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INITIAL FIELD TESTING DEFINITION OF
SUBSURFACE SEALING AND BACKFILLING TESTS
IN UNSATURATED TUFF

by

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ABSTRACT

This report contains an initial definition of the field tests proposed for the Yucca Mountain Project repository sealing program. The tests are intended to resolve various performance and emplacement concerns. Examples of concerns to be addressed include achieving selected hydrologic and structural requirements for seals, removing portions of the shaft liner, excavating keyways, emplacing cementitious and earthen seals, reducing the impact of fines on the hydraulic conductivity of fractures, efficient grouting of fracture zones, sealing of exploratory boreholes, and controlling the flow of water by using engineered designs. Ten discrete tests are proposed to address these and other concerns. These tests are divided into two groups: seal component tests and performance confirmation tests. The seal component tests are the small-scale in situ tests, the intermediate-scale borehole seal tests, the fracture grouting tests, the surface backfill tests, and the grouted rock mass tests. The seal system tests are the seepage control tests, the backfill tests, the bulkhead test in the Calico Hills unit, the large-scale shaft seal and shaft fill tests, and the remote borehole sealing tests. The tests are proposed to be performed in six discrete areas, including welded and nonwelded environments, primarily located outside the potential repository area. The final selection of sealing tests will depend on the nature of the geologic and hydrologic conditions encountered during the development of the Exploratory Studies Facility and detailed numerical analyses. Tests are likely to be performed both before and after License Application.

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EXECUTIVE SUMMARY

INTRODUCTION

Sandia National Laboratories, a participant in the Yucca Mountain Site Characterization Project, is responsible for implementing the repository sealing program. One aspect of this program is the definition and fielding of tests to evaluate the performance of sealing components. For planning purposes, this report presents an initial definition of the field testing program. These tests are intended to resolve various performance and emplacement concerns in repository sealing. Fernandez et al. (1987) identified sealing components to include shaft, ramp, and borehole seals and associated fill; drift fill and seals; and the Topopah Spring Member itself. The need for and suggested performance of these components depend on the geomechanical and geohydrologic conditions encountered at the site, the design of the repository, and the constraints imposed by other engineered systems. Despite uncertainties in these areas, a variety of field tests are planned to fit a range of hydrologic conditions and design constraints that may occur.

PURPOSE AND APPROACH FOR FIELD TESTING

The purpose of the current field testing program is to address regulatory issues and reduce uncertainties associated with the performance and emplacement issues for sealing components. The primary regulatory requirements, Title 10, Code of Federal Regulations, Part 60 (10 CFR 60) (U.S. Nuclear Regulatory Commission [NRC], 1986a), provide a basis for testing sealing components. The sealing program requirements fall into three areas: seal component testing (both performance confirmation and design), performance criteria for the complete repository system, and general criteria for the seals. The regulatory requirements pertinent to testing are given below.

The guidance for performance confirmation testing includes the following:

- The performance confirmation program shall provide data that indicate whether the natural and engineered systems and components required for repository

operation, or that are designed or assumed to operate as barriers after permanent closure, are functioning as intended and anticipated.

- The program shall start during site characterization and continue until permanent closure.
- The program shall include in situ monitoring, laboratory and field testing, and in situ experiments, as appropriate to accomplish the above objectives.

The design testing section states the following:

- During the early or developmental stages of construction, a program for in situ testing of features such as borehole and shaft seals, backfill, rock, and the thermal interaction effects of the groundwater shall be conducted.
- The testing shall be initiated as early as practicable.
- A backfill test section shall be constructed to test the effectiveness of backfill placement and compaction procedures against design requirements before beginning permanent backfill placement.
- Test sections shall be established to test the effectiveness of borehole and shaft seals before proceeding to full-scale operation of sealing the repository.

The NRC gives further guidance regarding the requirements for such testing in a generic technical position for in situ testing (NRC, 1985). Testing, to be conducted in the host-rock medium on engineered components, will start during site characterization and include in situ monitoring, laboratory and field testing, and in situ experiments, as appropriate. Prior to License Application (LA), the U.S. Department of Energy (DOE) should be able to use preliminary results of in situ tests to verify that the key design measures can be met.

In the Site Characterization Plan (SCP), the DOE presents the primary issue (Issue 1.12) addressing sealing components and a process to determine field tests. Issue 1.12 asks "Have the characteristics and configurations of the shaft and borehole seals [also other sealing components in ramps and the underground facility] been adequately established to (a) show compliance with the postclosure design criteria of 10 CFR 60.134 and (b) provide information for the resolution of the performance issues?"

