perhaps the end of September, is not likely to have a significant impact on the project. I think it will get going. The design is going to come out, it will be accepted and we will proceed.

Like the other two speakers, I would like to indicate that I am here speaking on my own behalf, not for the Los Alamos National Laboratories nor for the Department of Energy. I think I will talk about basically two things: (i) where we are with respect to underground testing, and (ii) importance of rock mechanics to the repository performance. Before I talk about the underground test area, I would like to point out that Los Alamos has an office in Las Vegas called the Test Coordination Office, a function that the Lab has had since the program started. Dr. Ned Elkins is the manager and also Deputy Program Manager for this Los Alamos field office. I think the Test Coordination Office plays a very critical role in the program in trying to pull the various priorities in focus. It looks at construction issues, for example, how they may impact the test community. It looks at the test community needs: their safety aspects, their requirements for power, water, air, space, and other requirements. It also gets beat up all the time because they (DOE) say, for instance, the Los Alamos test coordinator came and he told me he needed the alcove 30 feet wide and 20 feet high, whereas we had it designed 20 feet wide and 30 feet high, so start again. As a result, we serve as a bridge.

Second, there are several issues in my point of view in the area of rock mechanics. For the most part, I have been a field experimentalist. I have run field experiments in hydro projects and underground mines. I was involved with the underground testing in Germany where we used radioactive sources to see the impact on the canisters and salt material under thermal loading. I think the key is changes in the rock mass environment due to mining. In the ESF phase of the program, we are not looking at thermal loads or at radiation aspects. We are looking at the overall rock mass behavior. We also need to understand more about the thermal loading impact on the stability, as Ed (Cording) was pointing out. This understanding will involve very long duration tests. As Kun-Soo (Kim) pointed out earlier on, it takes several years to plan and implement these tests. This brings on a level of frustration as funding requirements are very high and competing demands continue to come into play. On top of this, the tunnel boring machine is a very expensive machine to operate and maintain.

The third point is instrumentation and testing. I mean, most of the experiments that we do in the field are of short duration with a limited objective. If you look at a mining operation, the intent is to determine the stability conditions for the next few days or a week or a few months. We are not looking 10 years, 20 years, 30 years, 40 years ahead. Consequently, the data that you look at in the literature are very limited, for example, on the performance of instrumentation, how good it is and what it really means. There are three concerns that I believe we need to be thinking about.

The general tendency is to take rocks and keep crunching them, and obtain a lot of parameters. We then run large codes and perform calculations, having very limited rock mass behavior information. We are really dealing with the entire rock mass behavior which is not truly indicative in the small intact samples. Also, the modeling results are, what you perceive of as a modeler, how the rock ought to behave. That is the way I look at it, having not done much modeling myself. I think it is something you perceive. You assume that rock will behave this way, and here are the key parameters I am going to input. The results comes out and indeed, this is how it behaves. In underground operations, it doesn't quite behave that way. We must understand that and temper our thought process with that. I think this is very critical for both the repository and the ESF.

Large-scale prototype testing, I think, plays a very critical role; however, very limited work has been done. I think Mike (Voegele) indicated some work in the G tunnel back in the late 1970's. A large, but limited, G tunnel experiment was conducted in 1988 and terminated sometime in 1989. A single heater test and a horizontal mode have been driving the program since then as a source of data. The reason, of course, is that it is a very expensive test to do, but very critical. There are a lot of interactions now with international programs, trying to learn from the Canadians, Swedes, and others to pick up some information, but it is still very limited.

Consequently, most of our reliance is on computer codes and supercomputers that have a lot of power to perform calculations and produce a lot of output. But a question comes up, "is it really what is going to happen?" You cannot answer that because you do not know.

In talking about the designer's options, one ends up using standard engineering practices which lead to super conservatism. The designer has been told to design for 100 years, 200 years, maybe even 1000 years, and make sure there are no deleterious rock movements for a long, long time because of the waste package and due to issues of importance to safety and waste isolation. The designer has very limited information to work with, so the design ends up being a very conservative design. Consequently, the cost goes up for building these facilities. You may end up with having to put a fully groutable, stainless steel rock bolt that would probably never rust. The question is, what are we using that for, and what do we know about the rock mass behavior that tells us we must do that. But the designer has no other option; he has been given a set of requirements and what he should design it to. I think the key really is to have the long-term monitoring information that we must gather, and that takes time. The repository design ultimately must depend on what we learn out of the ESF. Perhaps as we look at it now, if the construction does start up early next month, we are looking at another 4 to 5 years of excavation. A tremendous amount of geologic information, rock mass response, fracture behavior, etc., will be obtained. Perhaps at that time, we will back off and look at our design. I think you were saying we should get real data to work with, to do these things. Yet, the current design must go ahead as though it is going to stay there for 100 or more years without being altered.

The one key issue that I think will probably take a lot more time to understand, will be to understand long-term thermal mechanical behavior of the rock. When I was in school, a Russian scientist came and put two electrodes into the rock, turned the power on and the rock became hot. He took the plugs out, put water on it and sure enough it broke. In our case, we are looking to raise the temperature of the rocks several hundred degree Celsius. Is it possible that a sudden in-rush of water, for whatever the reason, could result in major failures? Can we really understand that behavior? How does a sustained thermal load change the rock mass behavior? I do not think we have any data to tell us what will happen. The longest heater test that I believe has been run is probably 3 or 4 years at best. Sandia may have done heater tests at WIPP that ran for probably 6 years. However, I do not know of many heater tests that have run for a very long time, 10, 15, or 20 years. We do not really know how this rock or any other rock on the long-term will behave under sustained thermal loads. We are looking at very long duration testing programs.

To get such information, in my opinion, we have forgotten one of the old techniques that was used extensively before computers hit the market pretty heavily, and this is the concept when you do similitude model studies using dimensionless parameters and designing smaller experiments. I think a lot of the things we have learned about blasting came from this concept, where we took dimensionless parameters from the experiments. You could do lots of experiments for much less money, and it allows you to understand the whole process. Perhaps natural analogs, for instance, mines where the temperature is very high and remains consistently high for a long period of time, could provide useful data.

I think we have to think about sustained long-term prototype testing. Perhaps that is what we will end up doing in the ESF. As we go on with this site characterization effort, the first test or second test that we run, we will probably learn an awful lot as to what the rock is doing. We probably will not have had all the right parameters despite all our modeling efforts. I think we need to understand that.

The other concern that I have which goes back to my instrumentation background, is the evaluation of instrument's long-term performance. I wouldn't even have an instrument that could run for 10 years. If you can tell me they are doing the right thing, however, I would say that we do not know. A poor example was an experiment in Germany where we put some vibrating wire strain gauges in salt. They were able to calibrate them in the lab perfectly. When we put them in the hole, we could not calibrate them. We had voltage with respect to time, but what do you do with it? The question then is that when you talk about setting in the experiments for 10 years, 15 years, 20 years, can we really do this and do it reliably? What are we doing to improve that knowledge?

I sense a lot of frustration with respect to design and construction. I think I promised you, Dick (Bieniawski), that I would tell you about the underground facility with respect to the core test area. The concept now is that the entire ESF area is a core test area. The entire length (i.e., U-shaped construction loop) will be used for the purpose of opportunity. There is no dedicated block as the sketch showed, that was the old 2-shaft concept with a concentrated test area because that was all we had, a limited excavation. That is all really I have to say.

Z.T. Bieniawski:

I would like to thank all the panel members for their addresses and I now open the discussion from the floor. You are welcome to ask a question, make contributions, or ask anybody in the audience as well as those on the panel.

R.D. Manteufel (CNWRA): This is a question I had for some of the speakers, especially Ron Price, but I throw it open to anybody who would like to handle it. Why is there a lack of rock mechanics issues considered in the recent total system performance assessments that have been published? This symposium is looking at rock mechanics issues in both design and performance assessment, and DOE has spent a considerable amount of money and NRC has devoted a considerable amount of effort to performance assessment. The Nuclear Waste Technical Review Board has commented on that, focused in on that, and thinks it is a great way of integrating the program, comparing all of the processes, and trying to do a total system performance assessment. Some of the oversight committees for the Nuclear Regulatory Commission have come to the same conclusion (e.g., NSRRC, ACNW) and view that role of total system performance assessment is critical. I would like to emphasize that PNL has completed one total system performance assessment, Sandia has completed two, the most recent issued 2 months ago (TSPA93), DOE Intera/M&O recently completed a total system performance assessment, and the NRC has completed its first back in 1991 and has recently completed its second phase in total system performance assessment.

Z.T. Bieniawski:

So, your question now is?

R.D. Manteufel:

Why is there a lack of rock mechanics issues being included in these total system performance assessments. I will try and answer this myself, by giving three hypotheses; (i) rock mechanics issues are so unknown, we just do not have any models, data, or way to get a grasp of that for performance assessment; (ii) perhaps the DOE and the NRC's programs foster the isolation of specific disciplines and that integration, although it should be accomplished, is just not being accomplished because of the DOE and NRC's programs; or (iii) rock mechanics issues really do not have an effect on performance.

Z.T. Bieniawski:

Thank you very much. It seems to be a very good point. Would anyone in the panel like to comment?

M. Voegele:

There are at least as many perspectives on this answer as there are to the length of your question. I think the most important perspective that cuts right to the heart of your question, is that within the total system performance assessment approach, I do not see an attempt to model explicitly every parameter that could have an effect on performance. I think that the total system performance assessment codes are going to continue to be very, very high level codes that have to look at constituent behaviors in some sort of a rolled-up manner. I think that when you look at what they were doing to address the kinds of rock mechanics issues that we felt were important, you will find that the total system performance assessment calculations sort of bounded that problem by looking at the situation, for example, of what would happen if you had fracture flow versus what would happen if you did not have fracture flow. I think you have to take the next step and say, at least from some perspective, fracture flow could be governed or in fact some how impacted by thermal loading and opening and closing the fractures that might be due to mechanical excavation or some other aspect of it. I do not think that anybody is ignoring rock mechanics issues. I just think you are looking at the wrong level to see the rock mechanics issues coming out explicitly. There are so many other parameters in those codes that they have to look at that you won't be able to trace, you know, for example, the thermal effects on the aperture of a joint directly into that highest scale model. I think you will see it at some lower level, where somebody is doing some modeling to help figure out what are the constitutive relationships.

W.C. Patrick (CNWRA):

Although coming from the same organization as Randy (Manteufel), I would pitch another one maybe back in Randy's court and that is that PA programmatically, I would say "unfortunately," focuses only on postclosure performance assessment all too often. There are critical issues to preclosure performance. They are included in the regulation in 10 CFR 60.111, and I think we need to give some attention and some thought to those as well.

H.N. Kalia:

There is a preclosure performance analysis report that just came out, within the last month or so, and you might want to look at it.

M. Nataraja:

Actually there is a need for a development of a retrieval plan in the license application, which should be based on a preclosure performance assessment and the most crucial issue there would be thermal loads.

M. Voegele:

I would not suggest for a minute that you won't eventually find those documents, but I think that you have to be careful not to go looking in a PA total system performance assessment to find that kind of design information. I think that the program will have to force retrievability issues, will have to force radiological health and safety issues, which are PA issues in any sense of the term. You will have to force those through the design process. The people who wear the hats in our program that say "PA" on them are only interested in total system postclosure, 10,000-year EPA standard compliance. Things that they can put into their models will be rolled up from much higher levels.

F. Tsai (M&O):

Tomorrow I will present a predictive model results of thermal mechanical rock mass response along a drift. I think the M&O PA are looking at the geomechanics issues, although the current TSPA does not have it yet, nor is likely to have it in 1995. Under M&O PA, which is in the same level as Sandia's PA in the total amount of money in the 1995 budget. Inside the M&O PA budget, there is 44 k for geomechanics. We are doing geomechanics, but somehow the management feels the level of percentage can be allocated.

M. Voegele:

I do not want to pretend that I am responding for the Department of Energy in addressing budget concerns but truly, we have heard today that $800,000 a year is a small amount of money to be spent on rock mechanics testing. We have heard that the DOE does not want to spend money on geomechanics performance assessment. I can sit here and tell you why that is; this is a very expensive program to run. You look at the things that start from the top. Quality assurance is 10% off the top of the program. Management and administration is 10 to 15% off the top of the program. Environmental regulatory compliance is another 10% off the top of the program. Interacting with NWTRB and the NRC accounts for most likely another 10% off the top. There is not a lot of money available to do the fundamental research the way the country has structured the program today. I will quote Professor Cording, geomechanics is not a performance issue, it is not a suitability issue for the site, it is an engineering design problem. I think that if the program is turning its attention in the very near future to try to address suitability issues, some of the things which we, as people of a geomechanics background, personally take as very important, may end up being relatively low priority for a couple of years while the DOE spends a lot of its money on suitability issues.

R.H. Price:

I will respond to that. Out of that $800,000, 30 or 40% goes into QA, training, and administrative also. It is not just that 10% that covers QA. We just do not go in and run these experiments in the lab. It takes a lot of preparation and a lot of regulatory requirements that go into it.

B. Boyle (DOE):

I can address certain parts of the budget. Without going into details, there are always competing factors for the money and admittedly QA eats up some of it. But I would like to point out that the money you are going to get, Ron (Price), is based partly on your estimate and you will get the money to do the tests that you said you would do. As Mike (Voegele) said, there are competing things and perhaps some of the rock mechanics issues will not be answered this year. Perhaps they will be answered in a later year but the things that are going to be done over the next couple of years deal with what we really need. And some of the issues perhaps we do not.

Z.T. Bieniawski:

Bill (Boyle), may I ask you two quick questions, since you were good enough to identify yourself speaking for DOE? There is at the moment a consideration of new EPA criteria, and the National Academy of Sciences has a panel that is going to look into this and there will be possibly specific criteria developed for Yucca Mountain. The second consideration is that there appears to be some thinking within the DOE of placing more and more attention on the container as opposed to the rest of the system, in the sense that some other nations, for instance, are moving into the direction that one could perhaps design a container to isolate the waste for 10,000 years. As a matter of fact the NWTRB in one of its reports said that the board saw no evidence that the US scientific community could not design a container of such a long life. My question to you is, what would be the consequences and any possible changes in policy if in fact the 10,000 years criteria in itself would be augmented with different safety doses, and also if it would be possible to design a container itself for 10,000 years thus making the geologic system maybe not as critical as up to now.

B. Boyle:

With respect to 40 CFR Part 191, since people are still working on that however it's changed, we just planned as if 40 CFR Part 191 was still in effect since we don't know what we're going to replace it with. Now, if people can come up with a 10,000-year waste package which I would have to defer to Lawrence Livermore Lab or other people, it would, I think, change what was done in rock mechanics. Certain of the issues may go away completely so I, having worked at the NRC once, don't know that the geologists at the NRC would all that be thrilled with the concept of a 10,000-year waste package. They might have to be convinced first, but if they can be, I think that the Department of Energy would take that into account.

M. Voegele:

Let me show you a not-for-publication viewgraph that some of us tried to put together to address pictorially what the Proposed Program Approach (PPA) was all about. First of all, to address your question, Dick (Bieniawski), we are not going to propose that the US's standards change and right now Part 60 does not allow you to rely solely on a robust waste canister. The natural barrier performance is an important part of that postclosure performance. However, and this is the part that I want to make sure I do not get misquoted on because it is real easy for me to mess this up. If you want to build an argument to convince a regulator that you can move forward within your program, there are some things that you can do that are easier to do than other things and there are some things to do that you can do that are harder than other things. If you start with the easiest stuff and I don't necessarily have these all in the right order, and you know some of you might say "God he thinks retrievability is easier than criticality," and I'll tell you why it might or might not be that way. It is not the exact order here that matters, it is somehow that there is a difference. If you can really knock down operational safety deterministically and present a real convincing argument that nobody has a problem with operations in a repository, that would be a first step. Then if you can set up a program which is focused on an early as possible demonstration that if you do put one of these MPCs into a drift, that you can retrieve it under some off normal situations. That would be another step in building credibility toward this argument. The criticality issue is relatively easy in the near term. What happens in the long term, for example, if you start to get water that you did not have in the system before the criticality, questions get a little bit harder. Thus, criticality, I think, is a little bit harder than some of these things. Regarding the robust canister, maybe by the time you are really getting ready to get your application docketed with the NRC, you think you can make a very convincing argument built on a lot of thermal testing and a lot of materials testing that you can make, may be not a 100% guarantee of a 10,000-year canister, but a pretty high probability that you can make a longer life canister. I am going to argue to you that that makes it easier to make your case for reasonable assurance on the performance of the gradual releases and the natural barriers, because you've instilled a lot more confidence in other elements of the program. Now if you go in and you say I don't know if I can retrieve, I don't know if I got a robust canister, I don't know about criticality, and I don't know about gradual releases, for instance, that is not a very convincing argument. If you can win all these arguments, I think that will allow a little bit more flexibility in terms of bounding what the answer is for gradual releases. I think that personally, and this is the one that you cannot quote the DOE on this because this is me and me alone speaking. I think it will be easier to make an argument for flexibility and bounding of the natural system performances to get that initial construction application docketed at which point you will then have several years to deal with the NRC staff and on the issues that they have. The stronger these foundation arguments are, the easier it's going to be to get you into the real licensing arena and begin really arguing the real issues which is what's natural barrier postclosure behavior. That is just my perspective on it and one of the things that we had in mind when we did this PPA, was trying to find the things that we could build the most confidence in, the most quickly, so that we could start to take some steps in this process. I will say this again. We are not currently on docketing a license application that is not geared to demonstrate a reasonable assurance in the long-term behavior of the natural system. On the other hand, getting that initial docketing and focusing the licensing interaction sharply is a goal that I think will help us move the program forward.

M. Nataraja:

The 10 CFR Part 60 regulation does not allow you to take total credit for one part of the subsystem, just as Mike Voegele said. I am just confirming what he said. Also, although this is not a place for giving official reaction to a strategy, personally I think what he just showed is an excellent licensing strategy.

S. Serata:

I am very impressed with Professor Ed Cording's observation in the G-tunnel. He was pointing out change in shapes of the boreholes and also the deterioration and fracturing of the rock around the existing openings. I have been able to observe the same phenomena. I would like to direct a question to Professor Cording. Stress measurements including initial stresses and stress changes around the opening give an understanding of the basic behavior of the ground which cannot be determined by any other means; especially, for the situation when the long-term behavior is involved. The long-term rock behavior cannot be determined by simply measuring material property or initial stresses except for continuously monitoring change of stresses. What is your thought regarding the needs for stress measurements.

E.J. Cording:

I think that certainly we can come up with elastic, elasto-plastic stresses using some sort of continuum analysis, but certainly the natural jointing around the opening is going to influence those effects. The more jointed the rock mass is, the less you are going to be able to develop new fractures. The deformation is going to be on pre-existing surfaces. I think there will be conditions like as we have seen in other tunnels, where you may have a pre-existing joint that is not through going but then there is just enough additional stress effects to cause failure from that fracture to the surface. In most of the observations in the work I have done, we focussed on observing ground movements and displacements. That has been kind of my focus in terms of monitoring and looking at stability. I think it would be interesting to see what some of the stress changes occurring around the opening are. I am certain that there will be situations where the stress levels will not be as high as one would predict from a more idealized rock mass with fewer joints in it. However, I think I would go back again to the observations as to what is really happening in terms of how the rock is breaking up around the opening or the support system, and I think that is the key part of it. What I did describe for the tuff at Ranier Mesa is not the same situation we will have here because the materials are much weaker. It is weaker even than much of the Calico Hills, at least it was non-welded tuffs we were working in. I think that we won't be seeing those same effects on excavation but there could be some effects which I think will be relatively minor but still could affect long-term stability in terms of thermal loading.

H. Gao:

My name is Hang Gao from Columbia University. I have a question for Professor Bieniawski. My question is about the use of safety factors in the design. I read a paper of your's last year, in which you said a safety factor is a factor of ignorance. That means that it ignores everything, perhaps not everything but at least something. As I understand, this is a very traditional way, because in the design we have many uncertainties. As a result, your final design could be underdesigned, and thus we use a safety factor to cover the uncertainties. I think this is a scientific way. Could you comment on the safety factor, for example, could we use it in a repository design? If not, can we come up with something like a probability method or other approach? Thank you.

Z.T. Bieniawski:

Good question. I have always referred and I will for the future always refer to the factor of safety as being the factor of ignorance. My students are very carefully trained in this respect because we need to ask ourselves a question, "What exactly is a factor of safety?" We do have a factor of safety basically for two reasons. The first reason is that because we do not understand, on one hand, certain aspects of behavior and most of all because we cannot with certainty determine properties or stresses in the ground; we got to have a certain factor of safety. Because we do not know certain things, the name ignorant is appropriate. Historically, it is interesting to consider that the factor of safety evolved actually in field of rock mechanics predominantly in mining considerations of pillar stability. Now what is happening in a typical pillar? You will see that you work on assumptions, for instance, such as what is known as tributary area loading where a series of assumptions are made. Those assumptions may not be valid. That is item number 1. Item number 2, there are always uncertainties even today where it comes to the determination of rock mass strength. The second major consideration is that the public prefers the image of a factor of safety because the public feels safer with a number like 1.5, 1.3, rather than the probability of failure. There has been a number of excellent articles on the topic in civil engineering, that indicate in fact the engineering community is not quite ready with the probability approach being offered to the public. This is because if you were to tell the public that for instance you have designed for a factor of safety of 1.3 in terms of pillar design and this still means a probability of failure of something of about 10 or 20 percent, since the factor safety of 1 typically means probability of failure 50 percent, the public will not feel comfortable. Kun-Soo Kim will be pleased that we too are now working very, very actively in the area of probability approach, because in rock mass strength it is a very, very important consideration. Namely, not so long ago Professor Hoek has come up with detailed calculations of the variability factors in rock mass strength where he showed that, using the probability approach, you can make very interesting comparisons with the factors of safety and you will find that the situation drastically changes in terms of factor of safety if for instance there is a very wide variability in material properties or *in situ* stresses. In essence what I would say is we are using the factor of safety in desperation. I still consider it to be a factor of ignorance. I do not think however that for repository design, where we are dealing with safety concepts, licensing concepts, and periods of times involved of 10,000 years, that the safety factor approach is the correct approach. I think this is the proper field for probability approach. Any comments? Thank you, gentleman at the back.

S. Blair:

I have a two-part question addressed to the panel. First to Mike Voegele, I guess I agree with you that hydrology and probably geochemistry are going to drive a lot of the PA. However, it seems like with the proposed multipurpose canister, rock mechanics is going to play an important role because we are going to have blocks of rocks over long periods of time falling on the canister. The question I have is how big are they going to be, how can they transport water there, what kind of point loads will we see, and that gets into the second issue of spalling and how long can we expect these drifts to stay open. It may also get into rock mechanics in the way of what kind of support structure do we install, and what is it made of, because the chemistry is fairly important in what kind of shotcrete we use as a support structure. Over long periods of time when the support starts breaking down, you have the rock collapsing in, and so we have to make the hydrologists and geochemists aware that we can probably help them out in answering some of these questions. The second part of my question, is that we need more long-term data to answer some of these questions. But, every year it seems like it is going to be one more year before we are able to initiate these tests. Then, they say one more year after that. Do you have any insight into when will it actually get there? I would like to pose those two parts to this panel and see what they think.

M. Voegele:

Not everything I say will end up in proceedings, right?

Z.T Bieniawski:

Right, you will have a chance to see your final remarks.

M. Voegele:

I don't mind you're recording this. It is just that when it shows up in proceedings that I mind things like that. I think the safest way to answer that question is that I do not control the budgets. I do support a construct of trying to find discrete ways to move this program forward. The people who are sitting in this room are probably most sensitive to the fact that NRC is a player in this DOE program. We cannot move forward without the NRC. However, we have other masters as well. The political system of this country is a master for DOE's program and we have many external critics, like the Nuclear Waste Technical Review Board, who have their own interest in the successes of the program. The approach that the DOE is trying to put in place right now is one that stops trying to eat the entire elephant in one sitting and starts to take a smaller pieces at a time. The three major steps of this proposed program approach really focus initially on determining suitability in the sense that the Secretary of Energy has to make a suitability finding to congress about the Yucca Mountain Site. Those suitability issues are relatively high level site specific suitability questions. The DOE is proposing currently to really focus the efforts of the program on those suitability issues. If the site is found to be suitable, we will as quickly as we possibly can put together an environmental impact statement and a license application to move forward into the negotiations with the NRC. It may look like for the next few years, that the DOE is paying more attention to its own regulation than it is to 10 CFR Part 60, and I think that is probably going to be what happens. I think DOE is going to be focusing on getting 960 behind it with the formal finding, the technical site suitability finding that DOE intends to make. So, the downside of all that is that if you are going to focus your resources very pointedly in certain directions, then other directions are going to suffer. There are just no two ways about that. I guess maybe I should be letting Bill Boyle stand up and say these things, but that is what is going on in the program today. The DOE is proposing to focus its efforts, rather than everything at once, a little bit more sequentially.

Z.T. Bieniawski:

Bill Boyle, would you like to elaborate on it?

B. Boyle:

I will address a few issues here that have come up. Dr. Serata and Professor Cording had an exchange on the importance of measuring *in situ* stresses and paying attention to displacements in the underground. I would like to point out that Sandia (National Laboratories) has an *in situ* design verification study plan, and they will be making both *in situ* stress measurements and deformation measurements and also paying attention to any failure modes that might exist as excavation proceeds. Back to the budget issue, Mike Voegele described it correctly. The Department of Energy is focusing on satisfying its own regulation first. There is a lot of money out there, and I would like to point out for Kun-Soo Kim that there is full funding for planning of the thermal-mechanical test for Sandia. I do not know if Lawrence Livermore's funding came through completely, but the plan is at this time next year that Sandia's test will be fully described and they will be well on their way towards design and getting the test implemented. Also, for Mysore Nataraja, there is full funding for Sandia for their *in situ* testing on seals. Thus, certain programs that were viewed as more important did get their funding completely intact. One last point I would like to bring up which goes back to this morning's session, if I understood Professor Bieniawski correctly, is that the right rock mechanics information for the most part is being gathered. Perhaps there is a glitch in getting it brought into design and I work on the site characterization part of DOE, so I will take that information back that we are gathering the right information, but I will also take back the information for both the repository designers and the ESF designers that you and your students as completely independent, unbiased group have problems with what they did. I also in the past have had problems with what is in some of the designs and my comment has been that when you come to it cold, it is easy to have a lot of questions. I find that the designers, none of whom are here today either from DOE or from the M&O, do a better job when they can explain themselves in person. However, that is not what is reviewed, it is their documents. But I will take back your comments and let the designers know.

Z.T. Bieniawski:

Please do, thank you very much.

A.H. Chowdhury:

This is Asadul Chowdhury from CNWRA. This morning the question of disturbed zone came into discussion and in addition to having the impact on the preclosure groundwater travel time, there are two ways that the disturbed zone may effect the postclosure performance. One is that it may change the fracture permeability due to the disturbance of the area and also it may have impact on the waste package due to the instability of the opening. My question is what program does DOE have currently to address these factors that may have impact on the postclosure performance?

S. Blair:

In answer to your question, we do on paper have plans for that. In practice, basically that work is in the planning stage and I think Stephen Brown and Ron Price (Sandia National Labs) have got some information on that as far as fracture deformations.

S. Brown:

The work we have done on coupled flow and deformation of rock joints was not funded under the nuclear waste program at all. It is funded under oil and gas programs and also basic energy sciences in DOE. As far as I know at Sandia for laboratory type of work, there has not been any work along those lines.

Z.T. Bieniawski:

Further comment from Dr. Mike Voegele.

M. Voegele:

First of all, we have tried very hard to reserve the term disturbed zone for that point that has to do with the beginning calculation for the pre-waste emplacement groundwater travel time because it is very different. There are disturbances, but we might talk about a modified permeability zone or something like that. You might see that is our discussions as opposed to referring generically to that whole thing as a disturbed zone. Most of the Livermore (Lawrence Livermore National Laboratories) field programs are focused on determining what the near-field effects are of waste package emplacement. I have not said this today, I probably have missed three or four times the opportunity to say this, but the MPC is something that the Department of Energy developed to meet their commitment to the utilities to receive waste in 1998. If you go back and look at the MPC design documents very carefully, you will find words to the effect that although it would be very nice if this could be licensed as a repository disposal system, we have no way of knowing at this time if we can license an MPC as a disposal system. The very obvious answer to that question comes from the thermal loading. If we cannot withstand those high thermal loads, then we possibly cannot use the MPC in the repository. There is always the option that the MPC's will have to be repackaged into a waste package. Then, you come up against the point that Livermore is facing today and that is what are the single most important things in disturbances associated with the performance of repository that tie it back into the thermal loading strategy and the waste package lifetime and so forth. I hope I have taken you to the point where you can see that we really have some systems issues facing us that have to be well integrated and in terms of the viewgraph I held up earlier showing these different stages of the PPA. I think it may manifest itself more in terms of conservative design and flexible designs that allow you to go within ranges. The goal of a license application is to get a document which really is a set of conditions within which you must operate your nuclear facility and there is absolutely no reason that we can't go in and try to make an argument that bounds the acceptability of certain ranges. It may be in the Department of Energy's best interest to say, look we fully intend during our repository operations to put some of this in a little bit hotter, some of this in a little bit colder, but we are going to stay within the ranges that you set in our license documents. It is not inconceivable that because the system's pieces may not all be fully flushed out, in fact, until you have some actual repository operating experience that does not preclude you from moving forward and coming up with a set that sufficiently bounds the range within which you need to work. We do not necessarily have to pin down the final thermal load at the time of construction authorization application. We do probably have to come up with some ranges that we can convince the staff and ultimately the Commission (NRC) that we can meet the performance objectives if we stay within these ranges. We then have some flexibility, and maybe after 50 years of operating in a cooler repository the DOE can find that in fact it would have been alright to operate in a warmer repository. So, you can move some of these canisters back a little closer together or, conversely, it may turn out that although you have acceptable performance with a warm repository, you may after years of monitoring discover that you have better performance with a cooler repository. Thus, the decision may be made to spread some of that material out. The only point I am trying to make here is I do not think we can pin down in a deterministic sense an answer to every question that we can pose, but we have to find a way to come to grips with it so that we can present a convincing argument so that the NRC can in fact grant a license.

S. Blair:

Just to refer to your question from another angle, I would like to see what the opinion of this group is on the fact that the hydrologists are basically using permeabilities on the order of Darcys or hundreds of Darcys in their models. Getting back to what you said before, does it matter then whether we dilate the fracture or not? If the hydrologists are using permeabilities of that magnitude, how do we deal with it? I do not have a good answer to that because it is hard for me to imagine that tuff is that permeable as a rock mass or maybe some of you think that it is or know that it is and can enlighten me.

M. Voegele:

If the DOE can present a convincing argument to the NRC that the permeability can be X and it does not cost you a fortune to build a repository assuming that the permeability is X, if the permeability happens to be .01X and that gives you even better performance, why worry about it? Solve the problem, bound the problem, and don't hesitate to be conservative if it does not introduce huge costs into the program.

H.N. Kalia:

I think the attempt is being to get the baseline information on permeability. The radial borehole tests in the ESF will be performed to get the information on what the permeability, porosity, and gas permeability ranges are. As you go forward with construction, the changes in those parameters are also being documented. It is a continuous test that runs for 3, 4, and 5 years. The baseline data will be available and if you want to define mechanical disturbed zone, one can do that. Data is going to be captured.

Z.T. Bieniawski:

We are going to move now to the gentleman there who had his hand up and we will try a different topic.

Z. Zheng:

My question goes back to a previously discussed topic, and it is addressed mostly toward Professor Cording. In terms of borehole breakout and stability of the tunnels, you may have reviewed this one proposal to NSF a few years ago I sent in utilizing the borehole breakout concept in oil and gas industry to get tunnels more stabilized and the reason I am saying that may not be applicable in here because of the lower stress conditions. However, with the consideration of future thermal stresses, maybe the tunnels will have spalling effect or a spalling phenomena as you addressed in the G-tunnel, and also South African mines and other places. However, this research we conducted at Terra Tek and also presented at the rock mechanics symposium in Austin is that we used the stress conditions and rock properties to try to predict what kind of spalling will take place around the tunnel and previous analysis show that the borehole breakout showed a very stabilized condition. We try to pre-make the tunnel into a stabilized condition say where rock wants to spall in that particular direction and what if we manufactured the tunnel in that shape to start with? That will come out with a stable tunnel shape. We conducted two phases of research on this already and it's proved that if we use a traditional design of tunnels and tunneling with elliptical shape or other shapes to counteract with these stresses, the strength compared to pre-carved breakout shape is let us say 30% to 40% lower than if I had a precast or precarved breakout shape in a tunnel. Also with regard to microscopic study around the tunnel, we made experiments on man made material and also rocks and microscopic studies around the tunnel opening after the test also reviewed that there are much, much less microfractures generated around the tunnel perimeter compared to other shapes if we carved the tunnel with a breakout shape. I am addressing to you and also to most of the other people involved in design, what is the feasibility of this kind of wild idea in stabilizing the entire tunnel?

E.J. Cording:

I think the stress conditions will be changing with time as one heats the repository. If you are getting some fairly good temperature changes, and I think most of the information I've seen is showing the horizontal stress is lower than vertical at the present time. However, the thermal effects should principally be adding more horizontal stress because that is the constrained direction in general and as well as gradients locally depending on how the waste packages are placed. So, that might be a complication in looking at what is the ideal shape. I have watched the construction of large chambers and I have tried to avoid just looking at the final shape of the cavern because the fracturing takes place as you increment the excavation. In a situation where you do have high stresses, fractures will occur in the bottom of the excavation as you come down. When you look at the wall of the excavation, you will see these fractures looking in on you, and so it has to do with the sequence. Certainly, however, in this project, you are going to be excavating up the full opening all at once. They are small openings or they should be for the most part. I think one of the benefits of a TBM is that at least you are getting a circular shape which I think is desirable. Straight walls and those sorts of features I think are some of the worst conditions, because of the deeper depth to which fracturing will take place if it's going to occur. Then, you look at the equipment and it is possible you can design an elliptical sort of excavation procedure, but I think that the present state of the equipment is such that circular is pretty much, the only really proven efficient tunneling method I think for TBMs in hard rock.

Z. Zheng:

I would like to continue a little on this. In our baseline research while I was a student at Berkeley, we found out the spalling effect is very severe especially in circular shaped tunnels and many cases we found out the breakout kind of pointed tip shape stabilized the entire cross section. I was thinking as you addressed that stress changes with time, and I was wondering if using the present data we could predict what is the worst condition in the future caused by thermal loading. For example, if the temperature increased to 200 °C and the thermally induced horizontal stress is much higher than the vertical stress, this maybe causing spalling on the roof and the floor; by treating this as the worst condition, one can precarve some kind of triangular shape around that region to stabilize and reduce the possibility of future fracture generation conditions.

Z.T. Bieniawski:

Dr. Zheng, may I ask, did you do this work at Berkeley with Professor Cook?

Z. Zheng:

Yes.

Z.T. Bieniawski:

Well, this brings to the very interesting point. Professor Cook spent some considerable time of his professional life in South Africa and the phenomena of spalling on boreholes in fact all started when considerations were given to the stability of excavations under the conditions of very, very high stress field. What was in fact found, is that the most stable configuration for deep level tunnels in gold mines, and we talk here at a depth of 10,000 feet, is a square shape. Why? This is because if you do have the greatest stress concentrations in the corners, it is actually the fracture propagation that is of greatest importance. In a square shape, under very, very high stress conditions in high elastic quartzitic rock masses, fractures actually originate, not at the corners, but inside the rock formation. In that stress configuration, the propagating fracture gets arrested. This led them to a complete redesign of all the drifts into a square configuration and it was further found that when the different shapes are used and spalling does take place, under no circumstances should you then clear the spalling. Rather, it should be left there and used as the reinforced lateral effect. Before anybody on the panel comments on this further, I am not too sure in my own mind whether the phenomenon will be applicable that we will have such very high stresses at Yucca Mountain.

E.J. Cording:

I think that for most of the loadings I see that the stress conditions are such that you are going to be in a range that is still well below the strength of the rock for the most part. There are other treatments for these zones like going in and blasting as hard as you can, for example, the Sudbury mines where they go in at the 6,000-7,000 ft level and perform a lot of prefracturing. They do not want smooth wall blasting because those slabs will come off or the failure would be after the excavation rather than before. My feeling is that we are not in a situation where we are forced into the real high stress environment type solutions.

M. Onofrei (Atomic Energy of Canada):

I do not think you should have to go as far as Africa. If you would come to Manitoba and the underground research laboratory (URL) you will see those phenomena which you are discussing. We excavated our 420-meter level now and you can see extremely high stresses in the rock. The boreholes change over a period of less than 1 year from perfect circles to elliptical.

K.S. Kim:

Our discussion is circling around the empirical understanding of the rock failure phenomenon. I think we should strive to be more quantitative. This is a nuclear waste repository facility we are talking about, and Professor Cording stated that the stress would exist within a certain range. We have a general understanding, but I think we would have to make an effort to define these ranges more quantitatively, and also to define the rock mass strength. We talked about a safety factor approach, or according to some people the safety ignorance approach, to psychologically sooth the public. However, I do not think we can really convince the public just using arbitrarily chosen safety factors of 4 or 8, or whatever, to make the public feel good. We now have the computational capability and funding to come up with more quantitative estimates.