The four-step process to determine the appropriate field tests for resolution of this issue follows. The first step is an evaluation of the information needed to resolve performance, design, and emplacement issues. The second step is an evaluation of data adequacy needed to show compliance. Specifically, is the information currently available at an acceptable confidence level? This step requires an understanding of the geohydrologic conditions encountered at the site, the thermal and mechanical conditions at seal locations, and the performance required from the sealing subsystem. As information becomes available from various SCP activities, the information will be integrated into seal design and the necessity for seal components will be reassessed. The third step, the primary objective of this report, is definition, evaluation, and selection of in situ tests of sealing components. Note that in proposing tests in this initial field testing definition, the program maintains flexibility to respond to performance and emplacement issues. The fourth step is the detailed design of specific field tests by performing detailed numerical analyses and preparing work plans to evaluate the performance of seal components.

Steps 1 and 2 were completed in previous SCP activities and, in part, in this report. This report presents the third step and provides preliminary definition of field tests for planning purposes. The project fully expects that the need for and detail of the sealing tests will be refined as the project obtains information on the characteristics of the site and as the required performance of sealing components is modified as a result of a better understanding of the site.

INFORMATION NEEDED TO REDUCE UNCERTAINTIES

As mentioned above, field testing reduces uncertainties associated with the performance and emplacement of sealing components. This report first summarizes the uncertainties in the sealing components performance and the uncertainties associated with the sealing components emplacement, focusing on the tests and facilities that may be needed to reduce the uncertainties. This report then correlates each uncertainty with one or more field tests to reduce the uncertainty. Ten field tests are defined below.

DESCRIPTION OF SEALING TESTS

A broad range of testing is proposed to accommodate hydrological conditions that may be encountered in the potential underground workings. The philosophy proposed in this report is one of progressive component evaluation. The intent is to understand the basic performance of sealing components initially and then apply this understanding to the subsequent, more complex tests. As the site information becomes available, the field testing concepts may change. However, until this information is available, two categories of tests are proposed--seal component tests and performance confirmation tests. It is recommended that, at minimum, a fundamental understanding of seal component performance is obtained prior to LA. In general, this would suggest that the seal component tests should be completed prior to LA. Seal component tests will evaluate specific structural or hydrological issues and emplacement issues associated with seal installation. Performance confirmation tests will evaluate the overall performance of the seal system in a specific geologic environment.

Seal component tests include the:

- Small-scale in situ tests,
- Intermediate-scale borehole seal tests,
- Fracture grouting tests,
- Surface backfill tests, and
- Grouted rock mass tests.

Performance confirmation tests include the:

- Seepage control tests,
- Backfill tests,
- Bulkhead test in the Calico Hills unit,
- Large-scale shaft seal and shaft fill tests, and
- Remote borehole sealing tests.

The repository sealing program recommends six general locations for conducting these ten currently proposed tests. The first location is within the densely welded portion of the Tiva Canyon Member and the second is within the nonwelded Paintbrush tuff. Most tests can be performed in welded and nonwelded tuff in these two areas. Both locations are currently planned in the north-ramp access to the Exploratory Studies Facility. The third test area includes a drift and series of alcoves within the Calico Hills unit for the backfill tests. The fourth test area is also a drift in the Calico Hills for the underground bulkhead test (if necessary). The fifth test area is an alcove within the Topopah Spring main test area for performing the grouted rock mass tests. The sixth test area would be a drift ~25 m long in the Topopah Spring main test level or in a drift off the Calico Hills access ramp. These tests will be coordinated with other project participants to increase the data acquisition efficiency and to decrease the data acquisition costs.

Common objectives for all tests include the following:

- Demonstrate placement of sealing components using current technologies.
- Determine effectiveness of testing instrumentation (i.e., range, sensitivity, and accuracy).
- Develop quality-control procedures for the placement of permanent sealing components.
- Establish seal system reliability under anticipated and unanticipated conditions.

In describing each of these tests, this report presents test layouts, important test parameters, performance objectives, and supporting analyses. The descriptions include an initial definition of instrumentation, areas of emplacement, and the number of output channels for each test. Selected scoping calculations that support the development of the field testing concepts are also presented in the supporting appendices. These calculations include hydrologic flow through seals, thermomechanical analysis of a grouted zone, hydration analyses of cementitious seals, water flow within a backfilled drift, and water flow below a backfilled drift.

This report also describes the fielding of each experiment, miscellaneous requirements for laboratory testing, and general safety and quality assurance requirements for testing. Seal component tests and performance confirmation tests are discussed briefly below.

The small-scale in situ tests evaluate the thermal and stress response of two different-diameter cementitious seals in welded and nonwelded tuff from the time immediately following mixing through complete hardening. The main test objective is to evaluate the structural performance at the interface zone dependent upon the moisture and thermal environments during cement hydration. The first tests would examine measurements of a 0.46-m-diameter seal instrumented for temperature, interface stress, and the interface circumferential and radial strains. Additional temperature and stress instrumentation could be placed in the surrounding rock. The tests would be conducted on (1) a plug of equal length and diameter and (2) a plug with the length equal to twice the diameter. A second series of tests would examine the development of interface stress for larger (0.91-m-diameter) seals.