F. Tsai:

I would like to give you a few number which I will present tomorrow also. For the thermal stress, if you agree in the numerical result of a model with one-drift and two fixed side boundaries as mirror images. Ignoring the tunnel, you have a one-dimensional problem, because expansion is allowed vertically while not in the lateral direction. Only two parameters are needed to calculate the thermal stress. The first is the Young's modulus and the other is thermal expansion coefficient. The mean value of Young's modulus of the TSw2 unit is 320 gigapascals and the mean value of the thermal expansion coefficient is about 1.0×10^{-5} /°C. This translates to an increase of 1 megapascal lateral stress for each 3° C. For 110 kw per acre thermal loading, you will have about 160 °C temperature increase in the near field in the area about 10 meters above and 10 meters below the repository horizon. This means you are going to increase 53 MPa in stress. The initial horizontal stress is 4 MPa. As a result, you have a lateral stress 57 MPa and the vertical stress remains at approximately 7 MPa. In the Reference Information Base (RIB), there are rock mass criteria established by Sandia, the equation is

$$\sigma_1 = 16 + 9.2\sigma_3 \tag{1}$$

where σ_3 remains equal to 7 MPa because of the free expansion vertically and σ_1 is about 60 MPa. The criteria added together is only about 48 MPa. Thus, the safety factor is under 1, if the thermal loading is 110 kW per acre. In other words, you really cannot load the repository above 100 kW per acre. This is my comment to what Professor Cording said, where he thinks the thermal stress will not create a rock mass problem. However, according to the current RIB it will.

Z.T. Bieniawski:

Earlier today, I showed the layout of underground excavations in the core test area and the gentleman from the audience stated that it has been changed and is no longer applicable. May I reciprocate that that criterion you referred to is no longer applicable. The reason for it is that it was built using empirical approaches involving rock mass classification indices which were tautly built on the basis of drill and blast excavation. We have reanalyzed the data and recalculated the effects that I showed early this morning. Based on the effects due to the tunnel boring system, as we have here now at Yucca Mountain, you can show that the rock mass quality in the tunnel boring situation changes by more than the whole rock mass class. This was something that was known very well in Europe for some time when two rock mass classes could be checked, as Dr. Cording will recall. If you do that, that very criterion would be very, very much changed, and you will find that the stress-strength calculations are very close.

E.J. Cording:

I would like to respond on that also, because you were referring to what I had said. Was the strength criterion you were referring to a rock mass strength?

F. Tsai:

Yes, rock mass strength.

E.J. Cording:

I ran almost exactly the same calculation you did but I used real strengths in terms of the way I have seen actual performance and excavations. The numbers I come up with would show that you would be getting some significant stress changes, and I am not going to argue that, particularly if you go to the high thermal loading situation. I think there will be some slabbing. I ran the same numbers at a 100 degree change using exactly the same approach you had described, and I come up with the tangential stress being about 40% due to the thermal effect, 40% of the compressive strength of the rock. It will depend of course on what real stiffness you have in the rock mass and those sorts of things.

F. Tsai:

40% of the intact rock strength?

E.J. Cording:

Right, and what I usually consider is the strength could be lower than that by a factor of 2 or so, so you are getting to the point where you can expect some mild effects at that point. But, I think one has to be very careful using rock mass strength parameters because they are very difficult to evaluate.

B. Boyle:

I will bring up the budget again and the RIB values. Sandia had a task this coming FY to update those RIB values for rock mass strength. Realizing that they are numbers that are hard to quantify, Sandia got more money than they initially asked for in order to provide a rationale for why the numbers they come up with are the appropriate numbers.

Z.T. Bieniawski:

Now, is this the work that Sandia is going to do or they have already done? Are you referring to the report by Lin and others?

B. Boyle:

This is work that will be done starting Oct. 1 through Sept. 30 of 1995.

S. Serata:

I never expected to get into this discussion, but as you started with the geometry and long-term stability of the opening, I should make a summary of my experience. There is a very interesting possibility for you to look into the method of so-called stress control. By bringing the parallel rooms together at a certain close interval, you are able to throw the tectonic stresses way up into the roof and surrounding ground. By doing so, you are able to establish the long-term stability. In one of the pictures I quickly went through, I am not sure you could believe what I said. The roof caved in if the room is only 20 ft, but the roof is now totally stable when I enlarge the room. This is because a tension develops and is compensated with tectonic stress, and as a result, the tectonic stress is taken by the large artificially induced protective stress syndrome. The purpose of my emphasizing the stress measurement is because of the economic advantages actually achieved by Canadian Potash Mining and also the salt mining as well as coal mining today. Concluding this, there is a definite possibility of stabilizing the ground, and also absorbing the thermal stress by introducing this stress concept. Since you people have such a tremendous computer capability, I am sure you will be able to calibrate your program with field conditions where we already have 20 or 30 years of field records. I feel that you have a very interesting possibility. Thank You.

Z.T. Bieniawski:

On this very happy and enthusiastic note, I would like to thank you all for being so good as to stay well beyond the appointed hour of adjournment. Our chairman indicates that now may be a good time to adjourn, so I would like to take this opportunity to thank panel members for their participation and for preparation of their remarks and all of the audience for participation in discussion. Thank you all!

Panel Discussion on Near-Field Coupled Processes with Emphasis on Performance Assessment

Panel Moderator:	R.B. Codell, Division of Waste Management, Nuclear Regulatory Commission (NRC)
Panel Members:	R.G. Baca, Center for Nuclear Waste Regulatory Analyses (CNWRA) M.P. Ahola, CNWRA L. Jing, Dept Civil & Environmental Engineering, Royal Institute of Technology, Stockholm, Sweden S. Blair, Lawrence Livermore National Laboratories L.S. Costin, Sandia National Laboratories E.J. Cording, Nuclear Waste Technical Review Board (NWTRB) J. Philip, Office of Nuclear Regulatory Research, NRC

R.B. Codell:

I would like to start: there are several people who wanted to make brief presentations before we start our panel discussion on the general topic of near-field coupled processes and postclosure performance assessment with an emphasis on rock mechanics. Mikko (M.P. Ahola) is closest, so why doesn't Mikko start.

M.P. Ahola:

I have a couple of figures to show which relate to the first question that Dick (R.B. Codell) had posed in his opening remarks regarding the potential impact of near-field rock deformation on repository performance. From the rock mechanics area, we primarily focus on the near field disturbed zone, the extent of which I do not think is fully known. This disturbed zone will likely change with time. There are a number of issues during both the preclosure and postclosure which could have an effect on some of the repository performance objectives (Viewgraph 1). I'll just focus mainly on the DOE's most current scheme of in-drift emplacement, and look at both a case of unbackfilled as well as backfilled drifts. I do not think it has been finalized as far as when backfill operations will commence, if backfilling of the emplacement will take place at all. Essentially the canisters will be placed along the length of the drift, and it may be beneficial to keep the drift unbackfilled for retrieval purposes during the preclosure period. This retrieval period could extend 50 to 100 years after waste emplacement, and over that period of time, there could be conditions where you could get some rock failure or spalling as you've heard discussed yesterday depending on the particular *in situ* and thermal stress field environment. I think from a stress state standpoint, the repository is located in a fairly ideal type of environment, however, the impact of thermal and other loads should be considered. For example, you have heard Simon Hsiung's presentation on the repetitive seismic effects on drift stability. The effects of several hundred years of heating on the unbackfilled drift may result in yielding around the tunnel, which

POTENTIAL IMPACT OF NEAR-FIELD ROCK MASS DEFORMATION ON REPOSITORY PERFORMANCE:

- Retrievability
- Integrity of waste canister
  - Damage due to rock deformation/rockfalls
  - Increased permeability and subsequent flow into the drift
  - faster radionuclide transport
- Near-field performance assessment (PA) and total system PA calculations

Viewgraph 1

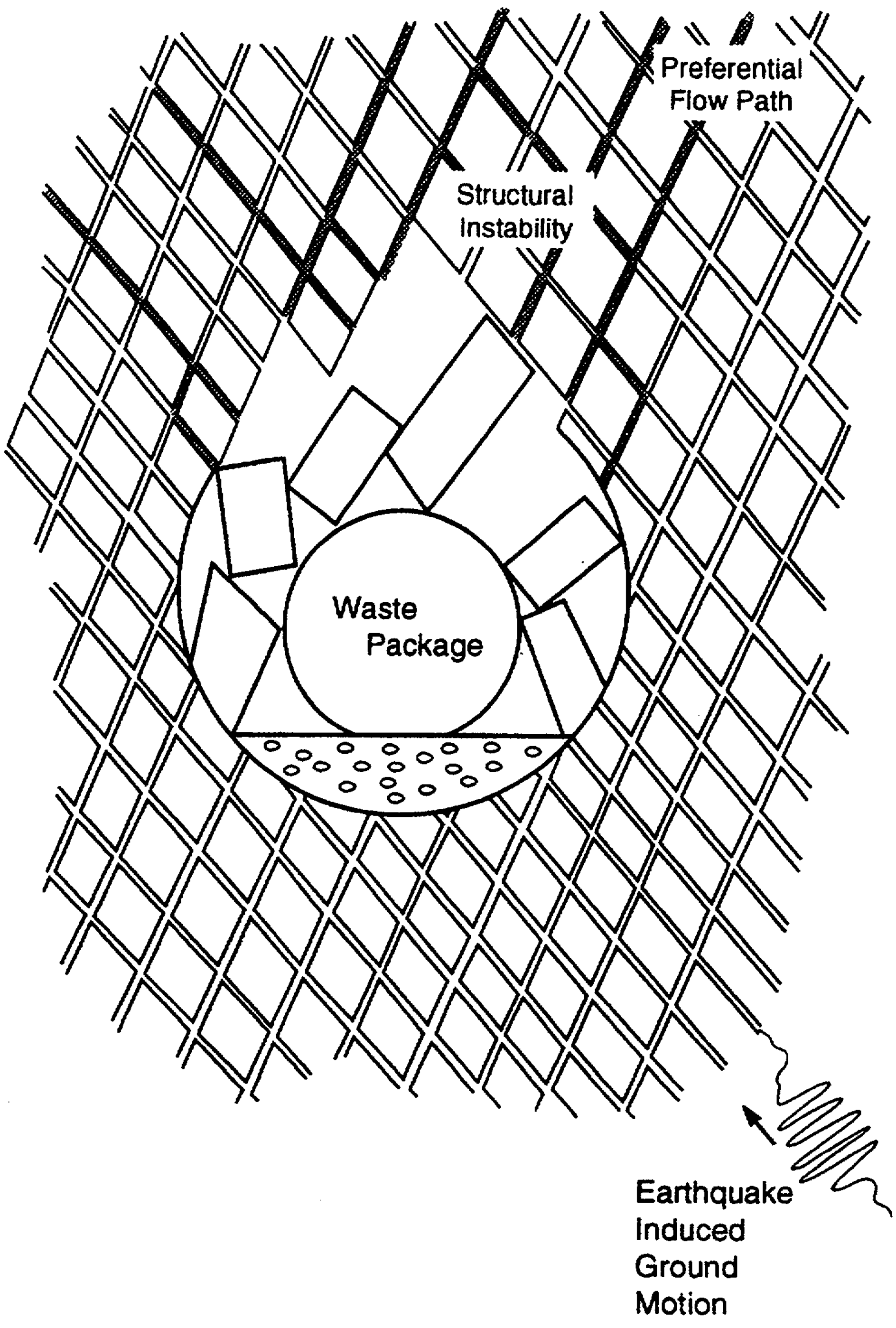
may develop into occasional cave-ins. Under thermal loading, I do not think it is fully understood how the material properties and rock strength may change with temperature. There have been some reports that suggest that some of the mechanical properties, such as Young's modulus, are somewhat reduced, maybe 10, 20, or 30 percent in response to rising temperature. In addition, at certain levels of saturation the properties could be affected even more. As a result, thermal loading may have the effect of increasing damage due to repetitive earthquake induced motions. A small seismic input could have a stronger effect under the adverse environment of thermal loading, as well as maybe the drying and continual redistribution of moisture (Viewgraph 2). The adverse condition of rocks impacting the canister may occur that could affect the retrievability performance objective. It may also affect some of the waste canister lifetime performance objectives (300 to 1000 year lifetime). The corrosion models have some kind of a function that depends on the amount of moisture that gets to the canister. In performance assessment calculations, they essentially decrease the thickness of the canister wall to account for corrosion; so, at a certain point, maybe initially, if something fell on the canister it would not be of much significance. However, over time, as the thickness of the canister wall is reduced by corrosion, such point loads due to rock-falls could also affect the lifetime of the canister. I am told that this corrosion could occur fairly rapidly under certain environments, i.e., moist environments.

The other impact on the canister could be the possible increase in permeability and flow into the drift; Dick (Codell) showed some slides indicating the possibility for a condensation halo above the emplacement openings to occur due to the evaporation of moisture and redistribution in condensation at some level above the emplacement horizon. There have been some reports that showing significant moisture within the repository rock itself to cause that type of phenomena where you could have some condensation above. If you had had some caving of the rock within this disturbed zone you could have a significant increase in permeability, all the way up to that condensate zone. This may cause some reflux of fluid back down to the waste canister. Even if at the initial state, the fractures are fairly tight such that the water never really gets all the way back down before it vaporizes, under this dilation or fracture opening, you may create the condition where water is able to flow back down and impact the canister. This would also affect the lifetime of the canister or postclosure lifetime of the canister due to increase in corrosion. The release from the canister could also affect radionuclide transport both through the gas and liquid phase due to this disturbed zone. Mainly, you could have a higher conductivity or permeability within the disturbed zone.

Finally, the near-field rock deformation could impact some of the performance assessment objectives for both the near-field as well as the total system performance. Some of the present flow codes assume a continuum, however, they do not have a good feel for how the permeability changes within this near-field zone. Considering this increase in fracturing and deformation in the near-filed, the present system codes could be underestimating or overestimating the fluid flow or the permeability within this region, which could effect the performance assessment calculations. One of the objectives in our work is to try and be able to give them some sort of permeability within that zone either using discontinuum or other types of codes.

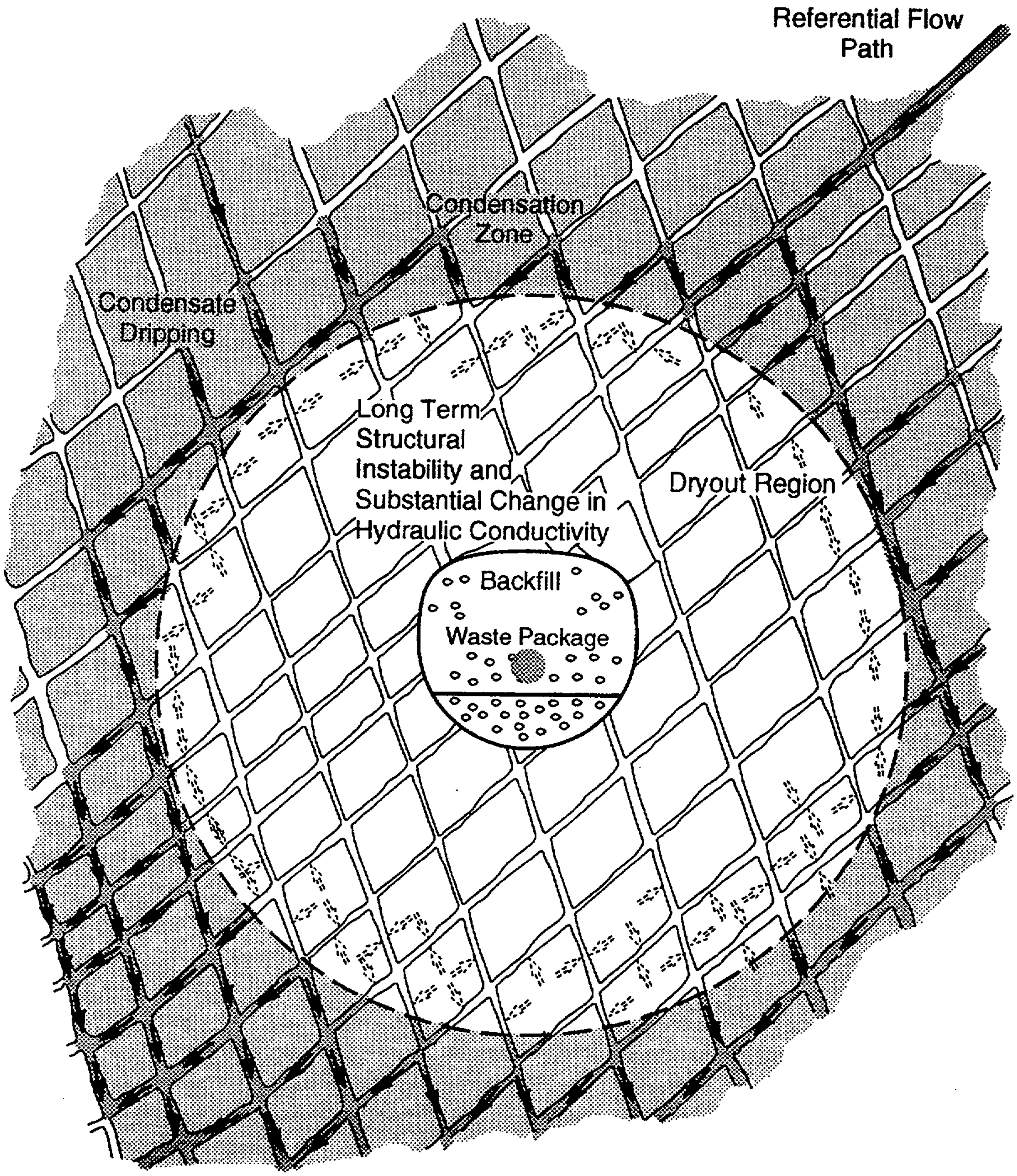
Viewgraph 3 shows an in-drift emplacement scheme with backfill. The backfill may help to support the opening much better, and may not have such a large mechanical effect on the waste

GEOMETRY & FAILURE MODES OF IMPORTANCE: IN-DRIFT EMPLACEMENT



Viewgraph 2

GEOMETRY & LONG TERM EXCAVATION DETERIORATION: EFFECT ON NEAR-FIELD HYDROLOGICAL PROPERTY



Viewgraph 3

package compared to a drift collapse. The backfill may also reduce the amount of deformation within the disturbed zone resulting in less impact on the near-field permeability. Those are the figures I wanted to show. Are there any questions I can answer?

R.N. Datta:

So, the possibility of increase in permeability will be reduced by backfill.

Mikko Ahola:

Yes, that could be. The backfilled case would probably be a little less severe than the first case. It would depend on the properties of that fill material, and how much it will actually act as a support on the tunnel walls. You would like to get perfect compaction in a horizontal drift, especially since backfilling will commence after the canisters were in place. As a result of this ineffective compaction, you would have some settlement in the top of the drift I would guess.

R.B. Codell:

I would like to hold further discussion until everyone's had a chance to talk. We'll do it that way. Thanks, Mikko. Who would like to go next? I think Jake (J. Philip) had some slides.

J. Philip:

Actually most of the stuff that I was going to talk about has been talked about by Mikko (Ahola) and partly by Dick (Codell), but I think that we've got to concentrate on an unsaturated fractured tuff and see what types of coupled effects are relevant to our particular situation at Yucca Mountain. Dick (Codell) talked only about postclosure but I think that we will have to talk both about preclosure and postclosure because as Mikko (Ahola) pointed out that we have the retrieval option there. So both the preclosure and the postclosure have to be looked into.

The types of coupling that are important as Dick (Codell) mentioned have to be evaluated. It is likely, for example, that the thermal-mechanical coupling may be important while the mechanical-thermal coupling may not be. And the question comes again to the disturbed zone now the near-field, we talked about the near-field processes, now how far is the near field extent of the disturbed zone where these coupled processes are effective? There has been some, a little bit of data, that show that it may extend quite further than what we have normally talked about in the near-field, so the extent of the disturbed zone needs to be looked into. Of course, right now we have just talked only about the modeling aspects and what we have conceptualized, but then how much do we have to look at THMC (Thermal-Mechanical-Hydrologic-Chemical) effects from actual data? Modeling is fine, but maybe we would need to look at some benchmark tests as we have done at DECOVALEX, and some actual tests that we have done at DECOVALEX; what should be the extent of that, what types of tests should be run, and specifically for Yucca Mountain, looking at the unsaturated case. The scale of the tests are important. You could have a lab scale or a field scale which again comes to the next point, the designer's experiments. Maybe if he could reach some agreement on what the coupled processes are and if you, after some understanding of it, could design some experiments to specifically look at some aspect of coupled processes, like we have done a little bit in DECOVALEX, where we made some modifications to Nick Barton's experiment at NGI, based on some of the modeling and some of the results that we looked at by different groups. So we made some design changes in the experiments to look specifically and focus on certain measurements. And of course, it all goes back to where we can use that in the performance assessment of the whole system; whether this is going to be just a subsystem performance assessment which would be an input to the whole system or part of the total system analysis. That's all I have.

R.B. Codell:

Thank you Jake (Philip). I would like to call Bob Baca up to the microphone next.

R.G. Baca:

When Dick Codell was giving his talk about coupled processes, and as I was taking account of all of the things he was enumerating, I was wondering as a total system PA (performance assessment) person how we could ever try to incorporate even a significant portion of those into a TSPA (total system performance assessment) type of code. And it seems to me like we have a little bit of a paradox in that we know there are so many different phenomena that are going to be important to total system performance, yet DOE's Proposed Program Approach is currently aimed at just giving us selective data. So I'm wondering, with limited data and limited knowledge about a lot of those couplings, how in the world are we going to decide which are the important ones and which ones do we need to put into our total system PA code? So I'll leave that question for people to think about, maybe we can dialogue about it a little bit. What I'd like to do is to tell you what I think we can do about the coupling between rock mechanics or thermal mechanical type of phenomena and postclosure PA. And what I'm going to do is I'm going to draw upon the work that I was involved in at the BWIP project, I'll show you how we used to go about doing total system PA. We broke the problem into two major parts. We said we have an EBS, engineered barrier system, and we have the geological system, natural barrier, and we broke this up into two separate parts. We said: Okay, now we have a waste package and something we call the seals. In the natural barrier we broke that down into two parts which we called the emplacement horizon and then the far field, the longer range geologic setting. So we have the engineered system which hopefully provides defense-in-depth to the performance we get from the natural system: draw a box around that and we call that the total system. Now, like most engineers, we look at this problem and we say: Okay, how can we break this down into a simpler thing to analyze. And of course the first thing we did was to say, well, let's look at the seals.

M. Nataraja:

You mean backfill as opposed to seals?

R.G. Baca:

I am going to show you a little conceptual drawing of what seals meant, seals in the context of the BWIP repository. And so, we figured that we could solve this problem, get it out of the way, and then reduce our total system PA to simply looking at a waste package and then the two parts of the natural barrier. Now seals, in the context of BWIP, really referred to, forgive my art here, this is the underground workings, this is the top of the emplacement horizon, the bottom of the emplacement horizon, this was called a Cohassett formation and it was about 70 meters thick. The design was, we would have a number of shafts centrally located, and what we referred to as the seals or seals pathway really had to do with the barriers that we put into the shafts after closure. So if you took a blowup of that, what seals meant to us was: here is a shaft, they would put a concrete plug and then that would alternate with the basalt that was removed at the time of the construction of the shaft. So you had this alternating sequence of concrete plug and then this backfill material. Now the rock mechanics people would tell us that there was an affected zone, a mechanically affected zone, where fractures would generate, would be created as a result of construction. And so we analyzed this system as our seals pathway, the idea being that in the basalt setting we had a hydraulic gradient, a natural hydraulic gradient, that was upward and then there was another gradient that was induced by the thermal effects, so we had sufficient driving forces to move material that might migrate from the waste packages towards the shafts and then move up these pathways.

R.N. Datta:

In this case, seals are located in the emplacement horizon, which are part of the geologic settings; how can you make the seals part of the EBS? In your previous slide you had the seals as part of the EBS.

R.G. Baca:

That's correct. Well this is an engineered barrier, okay, and the original design for BWIP also included barriers inside the drifts. So all of the, what we called the EBS, was any part of the man-made barrier. So of the significant part of the EBS, the part that we called the seals was the barriers within the shafts. So any material that could move away from the waste packages could move along fractures in the roofs of the drifts that eventually might find its way up towards the shafts became a pathway. And so you have high permeability zones, you have a negative hydraulic gradient, material follows the path of least resistance, the idea was that it would somehow either by diffusion or by hydraulic gradients, move up these shafts.

R.N. Datta:

The NRC definition of EBS ends at the surface of the openings and DOE is kind of accepting that.

R.G. Baca:

Well this dates me, this is work that was done back in the early 80's, but I think that the concepts that we're using here are still applicable to the Yucca Mountain site because this is a potential pathway for material to move out of the system. I think that in the case of the Yucca Mountain site, I'm more concerned about: let's call this our emplacement horizon and here's our repository. In this case, as I recall, you have two very large shafts at the ends and this is for air circulation. And so my concern is that we have a lot of perched water in this mountain and it might flow through fractures along these shafts after you've sealed them and, therefore, there might be a recharge condition that might bring water down into the repository horizon. We know that the ends are not as hot as the center so maybe the dryout zone isn't as effective even if you go to a high heat load type of strategy. Another aspect is that the chemistry of that pore fluid could be dramatically different than the natural pore fluid and it might affect the corrosion rate so possibly these areas might be more susceptible to more rapid failures, if this kind of scenario is taking place. So I guess where I'm leading to is that where rock mechanics could make a contribution is to help define the characteristics of that mechanically disturbed zone in the shafts, any possible creation of vast pathways or high permeability zones, and that information should be used in a performance assessment of this particular component of the repository to find out if it's significant. If it's not, then we're done with looking at that possibility. So, this is in my mind, one of the first areas where we should examine the importance of this pathway working in conjunction with the rock mechanics people. Any questions?

R.B. Codell:

Thanks Bob. Who would like to go next? For the record, this is Larry Costin in front of the microphone.

L.S. Costin:

I took a little bit different tact in preparing something to discuss here. I thought I had probably 5 or 10 minutes to bring up a few points and what I wanted to do was to try perhaps stimulate some discussion for the discussion period by providing a few comments on this morning's presentations and maybe a few thoughts of my own in regard to that. The first comment I had, and I would hope that Dick Bieniawski was still here, but I guess he's not, because this involves something that is sort of near and dear to his heart. That is, a couple of years ago we had a workshop on design methodologies. And in that we had some fairly extensive discussion about the technology transfer between the rock mechanics community and the design community and it was sort of a universal conclusion from that workshop that there essentially was no technology transfer between the rock mechanics community and the design community, that the designers continued to design things the way they had always designed things using the same empirical rules and things that always seemed to work, but they really hadn't taken into account the knowledge that's been gained over the past maybe 20 or 30 years of looking into the details of rock physics. And I think that the conclusion of that conference anyway, or workshop, was that somehow the rock mechanics community had to do a much better job of somehow making that physics more palatable or packaging it in a way that it could be used in the design process and that was part of the idea of developing these design methodologies that we've allowed people to incorporate that kind of idea. So I really wanted to make the comment because in at least one of the first talks this morning, there was a lot of discussion about we have to sort of package the testing so that the designers can use the test results, and I think in some sense that's true but in another sense there, what we need to do is design the testing to be able to understand the physics so that can be translated to into a proper design. And I think that you'll see that methodology at work perhaps in the WIPP project a little more than in this current project. We've had some involvement in the WIPP project for quite a while and really one of the outcomes of dealing with the complications of salt mechanics and salt deformation is that we discovered, that lo and behold, it is very difficult to use empirical rules to do designs and in those kinds of materials if indeed you want the behavior of the openings and the performance of the repository to meet the kinds of regulations now that the EPA and NRC are promulgating. And so you really do have to understand the physics, you have to be able to produce good constitutive relations, you have to be able to compute results for long periods of time, and understand what the time dependent nature of the material is. The time dependent nature of the material may be less important in terms of the Yucca Mountain Site but we can't really demonstrate that at this point. That was one point I wanted to make.

I think the other point was if we look at preclosure vs postclosure and near field vs far field, I start to get a little confused in that to me at least in my notion this would be my model sort of the near field and I think by not taking into account a large enough rock mass and the details of what's going on on the mountain scale in terms of your near-field calculations, you're missing a lot. For example, the fact that you have potentially a heated mass within, inside a larger cooler confining rock mass, creates a lot of problems that are not well understood if you only focus say on a drift scale near field. One of the things I wanted to, just as an example of that, is this is a calculation of heated repository and what you're seeing displayed in the colors is actually at the vertical displacement in one of the layers, the Tiva Canyon layer which is at the top most layer and you can see that there is quite a bit of heave (vertical motion) over the site caused simply because you're constraining the repository: I mean the heated area is not free to expand in all directions equally. And if you look at this, the mechanical ...

M. Nataraja:

Under what scenario is that, what temperature?

L.S. Costin:

The time scale is 300 years, and this is an emplacement that involves I think it's 100 kW per acre, something like that.

R.N. Datta:

Does that correspond to high thermal loading?

L.S. Costin:

High thermal loading. You can see it's somewhere near about a third of a meter uplift on the surface due to that. If you translate that into what the mechanical effects or the effects on the rock layers below that of this heaving, you can certainly imagine that you would begin to fracture layers above the repository allowing significantly more infiltration than previously perhaps calculated using just the normal fracture densities.

The third point I guess I wanted to make was a little bit about the modeling approaches that were discussed this morning. And I think there are a number of approaches that need to be looked at. One of them was discrete element modeling that treats the rock mass as discrete blocks. These kinds of models work very well in terms of modeling the geometry of the situation and perhaps even the interfaces to some degree; but the problem, as discussed this morning, is the amount of computation necessary. When you begin to put that level of detail in the geometry, the computation begins to eat you alive. So, many people are looking at more of a continuum approach in which you can use sort of simplified geometry but retain the same elements in terms of how the joints behave; this is an example of how you might sort of simplify things. If you have a rock mass with the joints around an opening with sort of regular sets of joints, you can then put this kind of condition into a continuum framework, but still account for the basic nature of the jointed rock mass. One of the approaches that we've used (at Sandia) is again to account for the non-linear joint closure and also for the shear behavior of the joints; both of which can be incorporated into a joint model that calculates the stiffness of the rock mass under different loading paths. The nice thing about this approach is that you actually do specifically incorporate the fracture aperture or you keep track of the fracture aperture during the computation. So if you look at a calculation of a domain containing a set of emplacement rooms and focus on various points around those rooms, you can find, if a thermal load was applied, typical temperatures at various points horizontally. If you also want to look at, say, what are the fractures doing at those points, you can calculate or keep track of the amount of slip on the joints and you may find that some of the vertical joints have one sense as you go through the pillars they change sense of slip. But what I really wanted to indicate here is that we can also keep track on the changes in fracture apertures. And what you may find is that the change in aperture may be quite dramatic mainly because it takes very small changes in stresses to close those fractures to the maximum amount. And certainly, as I think as Frank Tsai's presentation this morning showed that it doesn't take that much temperature change locally to generate those kinds of stresses.

The other point I wanted to make relates to, I guess, Frank Tsai's presentation that looked at change in hydraulic conductivity of a joint due to mechanical loadings. We've done some tests, probably not related to the waste project but related to enhanced oil recovery, looking at what the effect of changes in permeability of joints is. Steve Brown is one of the people who have been involved in this project and could probably explain it a lot better than I can. I just wanted to show that in fact that we'd done some tests similar to the kinds of tests that were talked about this morning. The results of those tests are quite interesting and in fact quite comparable to what was talked about this morning; they basically show, for a mated joint, the flow rate decreases as increases in normal stress. And as the normal stress is relaxed the flow rate doesn't recover quite back to where it was before. However, the change is not all that great; it may be something like a factor of two. But, if you include some shear offset in that, then the changes become more dramatic and can be something like an order of magnitude. The real interesting thing about this is that, consistent with reported elsewhere, the flow rate or permeability changes as joints begin to shear and you can get large changes in permeability; but those changes can be either positive or negative, depending on the nature of that particular joint and that particular rock. If you have a very strong rock that doesn't create much gouge, you get large aperture changes because of the slippage, then you can get enhanced permeability. If you create a lot of gouge, you can get decrease permeability and it can be quite significant. Those are the points that I wanted to make sort of relative to some of the talks that were given this morning and hopefully will stimulate a little bit of discussion.

R.B. Codell:

Thanks, Larry (Costin). I didn't know whether Steve Blair wanted to say something, and also Lanrun Jing. Steve, would you like to?

S. Blair:

OK, thanks. I just would like to make a couple of points about the coupled processes concerning rock mechanics in performance assessment. First of all, it seems to me that the point's were made a few times at least from the perspective that we have at Livermore, that hydrology and geochemistry are basically the drivers in what we need to in the next few years, at least to get to where the program is trying to go in this PPA. So, we view rock mechanics as basically needed for interpretation. So that get's into the thing with fractures, fracture apertures, and other properties. It seems to me, at least for performance assessment, that's where we can make the biggest contribution, is helping the hydrologists and geochemists have some faith in what they've got as far as, are these predictions that they're making correct? The other point I want to make, I think that the near field basically does need to extend farther out, it's dependent on processes and you can say from the standpoint of rock mechanics, that maybe the near field extends out two or three shaft diameters or two or three drift diameters. That may not be true if you precipitate a lot of material in fractures at some distance away due to this refluxing or other processes. You may change the properties of those fractures and change the mechanical behavior out there. So, because the processes are coupled, I'm not necessarily sure that we can take such a narrow approach to what the definition of the near field is for rock mechanics.

I would like to give just a couple of more detailed examples of things that could be input, or I think should be at least considered for input into performance assessment. One is the cristobolite phase transformation, which affects a lot of different things. The thermal expansion of the tuff is nonlinear with temperature and it's thought to be due to cristobolite and other minerals. Now, if you want to consider a bounding calculation which we've talked about as far as getting to proposed Program Approach, you take those numbers, those raw numbers that come out of the laboratory measurements of thermal expansion, and put those into an elastic calculation (which would be the bounding one), and I think the stresses are going to go out of sight around the drifts. So what we need to be able to do is to decide what is a bounding calculation. We need to gather the data so that we can say, here's a bounding calculation for the rock mass, computed using properties that we have confidence in, rather than using a bounding calculation that is a purely elastic calculation. The most conservative number may not be the one we want to use. That's the point I want to make. And so I think that we need to resolve that with this cristobolite problem. Another problem is that if this mineral does actually expand several volume percent, then what actually happens in the rock. Do we create microfractures in the rock? Does it just move into the open porosity? What goes on? This relates back to the problem of drying and rewetting behavior, the actual reactivity of the rock as far as chemical reactions, but the cristobolite problem is a rock mechanics problem. So rock mechanics can contribute directly to resolution of these questions, so we can provide information to the hydrologists and geochemists there. As you know, the drying and the rewetting behavior is fairly important as far as the movement of the water in and out of the fractures; and if we change the rock matrix with temperature, then we need to know what is going on there. So that's some information that, I think, we can provide that the PA community is going to need. I won't go into great detail, Larry's done a good job of talking about the fractures and they've been talked about earlier in the meeting, but as far as what we can provide regarding heating and cooling cycles, associated drying and rewetting, and changes in permeability and shear strength; all of these kinds of things, and also this dissolution and/or precipitation at some distance away from the actual waste itself. I think that these are things that we can contribute to in performance assessment.

Finally, I'll say a little about, seeing as this is a coupled processes session, a little bit about the large block test. What we're trying to do on the large block test, what we propose to do, is look at coupled processes there. So, this is, I think, one of the first experiments where we were actually doing the thermal, mechanical, hydrologic, and geochemistry. Now, I don't know how many of you are very familiar with that test but the idea of that test is to actually test out this idea of will we develop a refluxing zone: What is the actual behavior hydrologically? Can we set up the hydrologic regime that the models predict? Can we actually do it? Can we measure it? Then, what happens as far as the geochemistry and the geomechanics of it? The way that the geomechanics contributes there is, can I help the hydrologist figure out actually what's going on in case it doesn't behave like they anticipate, or if it does, I can say, this is what's going on. If we get this precipitation of minerals that close up the fractures, what goes on in the boiling zone, do we have to worry about it, what goes on in it's refluxing zone, this thing that came up earlier, does the water actually follow these fast paths back down or does it all evaporate sooner? These are the kinds of questions that we hope to get at in large block testing. And I didn't bring along a whole lot of information on that. The main purpose of that test is to test the hydrologic hypothesis, and geochemistry and geomechanics kind of come along to verify these coupled phenomena. But basically the hydrology is the driver. Another thing about that test that comes in any time we start doing this kind of stuff that we were talking about coupled processes, it's a very complex test and I don't know how many were at the meeting in Las Vegas where Hemi (H.N. Kalia) talked and Kun-Soo Kim, and then Wunan Lin got up and talked about the large block testing, how much more complicated it was than these other much simpler tests that were done just for thermal mechanical goals. On the other hand, the only way that we're going to be able to address some of these problems is that we try something that gives us a little more information on what are the coupled processes, what ones are important? Thank you.

R.B. Codell:

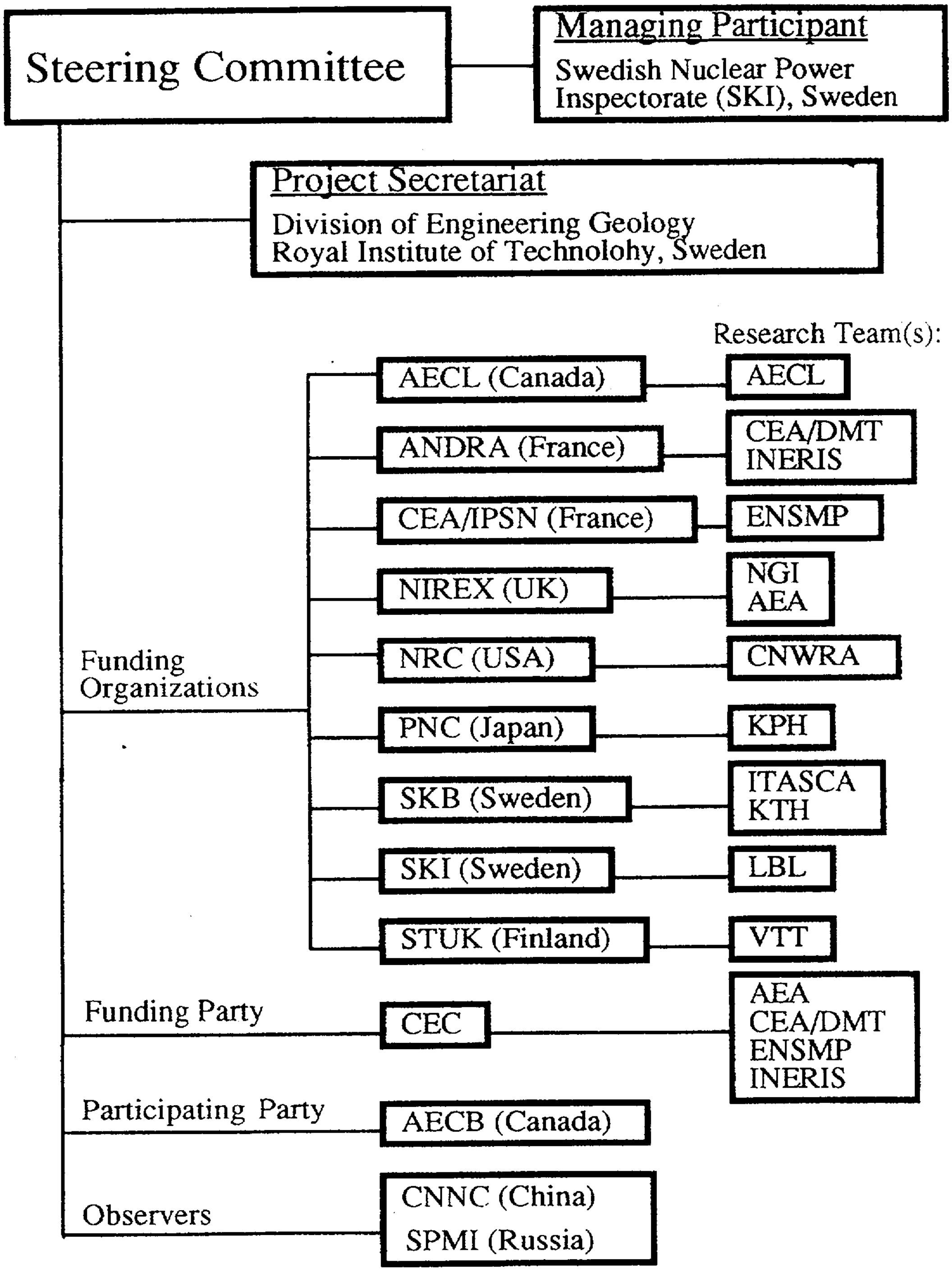
Thank you, Steve (Blair). We need to take a moment because the Center bought cheap tapes. Lanru Jing will now speak for a few minutes.

L. Jing:

My talk perhaps is a little off the subject for this workshop, but anyway it's related to the coupled processes. Talking about DECOVALEX project, because Mikko (Ahola) has covered part of it, I will just give you a really brief introduction about this project. I will just talk a little bit about the approach, definition, and current status. If time allows, maybe give you one example and a little about the future. For organization, there is only slight change. we have 9 organizations from seven countries (Viewgraph 4). CEC (Commission of the European Communities) is the founding party. Because Sweden and Finland are not CEC countries, so they cannot support the non-European community research teams. So they act as the founding party, but do not have a voting right in the steering community meeting. And then we have AECB (Atomic Energy Control Board) from Canada as a participating party. They just work on their own study or whatever problems are defined within the DECOVALEX project, and participate in the workshops; but they are not in the steering committee meeting. Then we have Russia and China as the two observers.

DECOVALEX works with the benchmark tests and test case problems. So far we have covered three benchmark tests and six test cases (Viewgraph 5). The six test cases are small-scale test cases, mostly small-scale laboratory tests. Test case six is *in situ* borehole injection test. The borehole is located in Lulea University about 300 meters below ground surface. TC2 (Test Case 2) is an *in situ* block test, 10 m by 5 m block test. For the next phase of DECOVALEX, the intention is that we will not study only small-scale laboratory tests; but concentrate on one or two large scale coupled TMH processes in underground laboratories. Viewgraph 6 lists the various codes used in the projects. Mikko (Ahola) has told you about that. We have almost every kind of method: we have finite element, finite difference, distinct elements, and a discrete fracture network. These are problems simulated by different codes. This is BMT1, benchmark test 1 (Viewgraph 7). This intends to simulate a large scale repository with 3,000 m by 1,000 m model size with a variable hydraulic head and a constant temperature at the top. A constant geothermal flux is applied at the bottom of the model and a repository is situated at a depth of 500 meters. The next one is BMT2 (Viewgraph 8). This is a very small near-field problem with model size of 0.75 m x 0.5 m. A heater is located at the lower left part of the boundary, and there are four fractures. This problem is proposed by AECL (Atomic Energy of Canada Ltd.) to closely study the behavior of fractures near the heat source. The next case is test case 1 (Viewgraph 9); this case was conducted by NGI using their coupled shear-flow test facilities. The boundary conditions are shown in the viewgraph. A pair of flat jacks were used to supply normal force at the boundaries to induce shear and normal force in the fracture. This kind of test is very poor when you shear you may introduce some bending moments in the sample. Consequently, a nonuniform stress is generated along the fracture. In the next stage, we improved the test using CNWRA coupled shear-flow test frame to study coupled M-H behavior of rock joints. The current status is like this: we have finished three cases so far. The Phase 1 is about the

2 ORGANIZATION



Viewgraph 4

4 DEFINITION OF BMT AND TC PROBLEMS

BMTs and TCs of the DECOVALEX, Phase I, II & III

	Physical Processes
BMT1	Coupled T-H-M processes in fractured rocks with two orthogonal sets of persistent fractures. Domain size 3000 x 1000 m, 2-D far-field problem
BMT2	Coupled T-H-M processes in fractured rocks with four discrete fractures. Domain size 0.75 x 0.5 m, 2-D near-field problem
BMT3	Coupled T-H-M processes in fractured rocks with a realistic fracture network from Stripa Mineof 6580 fractures, Domain size 50 x 50 m, 2-D near-field problem
TC1	Coupled shear - flow test of a single rock joint, 2-D problem.
TC1:2	Same set up as TC1 with different joint properties
TC2	Fanay-Augères (France) experiment of coupled T-H-M processes in fractured rocks, 3-D problem. Domain size 10 x 10 x 5 m
TC3	Large scale experiment (Big-Ben) of coupled T-H-M processes in engineered buffer materials
TC4	Triaxial experiment of coupled normal stress-flow processes
TC5	Coupled shear-flow experiments of a single rock joint on a direct shear machine, 2-D problem
TC6	In situ Borehole injection experiment at 150 m depth in fractured rock

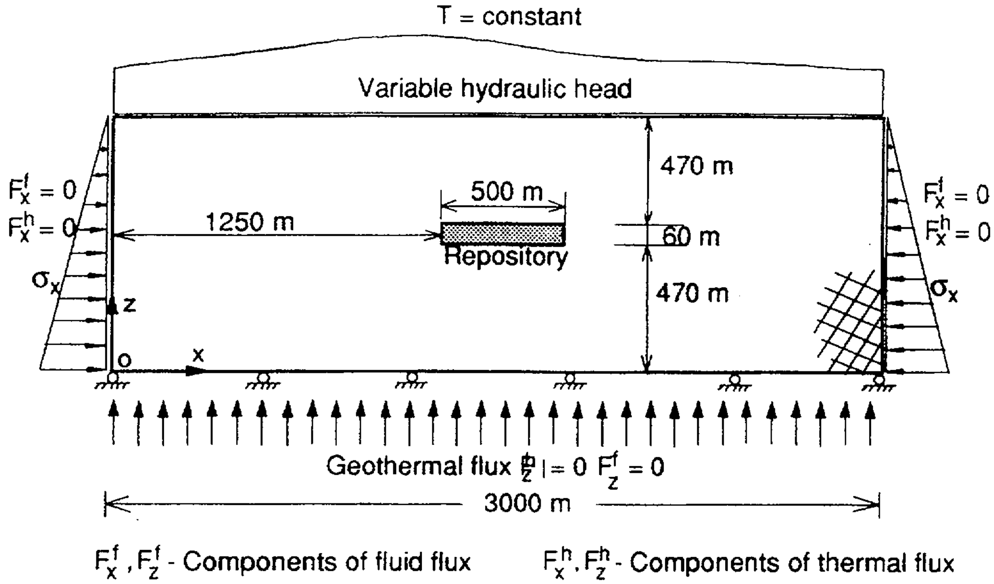
Viewgraph 5

Codes and their characteristics used in DECOVALEX

Code	User	For	Major Characteristics
MOTIF	AECL	BMT2 TC1 TC1:2 TC6	FEM solution of transport problems in porous/fractured media for 3-D problems of transient and steady-state groundwater flow, heat transfer and quasi-static T-H-M processes
THAMES	KPH	BMT1 BMT3 TC3 TC5	FEM solution of coupled T-H-M processes for porous/fractured media with crack tensor approach
ADINA-T & JRTEMP	VTT	BMT2	2-D FEM solutions of heat transfer (ADINA-T) and deformation analysis (JRTEMP) without fluid flow
ROCMAS	LBL KTH	BMT2 TC1 TC1:2 TC6	FEM solution of coupled T-H-M problems with special joint elements and a simple differentiation of a non-incremental formulation
CHEF HYDREF VIPLEF	ENSMP	BMT1 TC2	FEM solution of coupled T-H-M 2-D problems for porous or/and fractured media with special joint elements
TRIO-EF & CASTEM-2000	CEA/DMT	BMT1	FEM solution of coupled T-H-M 2-D & 3-D problems for porous media
NAPSAC	AEA	BMT3 TC6	DIscrete fracture network solution for ground water flow
FRACON	AECB	TC6	FEM solution with joint elements for coupled H-M processes of continua
ABAQUS	CNWRA CLAY	TC3 TC5	FEM solution of coupled T-H-M problems for continua
FLAC (2D)	ITASCA	BMT3	Finite difference solution of coupled T-H-M roblems of continua
UDEC	CNWRA, NGI INERIS ITASCA VTT	BMT1 BMT2 BMT3 TC1 TC1:2	DEM solution of 2-D, quasi-static, problems of coupled T-H-M processes for discrete, deformable block assemblages
3DEC	INERIS	TC2	3-D distinct element solution for coupled T-M problems of discontinua

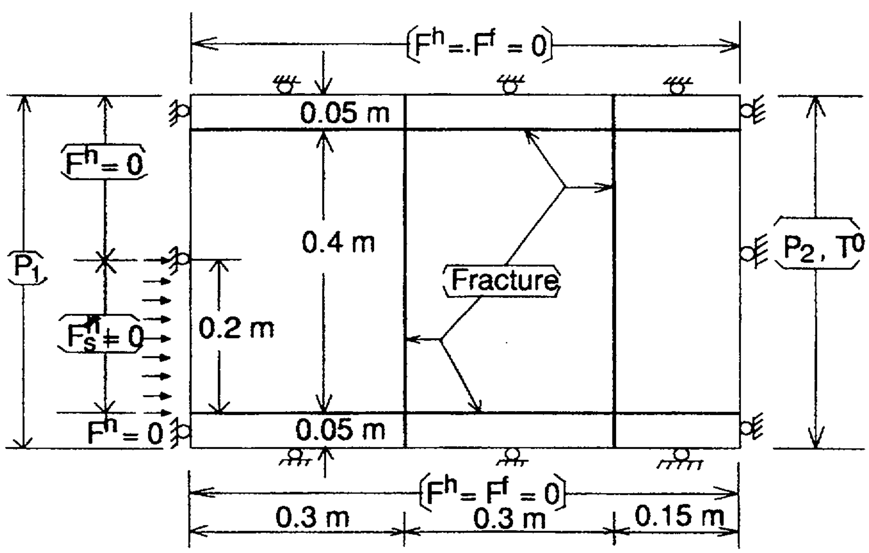
FEM, FDM, DEM and DFN methods have been applied for DECOVALEX problems.

Viewgraph 6



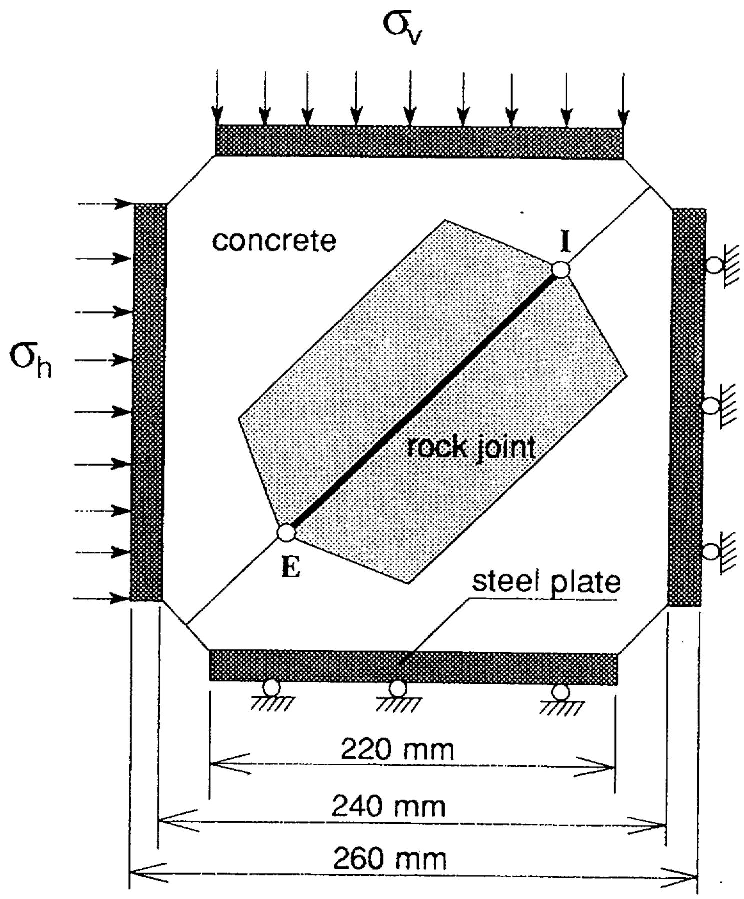
Thermal, hydraulic and mechanical boundary conditions of the far-field model, BMT1, DECOVALEX, Phase I.

Viewgraph 7



Thermal, hydraulic and mechanical boundary conditions of the Multiple fracture model, BMT2, DECOVALEX. $P_1 = 10$ KPa, $P_2 = 11$ KPa, $F_s^h = 60 W/m^2$ (heat source)

Viewgraph 8



Geometry and boundary conditions of TC1, DECOVALEX,
I - Water injection point; E - Water outlet point.

Viewgraph 9

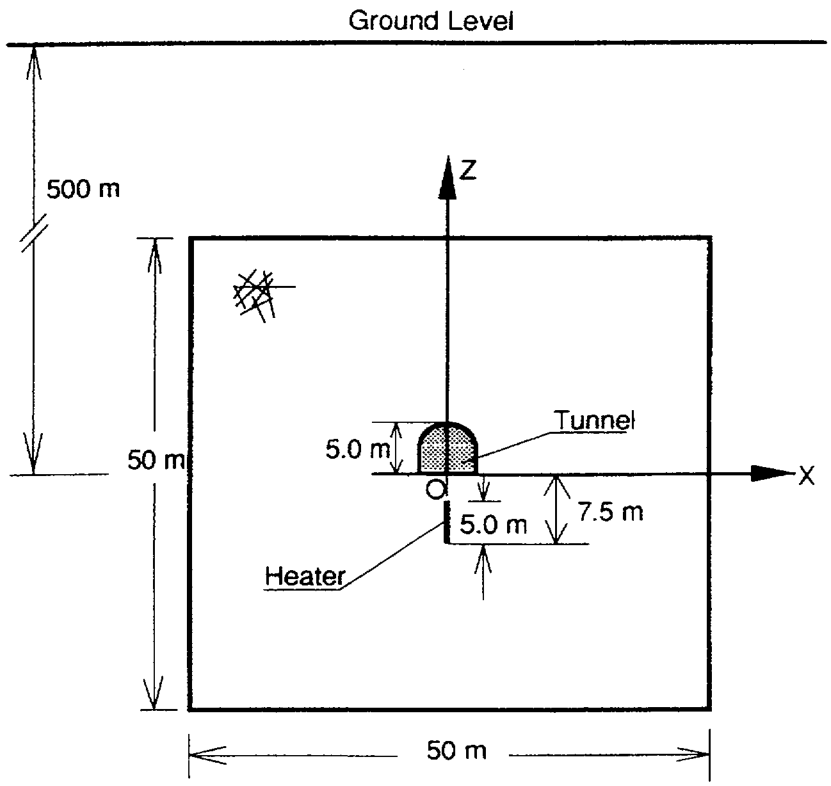
benchmark 1, benchmark 2 and the TC1. This phase was finished last year. Phase 2 is about benchmark 3 and improved TC1, and was finished in June of this year. Now Phase 3 is about all the rest of the test cases. It will be closed next month in Paris. Almost every party in this project is happy with the progress and the spirit of the project so far, so we are planning for the next three years contemplating on large-scale test cases. The next meeting will be probably in this room next year in March.

Next I'll show you one example of how we define our problems. This is BMT3 (Viewgraph 10). We define this near-field problem with a 50-m by 50-m rock mass. A tunnel is placed in the center of the model where a heater is also located. Now we have a fracture network looking like that shown in Viewgraph 11. There are 6,850 fractures in this piece of rock. It is up to a research team to define what kind of method they want to use. The mechanical boundary conditions are shown at the bottom of Viewgraph 11. We have initial stress in the vertical and horizontal directions. A self-weight of the overburden rock is applied on the top of the model with a fixed boundaries on the other three sides.

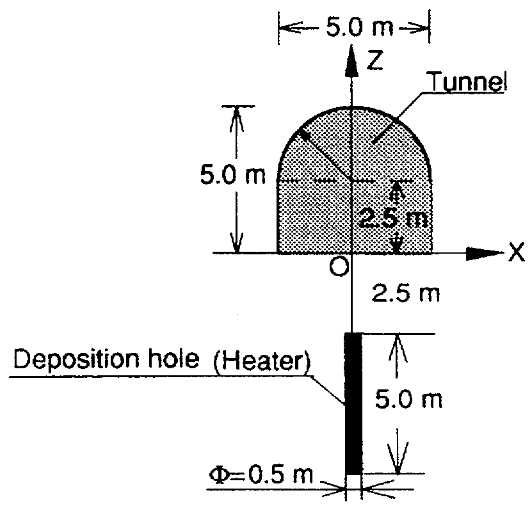
The thermal boundary and initial conditions are defined in Viewgraph 12. We have a constant temperature at the top and zero flux around the side and bottom. We have negative exponential law for the heater source and a convective boundary for the boundary of the tunnel. Initial thermal condition is a constant temperature in the domain. For the hydraulics, there is a constant pressure at the top, zero flux at the other boundaries, and along the internal surface of the tunnel is assumed zero pressure. This turned out to be a very interesting problem. We have 8 teams study this problem. The biggest issue is how to represent this fractured media by different teams. We have finite elements, we have UDEC, we have FLAC, we have a fracture network model. It turned out that we have eight different models. We even have three teams using UDEC; we have four different models. The results obtained by each team are different.

The loading sequences for BMT3 is shown in Viewgraph 13. You have three sequences of loadings. You have to reach the initial hydromechanical equilibrium (Sequence 1). At the end of the sequence 1, we set the time equal to zero and the tunnel is then excavated. Once the tunnel is excavated, the initial equilibrium is disturbed; you run the model until you reach the new equilibrium state. And at the time we call t* at which you introduce the heat. We run the simulation for 100 years. To compare the results, we defined some 8 points, from A to H and two profiles in this region (bottom plot of Viewgraph 13). For example, along these two profiles, we compare temperatures, stress, displacement, and across the tunnel surface we compare the water flux. At these 8 points, we compared the temperatures and the displacement and the stresses, too. To give you one example, Viewgraph 14 shows the results of temperature along the vertical profile predicted by different teams using different methods. The temperature prediction is very uniform from using different computer codes. But the stresses are quite different, some have really large stress concentration on the bottom corner of the tunnel (Viewgraph 15). You can see the real large discrepancies from the different teams. The water flux is even more different (Viewgraph 16). That's a typical result so far.

6 EXAMPLE - BMT3



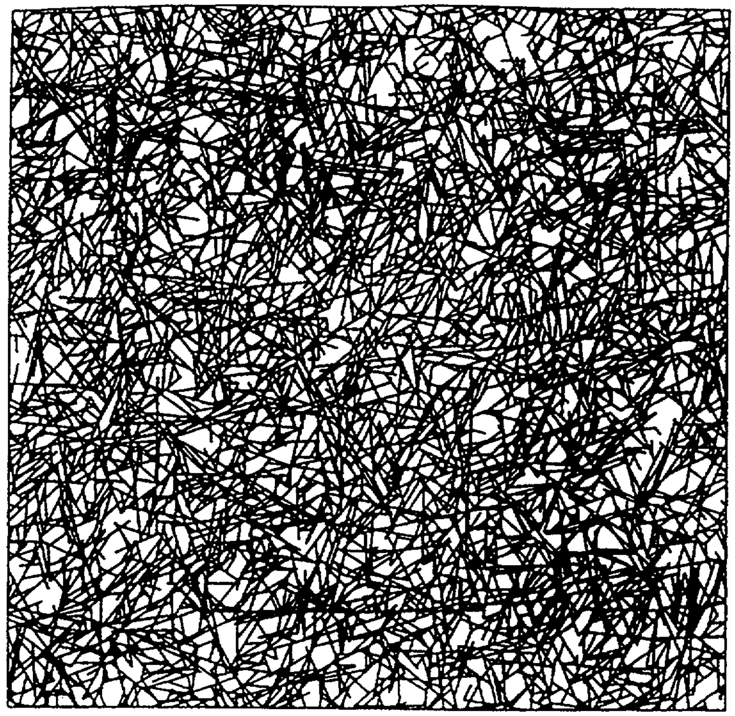
a) Overall Geometry of the model



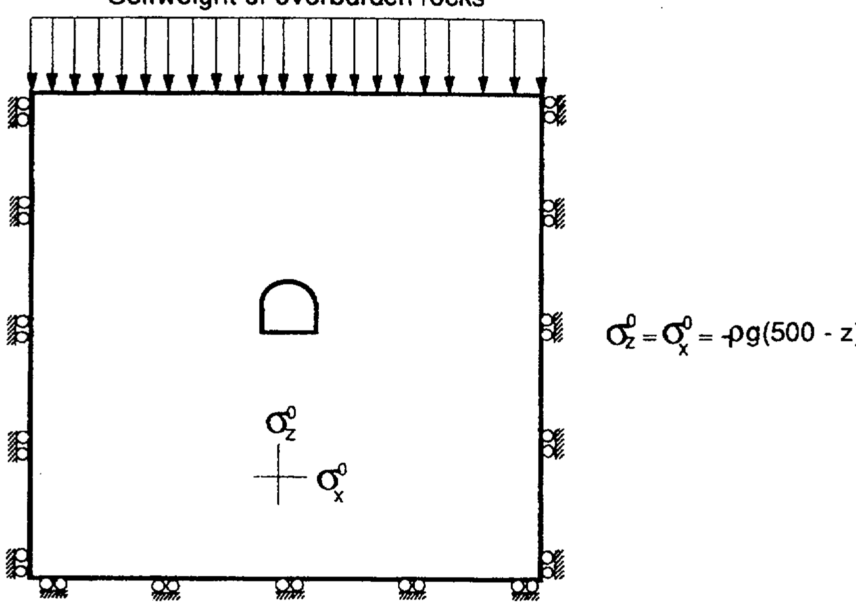
b) Details around the tunnel and deposition hole

Problem geometry of BMT3

Viewgraph 10

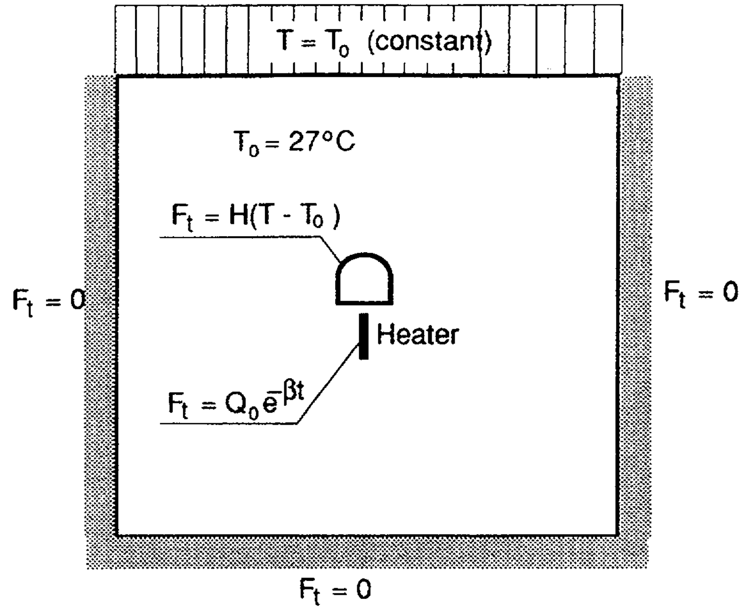


The Reference Fracture Network for BMT3

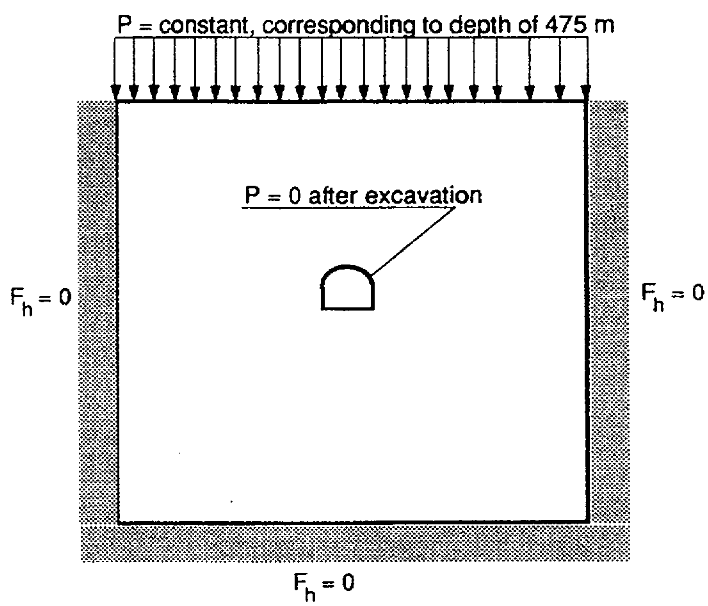


The initial and boundary conditions for mechanical effects.

Viewgraph 11

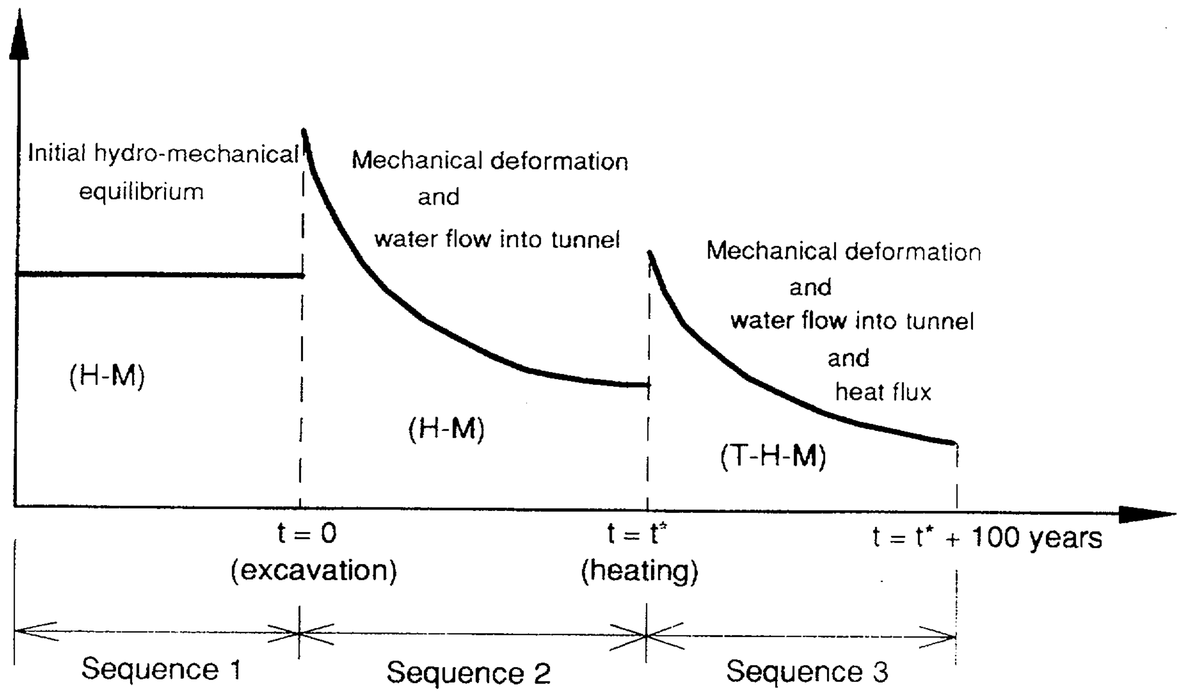


The initial and boundary conditions for the thermal effects, and the heat source, BMT3. T - temperature, T_0 - initial temperature, F_t - heat flux, H - surface heat transfer coefficient (= 7 W/m^2°C), Q_0= 470 W/m^3 , β = 0.02/year.

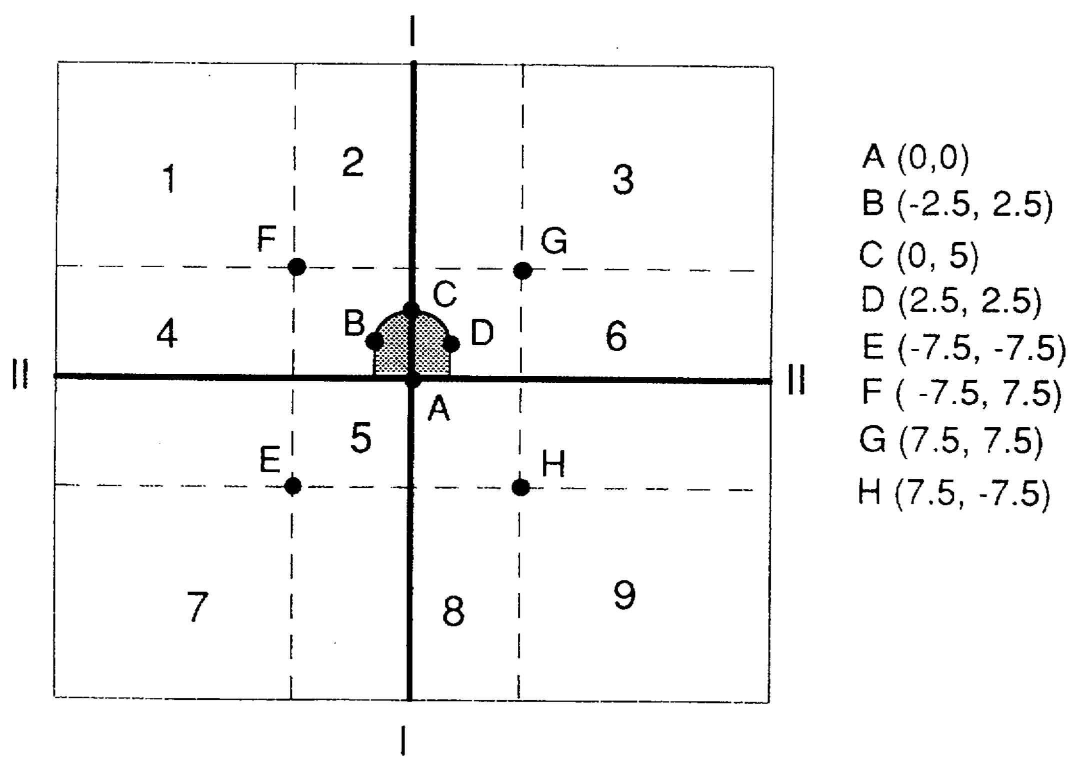


The initial and boundary conditions of the hydraulic effects, BMT3. P - water pressure, F_h - water flux.

Viewgraph 12



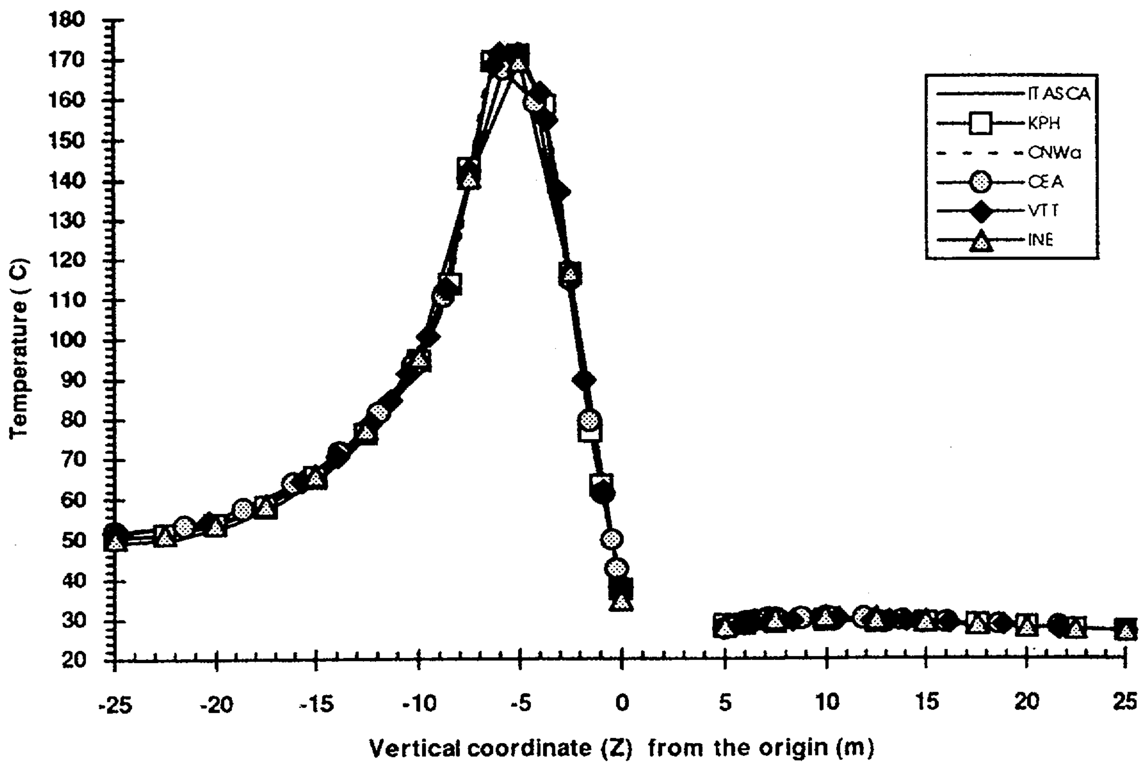
Loading sequences for BMT3.



Output specifications for BMT3.

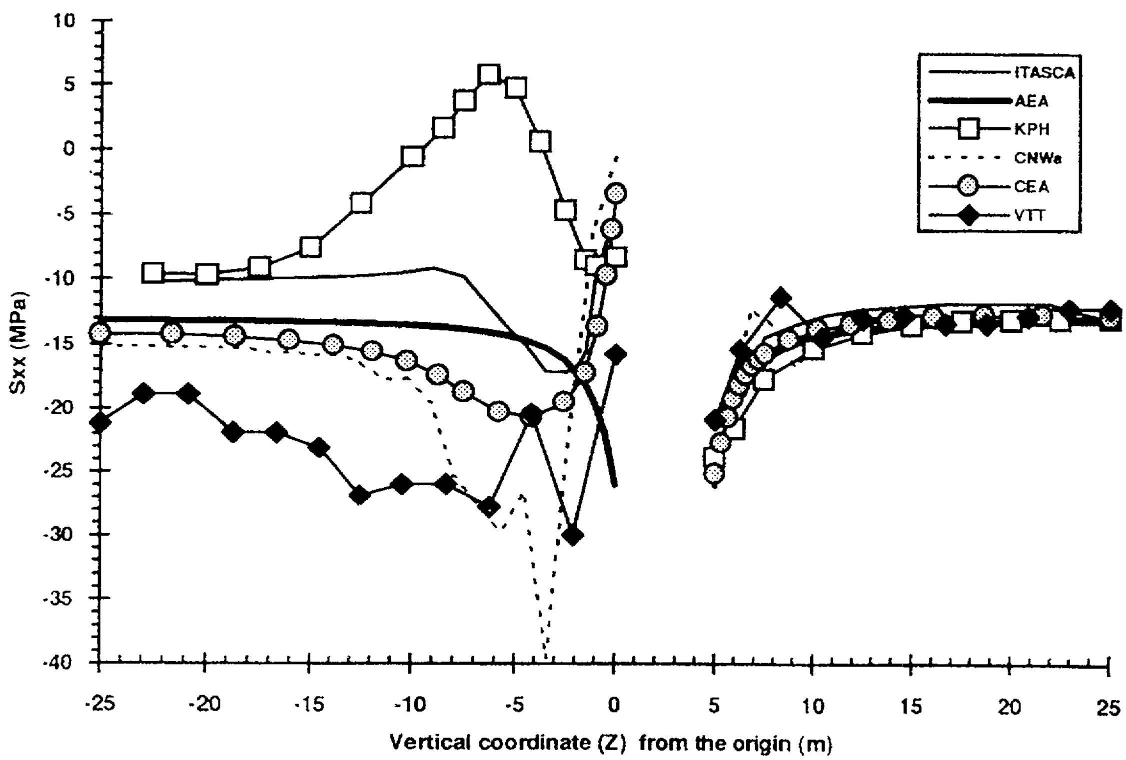
Viewgraph 13

h) Temperature along profile I at t = 4 years for BMT3

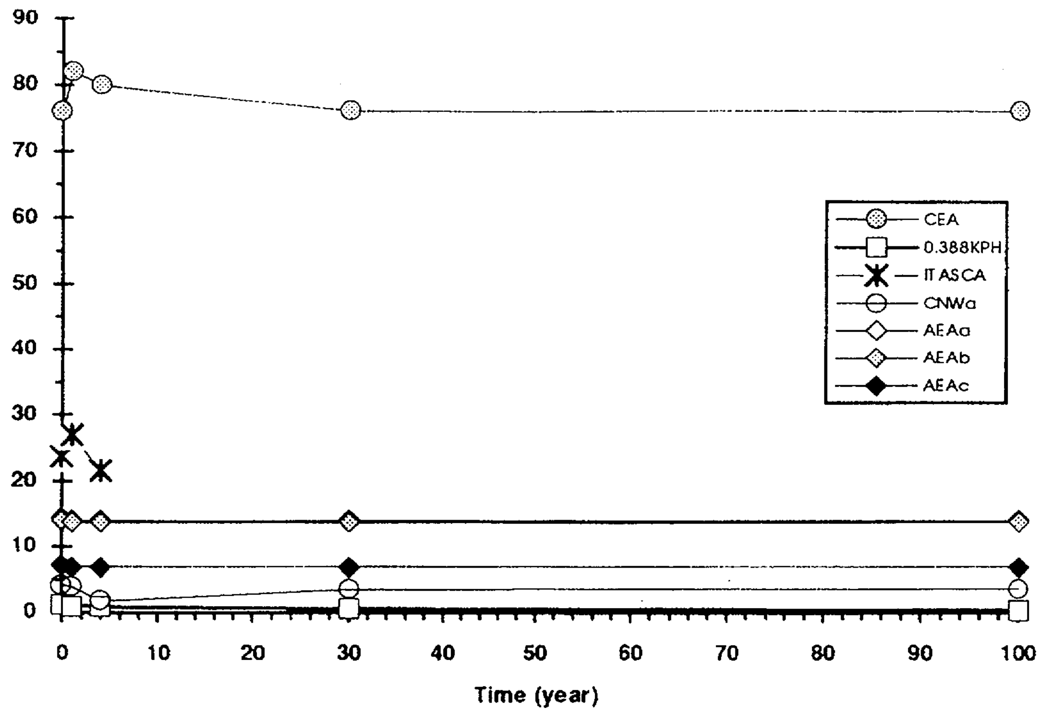


Viewgraph 14

g) Horizontal stress (Sxx) along profile I at t = 4 years for BMT3



Viewgraph 15



Water flux $(\times 10^{-8} m^3/s)$ across tunnel surface vs. time.

Viewgraph 16

If you want more information you can talk to me. We have finished two reports: Phase 1 and Phase 2; the Phase 3 report will be finished next year, so if you want more information, just call me.

R.B. Codell:

Thank you. I would like to start off with a question of my own to the panel. Since with these rock mechanic type problems you're really dealing with heterogeneous materials, and a lot of the problems seem to be based on uniform materials and yet even for a single run some of these calculations are extremely lengthy, how do we take these results and turn them into something useful? After all, one of the objectives here is to get something that we can start using in our performance assessment. What kind of approaches do people see where we can get meaningful results without repeating runs that take hundreds of hours each and it would have to be repeated hundreds of times because of the nature of the medium we are dealing with? Does anyone have any thoughts on how one goes about extracting these complicated results to turn them into something useful for performance assessment?

L.S. Costin:

You're talking about, I mean, variability of the rock mass over the site. Of course, one of the main parts of the site characterization program is to really understand what the nature of the variability of the material is over the site both from a rock mechanics point of view, and hydrologic and geologic perspective. The approach that at least is being considered and is being pursued is to look at a number of avenues primarily based in terms of looking at the geostatistics. You will never have enough data of the right kind to resolve all of the nuances of what is going on. You will have, always be working in, a world of extremely limited data compared to the complexities of what's going on. One of the ways to try to resolve that and in your modeling is to look at techniques like the geostatistics and to take, for example, the intact rock properties and being able to, with selected measurements over the site, statistically project what the variability of those properties may be over the site. Correlations with other properties also help, for example, I think that Ron (Price) probably showed yesterday the correlations with porosity. Those are measurements that are fairly easy to take and you can get a lot more over the site so you have at least some basis for looking at what the variability is over the site. What we have done is to take that information and put that into our models; and how that's done is you do a number of geostatistical realizations to look at what the variability is and you base your calculations on that. The selection of properties in a given location has some probability density and so you can do for five runs maybe, you can get a sense of what the impact of the variability of those properties, fracture properties, geologic properties, intact rock properties, etc., is over the site. And that's sort of a basic approach. The real problem comes in is how do you validate your modeling approach. There's been a lot of looking at comparison of codes among themselves. But if you begin to compare the results of those kinds of models to real experiments, where you can't really model the details of the things that you're going to see in the experiments like large heated block test, what you're going to find is that there is large discrepancies and the discrepancies may be real or may be because of the modeling techniques you're using. Those are the kinds of issues that I think are going to be more fundamental in trying to resolve. I think geostatistical methods at least provide an avenue for looking at the variability in a fairly simple way.

S. Blair:

I have sort of a follow-on question. Is there a particular quality of model that you've developed that seems to run in less time or is a better approach? Can you make any recommendations to that effect as far as to help us in where we can go, which avenues are better to pursue in the future?

M.P. Ahola:

Yes, I think they've come up with some better ideas or better techniques with these continuum models that are incorporated in the anisotropy or due to the random fracturing with these homogenization techniques and being able to come up with permeability tensors that take into account the anisotropy but I think that I agree with Larry (Costin) that there has to be some sort of statistical or probabilistic approach to that sort of modeling.

R.B. Codell:

It seems to me that one example that I can think of in Yucca Mountain, one of the things we want to know dearly, is what kind of dripping behavior you might have in the near field. This seemed to me would be very closely tied to actual discrete fractures and a continuum model might not tell you this kind of information.

L.S. Costin:

I think that one of the faults you can run into is to assume that one modeling approach is going to have fairly broad application and be able to tell you lots of different things about different applications and I think that's a real mistake. As you've seen today with sort of broad range of approaches; each of which has its focus in terms of detail and has a set of assumptions that's incorporated into it. You need to choose the right approach the right type of model for the particular aspect of what you're going to look at. And certainly if the kind of a problem, that you'd want to look at, has discrete fractures in it and has some mechanical hydrological coupling or thermomechanical hydrological coupling ability in it, it may be very fine for looking at one or two fractures, but when you try to look at mountain scale it's going to be hopeless, in which case you'd need other approaches. So I think it would be a mistake to begin to focus down on, say, these are the four or five codes that you know are going to do the job for us. I think there's always room for another approach to look at specific problems like that and to try to glean whatever information you can out of that.

S. Blair:

In addition to that, I think that a lot of the work has shown that channeling is what is important, connectivity of the fractures and if you used a discrete element model, you may not get all the fluid coming in only one fracture like has been observed, and so you need something to couple this discrete element model with some sort of a networking theory or something, you might as well go back to finite element. So I agree that you need to, especially for a problem like individual fractures, which was going to bring the fluid into the repository, it sort of tried this channel approach and it's not really turned out to be what was expected so I think that we need some new approaches there and I'm not sure discrete elements are necessarily the answer to that one.

L. Jing:

Within the DECOVALEX project, we felt that both continuum and discontinuum approaches have their shortcomings. For the discrete element methods, the problem of including all fractures is difficult. In practice, we do not know the exact geometry of the fractures. For continuum approach, the problem is proper homogenization schemes for establishing equivalent properties of the fractured media for continuum analyses. For BMT3 in the DECOVALEX project, two research teams used finite element method, using the same theory of Oda's Crack Tensor approach, but turned out two different values for the Representative Elementary Volumes (REV). The relationship between REV (representative elemental volume) and the size of elements is also not clearly understood. In addition, we found that the constitutive models for rock fractures which have been developed so far are not good enough for coupled processes. They work in some cases, but do not work in other cases. For the coupled T-M-H processes of fractured rocks, there are a lot of things which are not fully understood and there remains many fundamental researches to be pursued.

L.S. Costin:

There's one issue that may not have been discussed explicitly but certainly implicitly in almost everything that's gone on this morning and that's the problem of scaling. If you look at individual behaviors, how do you scale intact rock behavior to a rock mass? How do you scale laboratory fracture properties into rock mass fracture properties? How do you scale thermal mechanical properties or hydrologic properties that are measured on laboratory scale into a rock mass scale? Is the coupling that you observe on small specimens, or bench scale type experiments the same kind of coupling that occurs in a larger scale, the coupling change coupling. Those are issue that I think are really fundamental; we need to be able to take the kind of information that we are able to get in small scale field tests and laboratory tests, and make use of it in terms of performance assessment which has to occur on a large system scale. In order to be able to really to abstract that information onto a large system scale, one has to have a good idea what the fundamental scaling principles are, in order to build those models or build those abstracted models from the basic subsystem models. I think that's an issue that hasn't been well addressed in terms of looking at what are the appropriate scaling laws and how do we demonstrate that these in fact work.

R.B. Codell:

I think that's a very good point. I know that in hydrology, not me personally, but our contractors and the literature are just starting to get around to issues of scaling up to large rock segments especially since in the field of unsaturated flow it's hard to do large-scale tests. You can do them in saturated regimes but for unsaturated cases you're pretty much confined to small samples and scaling up is a real headache. One point, it may be food for thought, is that I remember explaining the problems we had to a physicist and I think these kinds of problems are ubiquitous across all fields of science. He said why don't you look at statistical mechanical approaches.

S. Blair:

I think statistical mechanics, actually may have methods that could be extracted but there's been a fair amount of statistical mechanics actually gone into some study of rock properties and it's been disappointing. I've adapted some of that into a model I've just built but it doesn't readily translate. It's not a bad idea, and it might fit into particular places but there's some problems. It doesn't readily translate.

W.C. Patrick:

I'm not terribly surprised by the response to your question with the possible exception of Larry's last remark. Almost all of the discussion related to how we were possibly going to lead to the point of abstraction so that total system PA could progress. All of the commentary was to go to ever-increasing levels of detail to include many, many other phenomena that have not been adequately addressed and evaluated in the minds of the rock mechanicians. I think one would conclude from the observations made thus far is that at least this group or those speaking from this group don't feel that there's enough of a basic understanding yet to proceed to the process of abstraction. I don't know if I agree with that personally. I guess I'm a little, in fact more than a little, surprised that some basic things—I believe you used the word "strategy" in your comment—some basic strategies could be used, could be contemplated by the group here. For example, such things as to simply recognize that a total system performance assessment cannot possibly, without two or three orders of magnitude improvement in computing speeds anyway, model the evolution of every single detail of the development of this system. One cannot start out by simply taking Mikko's example, or Dr. Jing's example, containing thousands of fractures that are "important" or "pertinent" or "relevant" whatever word you choose to a single drift, multiplying that by hundreds of drifts—thousands of square meters of area that would have to be modeled—unless we somehow as a community can determine strategically how to approach the modeling. Without such a strategy, I would doubt that we would ever get to the point where the rock mechanicians are going to be able to provide information useful to performance assessment, and perhaps they won't even be able to provide information useful to design. We must somehow cut through that gordian knot of how much detail has to be included in these models or, alternatively, develop a strategy wherein detailed modeling is done on perhaps a smaller scale and then the effects that are pertinent to performance are then abstracted and put into a total system performance assessment in some much more broadly conceptual framework than the detailed mechanistic framework that seems to be suggested by most of the commentors here today.

L.S. Costin:

That's in fact exactly the process that the total system performance assessment goes through. I mean there is a hierarchy of models and at the very bottom of the so-called performance assessment pyramid. I don't know whether you've ever seen a diagram; it's described as a pyramid where the bottom of the base of the pyramid is essentially your most basic subsystem models in which you do look at the details of what's important but you don't then blow that model up to ever increasing volumes and try to model the entire model using that kind of level of detail. What you do is you basically abstract that, the results of that and figure out what's important and what those relationships are and abstract those to the next level. And then you build a slightly bigger model or incorporate more subsystems together to look at the effect of that; you abstract the results of that and input into your system level model which ties all these things together and gives you at the end say total releases or releases over time. So I think that the strategy that you talked about is in fact the essential strategy where the major holes in the system are right now, at least as I see it, at the bottom of the pyramid. We don't understand the basic processes well enough in order to be able to model them and then to abstract those results in some way. We don't know what's important. We may be missing something that's important when you begin this abstraction process to the top of the pyramid. Therefore, you may have a fundamental flaw in your total system approach which has these little subsystem models in it. So in many respects, I mean things like how many waste packages are going to get dripped on, well in order to determine that, that may be a system level question you're looking at and you do many realizations and you end up with a probability curve of in any realization that anywhere between 100 and 200 packages might get dripped on in any one period of time, but how do you determine that. You have to go down to the very fundamental level looking at drift scale with detailed fractures, can any waste package get dripped on? Or what's the possibility of how is the local hydrology going to have to be in order for water to move to the waste package. You've got to understand that, once you understand that, then you can say, okay, here's a probability curve over the site at any point in time, it's likely that any given waste package will get dripped on at this probability level or whatever. And that's what you put into your higher level system model to begin rolling up into your source term; and given that then you have a corrosion rate associated with that and so on down the line. But I think what you said is essentially correct. You cannot use the bottom of the pyramid models directly at the top; you've got to have several levels of abstraction. The problem is that the bottom of the pyramid, with all of the things going on in terms of source term, in terms of thermal hydrologic mechanical coupling, is so complicated that it's going to take a lot of time to kind of sort that out and even decide what's important and what's not important. I think we have a good start and I think we've done a few total system performance assessments in which, by looking at the sensitivity studies based on those, you could kind of guess at what's important and then go back down to the bottom of the pyramid and focus on those. We need a better understanding of these particular issues because we see at the top of the pyramid that the results are fairly sensitive to those assumptions and that's the kind of process in terms of iterative performance assessment that you really need to go through.

M. Nataraja:

I would like to focus the panel and the audience on one question that was raised this morning. I guess the most important issue here is this repository is nothing different from any other problem if we didn't have the heat load. So the basic question was, at what stage of this program, we should know what the thermal load is likely to be. I heard some arguments yesterday that we need some flexibility, we should be able to go the cool concept, the hot concept, whichever. I understand and appreciate the need for flexibility but we've got to have some focus; otherwise all of these strategies that we are discussing may be totally out of focus if we don't know what the thermal load is likely to be. And I think that we need to address the question of at what stage in the program should we fix the thermal loading; unless we know, otherwise I think that most of our questions might be going in the wrong direction. So I'd like some thoughts.

S. Blair:

Mike Voegele could probably give you an hour long discussion on that.

R.D. Manteufel:

I really have just a brief comment, that is that the thermal loading should be based on input from the rock mechanics. How strongly does the thermal loading affect rock mechanics issues and where has that been fed into the iterative performance assessment decision making process. I think that's critical. Turn the question around, you're a rock mechanicist, you tell me how important it is and then we'll use that to make the decision or other people making decisions about what actual thermal loading will eventually be.

S. Blair:

I think that, as I said in my speech earlier, I think that rock mechanics is actually a secondary issue compared to the hydrology and geochemistry when it comes to thermal load. Certainly from the mechanical standpoint, we can handle much, much broader range of temperatures than the hydrologist or geochemist can, as far as the behavior of the repository and I'm sure Mike Voegele can expand on this at length. But I don't think that we—that's sort of an egocentric type view—from a rock mechanics standpoint, have been able to get very far with it.

M. Voegele:

This is a very difficult question that we've struggled with for quite some length of time on the project. I think Larry (Costin) remembers all the time we've spent trying to develop strategies for how you could address the additional design criteria for 10 CFR Part 60 in terms of rolling those up into a demonstration of compliance with the performance objectives. I don't know the answer to the question of when we can fix the thermal loading. I do believe I know some things about what we have to do before we can do that. I'm very frightened that the project moved too quickly to fix the thermal loading for the particular reason that the NRC has some procedural requirements that make us look at things like alternatives to those major design features that are important to waste isolation; and clearly thermal loading is one of those variables that is always going to have to be classified as an element of those design features that are important to waste isolation. I think if you rush to fix the thermal loading you will be later subject to the criticism that you didn't investigate the alternatives thoroughly. In order to investigate the alternatives thoroughly and move the program forward on a meaningful schedule, I think you have to turn to an approach where you look for some flexibility. I recognize that this means that we will not be necessarily finding a definitive thermal load very quickly, on the other hand, if there's coupling between these various parameters and the couplings that you're talking about are functions of temperature then I think it's appropriate to look for the range of responses you can have. The only way that I can deal with that is to say that I must proceed with enough flexibility to allow me to accommodate all elements of performance. That's the only way I can say that. I can't personally commit; and I don't think the program can commit to fixing the thermal loading until we understand what the real effects are between the couplings between hydrology and thermal loading and perhaps geochemistry in the program. Maybe Larry (Costin) has another perspective on that. We've fought this problem for years.

L.S. Costin:

I think that's essentially right. The problem is even more complicated than that. Primarily because what does thermal loading mean. Thermal loading is the amount of kilowatts in this piece of ground vs this piece of ground over here. In terms of any repository design you're going to have a range of thermal loadings at any given spot. In fact, you may have things like abandonment zones where you have poor ground conditions and you don't want to emplace waste and so you may have cool spots within the repository and hot spots within the repository depending on the waste stream that gets emplaced in a particular place. But those questions aside, if you look even back as far as the SCP (Site Characterization Plan), the question was addressed in terms of how do we stay within the parameters to meet the performance goals. We set sort of tentative thermal goals that had to be met in order to, what we felt at that time, stay within the parameters of meeting those guidelines. Those are now, with new information, being looked at again and again to determine if those are really the right goals or if we are even looking at the right phenomena to set those goals. And I think that's part again of the iterative nature of the performance assessment process. If you look at the last total system performance assessment that was done, I think you'll see in the bottom line in terms of the total release curves there's very little difference between high thermal load and low thermal load. They both perform on a system level about the same. Again, the devil is in the details. Those system level models did not incorporate a very good abstraction of what you might say is going on in the details of a thermal hydrology. We can't do those kinds of calculations yet because we don't understand the process well enough. I think we can do the calculations but do we believe the results? So, in doing the system level abstractions you have to say do we have a dryout zone or don't we have a dryout zone; well, let's see what happens either way. It turns out that because we didn't really consider much in the way of the details of those kinds of things that really the effect of low thermal load, high thermal load, it comes out in the wash. I mean, there are other major drivers to what gives you the total releases than that. Again I would hark back that in order to look at what is the thermal load that will eventually give you an optimally or a well performing repository the strategy that's currently being pursued is to remain as flexible as possible. Until such time as you get some kind of confirmation that this range of thermal loads which fits within our design goals will work and so you'll see my guess is that you will be sort of close to the performance confirmation period before you really have narrowed things down to say we're going to tailor things to this thermal load or we have a thermal management scheme that is going to allow us to emplace waste in such a way that we get the right kind of performance.

R.B. Codell:

We were planning to break up at 12:30, I'd like to take one more question from the audience. Who has the microphone? And then we'll have to cut it off and have conversations over meals.

E.J. Cording:

I guess it's got to be a good one yet. Larry (Costin), you described the heat potential for heave and a very large mechanically disturbed zone that seems to me to be a somewhat different view than we've discussed in the past. It seemed to me that the mechanical zone, in what I understood of ranges and conditions that might occur, is something that one could perhaps conservatively define. A mechanically disturbed zone would affect fracture flow that you define under different ranges and one could come up with some reasonable ideas as to what might happen; the heat pipe effects and just the normal dripping down in. You're coming into a more permeable zone. I think one could, at least I would think, describe that performance assessment and some of these issues where the rock mechanics is perhaps a secondary supporting issue but not insignificant but something that perhaps we could define such that we are not in a box where we can't provide useful information even at this point. The thing that concerned me, and I wonder if Larry (Costin) could briefly respond, was this possibility of large changes in permeability mechanically far away; and I just didn't think that the strains were going to be that large even though you're going to get some heave that you're talking about that much strain at distances in the far field and something that certainly if it's a significant question it needs to be at least put to rest.

L.S. Costin:

What I was trying to demonstrate was that simply looking at 2D near-field kinds of models and the effects that may have on the near field and trying to establish what a disturbed zone might be. This is kind of a myoptic point of view, I think. There are effects that occur on a large scale because you basically are heating a central core quite hot and it has quite a bit of expansion. You have an unconstrained surface up there and highly constrained other boundaries. Things are going to move upward. And how much depends again on the variability of the thermal expansion, how that occurs ...

E.J. Cording:

Do we have some sort of bounding. I don't like to use bounding characteristics, because I think we need to get some specifics on certain issues; but isn't there some way you could at least say what's the range of disturbance? Do you have something on that?

L.S. Costin:

We have for some parametric studies to look at that not in a whole lot of detail but the upper limits are about what I showed you. I mean that's the highest ...

E.J. Cording:

Heave is one thing, but strain is what we're concerned about.

L.S. Costin:

Actually the effect in the near field is one as you get high horizontal stresses, those have high normal stresses on the vertical fractures which are the ones which you are most worried about. Therefore one would assume that you're going to change the permeability for vertical flow especially for vertical fracture flow by at least some amount. Whether that's significant or not I mean we can show that you crush fractures together you change the ability of those fractures to carry water by you know a factor of two, three, whatever. Is that significant? That we don't know yet. Really what in terms of integrated permeability are you going to change it that much so that you, in fact, don't get shedding off in these cool spots because they're the ones that are under the highest stress and where do you end up ponding water above the perching water above the repository that doesn't go anywhere, doesn't flow down the fractures because the fractures are pretty tight. The other effect is with this heating upward, a nice fairly unfractured layer of unwelded tuff above the repository that may serve as a barrier to infiltration and may have an extension zone that potentially would fracture this layer enough that it becomes more permeable.

E.J. Cording:

At this point, we need to put some ranges and numbers on it so you decide if it really is significant or just adding to a very long list of interactions.

L.S. Costin:

Again, that's where you have to do the sensitivity studies in terms of your total system. The only thing that really in the bottom line makes a difference is, is it going to change your system level assessment? Are you going to change releases to the accessible environment significantly enough that it's a real concern and that we don't know yet because we really haven't done the bottom of the pyramid type modeling in order to be able to do those kinds of abstractions on a system level to really take into account that level of detail.

R.B. Codell:

Before the roar of stomachs drowns everybody out, I think I'll have to bring the meeting to a close. It's been very productive. I thank the panel members and the members of the audience for good questions and answers and we'll see you after lunch and we can continue this during lunch. Thanks a lot.

CONCLUDING REMARKS

Wesley C. Patrick

Simon had asked that I try to do some sort of a wrap up, a summarization of where we've gotten to today. I always find it interesting the way we end up these various sessions. Bob hasn't had a chance to do it yet but the general remark was that I really thank you for a very productive session.

So what have we produced? What have we produced in these last several days, recognizing that the point of this meeting was not to come to any final conclusions with regard to what is and is not important. In four important areas (Viewgraph 17), we have come up with some observations that I think can help NRC's program and DOE's program as it goes forward and I'd like to go over those. They fall into four general areas, some observations that the assembled group has made with regard to the regulatory environment that we're operating within, some technical issues that have been raised and I've tried to regroup some of those, some programmatic issues and then some specific "calls for action," if you will, that various members of the audience have brought to our attention over the last couple of days. These in no way should be interpreted to represent a consensus nor should they be necessarily taken to be the views that I have or my organization or the NRC or anybody I've known past, present, or future holds. Is that enough caveats? That's even better than what Mike (Voegele) does.

With regard to the regulatory environment, I think there's general understanding about what the purpose of Part 60 is with regard to NRC's regulation for design, construction, operation and performance of a repository. PPA has focused DOE's attention most recently on Part 960, something that from at least NRC's and its contractors' perspective had kind of drifted into the background until recent months. Its focus has and I think always has been site suitability. But based on the discussions we've had here, I would ask "suitability for what" and the two items underneath that tie into that question. First, my understanding anyway, initially Part 960 was dealing with siting. It was dealing with selection among a variety of sites. It was developed in a fashion that was in regulatory jargon not inconsistent with Part 60. NRC played a role in concurring in the promulgation of that regulation. I would ask at this point in the program as Congress has already selected a site for evaluation, what role does 960 now play? It's clearly no longer a selection among sites, it's moved to some other role in suitability and I would suggest is it suitability for construction, suitability to perform in accordance with Part 60 requirements. Several commentors made remarks regarding this one, not too much was said in this regard, but I think it is another balancing point.

M. Voegele:

Wes, are these things you would like a dialogue on, or just curious.

TECHNICAL ISSUES

- Spatial Variability of Rock Property
- Representation of Rock Mass Strength
- Measurement/Calculation of Loads and Their Effects
  - In Situ, Thermal, Seismic, Excavation Induced
  - Hydrologic Impacts, Opening Stability
- Integration of Data Into Design
- Incorporation of Key Rock Mechanics Information and Effects Into PA
- Required Extent/Degree and Types of Coupling: Design and PA
- Evaluation of Long-Term Seal Performance

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Viewgraph 17

PROGRAMMATIC ISSUES

- Development/Modification of a Coherent and Consistent Design Methodology
- Consistent Implementation of That Methodology
- Focus on Integration of Field and Lab Data into Design and Evaluation (PA)
- Determination of Priorities of Testing and Measurement Programs
- Assessment of Adequacy of T&M Programs for Designs, Construction, and Performance; How Much is Enough? (Technical and Regulatory Perspectives)

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Viewgraph 17 (Cont'd)

REGULATORY ENVIRONMENT

- 10 CFR Part 60: Design, Construction, Operation, and Performance of Repository
- 10 CFR Part 960: Site Suitability—but Suitability for WHAT?
- Selection Among Sites Versus "Suitability to Construct" and "Suitability to Perform"
- Role of NRC in the Proposed Program Approach (PPA): Regulatory Interactions/Implications of Strategy to "Move Program Forward"
- Role of the Site Characterization Plan (SCP):
  - Everything You Needed to Know About the Site But Were Afraid to Ask?
  - Everything You Never Needed to Know About the Site But Asked Anyway?

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Viewgraph 17 (Cont'd)

CALLS FOR IMPROVEMENT

- Where's the Integration?
- Probabilistic Design and Evaluation—Quantification of F.I.?
- Move Beyond Statistical Treatment of Data to Mechanistic Understanding
- Better Understanding of Joint Roughness, Seismic Pumping, Near-Field Fracturing, Affects on Near-Field Flow, Extent of Sealing, etc., etc.—What Really Matters? (Performance Issue)
- Get Into the Field—Get Underground: Larger-Scale "Prototype Testing"
- Development and Implementation of Test Program Strategy That is "Robust" for Various Design Options

September 20, 1994

Viewgraph 17 (Cont'd)

W.C. Patrick:

Well, these are things that have come up from my notes anyway through the last couple of days. These are points that different people have raised and typically we have not been able to thrash them out as the discussion has gone on. I would ask the chairman if he or any other members of the audience are interested in spending any time on them. What I was trying to do was to log these and any others that we might want to notice. Some are remarks and I think NRC and DOE may want to revisit some of these; the ones that they would feel are most appropriate or most important to them.

M. Voegele:

I guess I would just hate to see you carry that particular issue out of this little conference because there is a very, I believe, there is a simple answer to that one and maybe we were just remiss in not bringing it up over the past couple of days.

W.C. Patrick:

Go ahead.

M. Voegele:

I think that the primary reason that 960 did not go away when the waste policy act was amended. I think to share information, many of us have wondered why the DOE did not take the step when the waste policy act was amended doing away with Part 960. And I think that the issue is, at least from my perspective, relatively simple. The waste policy act prior to amendment still had requirements for DOE to make recommendations to the President and although the guideline process embodied that within it and so if the DOE did away with 960 entirely, there would be not generated in the public forum if you will process that the DOE had committed to, to making this recommendation to the President. And so I think what we are, the way in which we are dealing with this issue today is that in fact 960 will be used for the secretary to, as quickly as she possibly can, make a technical decision to proceed with licensing for this site. I think there's been several public meetings where those particular issues have been discussed, there's information in the record I think that will bear what I said but I think that you just have to think as 960 today as the DOE's internal decision document that we will use over the next few years to try to make a conscientious and definitive decision that it is worth proceeding for licensing for this site.

W.C. Patrick:

OK, I think that's an important clarification. Another item that comes out of that and is currently under discussion within the NRC is NRC's role in the proposed program approach. I noticed just late last week I believe it was the state of Nevada issued a letter informing the NRC what their role under the PPA would be with regard to site suitability. There are certain interactions and certain regulatory implications of what NRC's role should and could be with regard to the PPA which, as Mike (Voegele) has characterized it, is really aimed at how to move the program forward, a strategy in that sense for moving the program forward.

The role of the site characterization plan was discussed in several different contexts, I was having trouble thinking of some reasonably accurate and slightly clever way of stating it but several people noted, I believe Mike (Voegele) and I think our DOE representative, Bill (Boyle), had also made note that perhaps some of us misunderstood what the SCP was intended to be. That perhaps it isn't everything you need to know about the site but were afraid to ask, but perhaps there were some other things in there as well. Speaking for myself, I somehow gathered the impression in reviewing that document, particularly in the context of what the regulation calls for in the siting process, that those were necessary things not nice to know or might want to know or could know. I think I was encouraged by what I heard Bill (Boyle) say, but I'm not quite sure yet that the substantive content of the plan as I understood it would be satisfied, whether the particular tests were done in number and style and form may or may not be accomplished.

From those regulatory perspectives, then, there are a number of technical issues and programmatic issues that flow out. We spent a good deal of time talking about spatial variability of rock properties and rock mass strength and how we might best represent those, both for design and for performance assessment. A third area that ties in with general rock properties and strength are how the various loads manifest themselves: *in situ*, thermal, seismic, excavation-induced loads, and what their effects are. And I think that there was a general sense, a general feeling, that of the two principal areas, during postclosure hydrologic effects were the most important potential impacts, and during preclosure, and to some extent postclosure with regard to the engineered barrier system, opening stability was probably the most pertinent effect that we needed to turn our attention to.

Integration aspects were also brought out. You'll see this appear both for technical issues and for programmatic issues because I think it has both elements to it. But tying the site data into design was clearly the focus of this conference and something that's going to need some additional attention. Incorporation of some of the key rock mechanics information and the effects that result from rock mechanics performance into PA also came up over and over again as a key area. It's interesting how apologetic certain performance assessment people are. Randy (Manteufel) is not fully trained in the art of performance assessment so he said: hey, I'm not a rock mechanician, what would I know about this sort of stuff? But I think this is an area where certainly not hard headedness, but firmness is called for. The people who are the PA synthesizers and abstracters of information do need to hold the subject-matter experts accountable. And the converse is also true. Because clearly if a model doesn't incorporate certain phenomena there's no way that we can use it for decision making as to whether those phenomena are important or not. And I think that's an important aspect that was brought up by several people.

This one I don't even think we barely touched on: all of the super complex couplings that Dick Codell's session focused on today. How do we know which of those are most important or important at all, either for design or performance? I don't for a minute think that the lists are going to be necessarily the same.

Evaluation of long term seal performance was the latest discussion that we had here. Again I think it's particularly pertinent with regard to the last presentation to recognize that it's not just the initial materials behavior that we're interested in, but it's the long term performance aspects, degradation of materials, and so forth that are the technical issues at hand. I gleaned five particular programmatic issues from the discussion that we've had here. The first one and the second one really need to be treated together, I think Dick Bieniawski's observation was "what integration?" and "what design methodology?" I really don't see a consistency there. Mike Voegele pointed out that he's convinced and he gave a presentation that the methodology exists and that the methodology is sound, but I believe speaking for himself, he said it hasn't been implemented as consistently and as thoroughly and as conscientiously as it perhaps could have been. Integration again is an extremely important part, we mentioned that in the technical issues area, and won't go back over it again.

Determination of priorities in the testing and measurement program is a central issue. I was a little surprised at how much time this group spent talking budgets for a technical workshop. Whenever I see that happening, the thought immediately comes to my mind that there are gut-level questions in people's minds about whether the right priorities are being assigned; that, or they either have questions about it or they just don't understand that it's been done yet. So, I think that it would be worthwhile for each of our organizations to independently re-examine what the priorities are here. And again I think it's a place where performance assessment can aid us but, particularly when we have an implicitly postclosure performance assessment, we have to be very careful not to allow PA to drive all of the decision making. There are valid design concerns and preclosure performance concerns that do require a very vigorous testing and measurements program. Adequacy of the testing and measurements program for all of these things is a key question I get asked this at least twice year, and our program reviews at NRC subject us to the question "how much is enough?" And the thing that technical people sometimes lose track of is their answer to how much is enough technically may be quite different from the answer for what is required in a regulatory construct. This concept of reasonable assurance that NRC is striving for comes immediately to mind there. For those of you who weren't there or didn't see fit to go to a presentation that was given by somebody with the letters ESQ behind his name, a gentleman by the name of Michael McGarry, III, I believe, gave a very informative presentation which is contained in the last international high-level radioactive waste management conference. It really ought to be on the must reading list for all technical people and their managers dealing with just this issue. Another very informative one that was given in the same session was by the Canadian Atomic Energy Control Board (AECB) representative, whose name escapes me right now. It's in that same session, if you were to pick up the international high level waste conference proceedings and look for McGarry's paper and the other AECB paper in there which looks at basic concepts in data gathering, fulfillment of data needs and so forth.

The last area I'd touch on and I think I've captured them all here, I may have missed one or two from this last session, are specific calls for improvement that were raised and I'll just leave for your contemplation the general question of where's the integration. There was an interesting discussion at two times, two points during the conference here dealing with probabilistic methods and design. There was some talk about the FI, the factor of ignorance, and I would ask the question: "Is probabilistic design just a way of quantifying factors of ignorance?" There was a sense that there's a need to move beyond the statistical treatment of data into mechanistic understanding. Raj (M. Nataraja) set the tone for that, giving us Holland's little four quadrant system and a lot of our discussion here and several of the papers focused on quite a collection of data treated statistically. There were questions and comments from several people, myself included, as to whether we brought all the mechanistic understanding into those that we really needed to. We need to understand everything better. The typical end of a research program is to conclude that further research is needed but again I included them because there were calls for improvement in each one of those areas and a bunch that weren't listed but what really matter from a standpoint of pre- and post-closure performance. That's what we've got to get to at some point. Getting into the field, getting underground, moving to large scale. One gentleman used the term prototype testing. All of those were specific calls for improvement in the program and, finally, the development and implementation of the test program that's robust for the various design strategies. The converse of that point was also raised: "How can you possibly go forward without a firm understanding of what your thermal loading is going to be?" I recall the comment to develop a test program strategy that's robust, that would test across enough of the range so that if at some point in the future we found that performance overall was better for a hot repository, then they would know enough to proceed with a license with a hot repository. On the other hand, if it showed that the cool repository was the better way to go from overall performance, they would have enough data, information, analyses, and so forth for that. I don't remember who was the principal commentor on that, but I think that I've captured the thrust of the comment.

Those would be my summary comments with the one correction that Mike (Voegele) brought to mind. Aside from those, I would just like to thank everyone who took the time to come and present, to discuss, to engage in the dialogue. I think it's been a very important workshop certainly from the Center's perspective. I think I can say that that's the case from NRC's perspective as well.

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10. SUPPLEMENTARY NOTES

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11. ABSTRACT (200 words or less)

The Center for Nuclear Waste Regulatory Analyses organized and hosted a workshop on "Rock Mechanics Issues in Repository Design and Performance Assessment" on behalf its sponsor the U.S. Nuclear Regulatory Commission (NRC). This workshop was held on September 19-20, 1994 at the Holiday Inn Crowne Plaza, Rockville, Maryland. The objectives of the workshop were to stimulate exchange of technical information among parties actively investigating rock mechanics issues relevant to the proposed high-level waste repository at Yucca Mountain and identify/confirm rock mechanics issues important to repository design and performance assessment. The workshop contained three technical sessions and two panel discussions. The participants included technical and research staffs representing the NRC and the Department of Energy and their contractors, as well as researchers from the academic, commercial, and international technical communities. These proceedings include most of the technical papers presented in the technical sessions and the transcripts for the two panel discussions.

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