Instrumentation also can be placed near the seal center, where the largest confinement occurs, and near the ends of the seal, where the host rock formation provides less constraint.

The intermediate-scale borehole seal tests examine the hydrologic performance of borehole seals in welded and nonwelded tuff. The main test objectives are to characterize the geologic and hydrologic properties of the rock mass at sealing locations, to emplace seals, and to test each seal for hydrologic performance, using various direct, geophysical, and hydrological methods. Because the performance of the seals would be evaluated in situ, it is necessary to characterize each type of host rock prior to and following seal placement by direct and geophysical methods. Two different types of flow tests are considered: steady state and transient pulse. The transient-pulse technique would be used if the hydraulic conductivity of the seal is extremely low. To allow access to both sides of the borehole seal, the testing could be performed between two drifts in the seal component testing facility.

The fracture grouting tests are intended to assess the ability of grout to penetrate fractures and reduce the overall conductivity of the fractured rock. The main objectives of fracture grouting tests are to emplace grout to achieve a reduction in fluid flow through selected geologic media and to assess a strengthening of the rock mass. These tests are planned primarily for the fractured, densely welded portion of the Tiva Canyon and Topopah Spring Members. Because the intent of these tests is to assess the effectiveness of the grouting operation in various geologic

settings, several material and grouting techniques may be necessary. For example, because fracture apertures vary, several grouts may be required. The performance of the grout emplacement could be evaluated by dry-drilling primary and secondary grout holes for examination, determining in situ permeability, injecting grout, drilling grout from holes, and overcoring grout holes to determine the extent of the grout. Additionally, single and/or multiple grouted rows could be tested to understand the effectiveness of various grouting designs.

The project proposes surface backfill tests to obtain a preliminary understanding of the performance of the rockfill at the surface prior to placement of rockfill underground using different emplacement methods. The test would provide an understanding of particle breakage during excavation. A basic understanding of the relationship between the emplacement technique and the quality of the rockfill can be obtained using full-scale equipment. The benefit of conducting operations at the surface is that these tests could serve as prototype tests prior to fielding complex tests underground. Other issues, such as instrumentation suitability and hydrologic performance of the rockfill, also could be addressed.

The grouted rock mass tests include testing on an isolated block of tuff instrumented with various stress, strain, temperature, and deformation gages to evaluate the thermomechanical and hydrologic properties of a typical planar joint. The tests simulate loadings using four flatjacks. The main test objective is to evaluate the thermomechanical and hydrologic properties of a typical joint in welded tuff. The properties of the joint would be evaluated under grouted and ungrouted conditions at ambient and elevated temperatures. Hydrologic performance of the joint also would be observed during both phases of these tests. The tests are designed to control stress- and temperature-boundary conditions for the rock block.

The seepage control tests are intended to address several objectives. These objectives include understanding the hydrologic performance of drift fill with drainage through underlying rock, fines migration, and the potential clogging of underlying fractures. The tests would be conducted in fractured, densely welded tuff and would evaluate the effectiveness of a filter or single embankment in restricting the lateral migration of flow in a drift. The tests would simulate both anticipated and unanticipated conditions by introducing varying quantities of water into the

test area. This pulse of water into the drift backfill could modify the as-emplaced condition of the rockfill by dislodging the fine-sized particles in the drift fill. The effectiveness of the rock in draining water also would be assessed. The proposed tests could be performed in one drift located ~20 m above a second drift. The extent of rock saturation beneath the upper drift could be assessed from the lower drift. This assessment of rock drainage requires characterization of the rock. Therefore, prior to introducing any water into the test area, the rock mass beneath the test section would be hydrologically characterized, using suitable direct or geophysical techniques such as surface resistivity, borehole induction log, ground-penetrating radar, and cross-borehole seismic tomography.

The backfill tests are intended to determine the thermomechanical and hydrologic performance of various rockfill material. These tests would be an extension of the surface backfill test and would use the engineered backfill specifications determined in the surface phase. The thermomechanical response of the backfill would be evaluated to determine the coupled response of the rockfill and the rock under simulated thermal loads. Heaters can be emplaced around the drift to simulate both far-field and near-field thermal loads. The displacement of the rock and the rockfill includes the response of water on the compaction behavior of the rockfill.

The bulkhead test would be performed on a rigid bulkhead in the Calico Hills unit. The primary test objective is to understand the hydrologic and structural aspects of a full-scale bulkhead test if such seals become necessary. Uncertainties associated with the emplacement of the large-scale bulkhead can be addressed. This performance confirmation test would show that cementitious material can be emplaced to obtain low conductivity and that seal construction activities control thermal effects of hydration. Grouting of the modified permeability zone (MPZ) and the interface zone also would be proven. Water pressures would be allowed to develop naturally behind the bulkhead, inducing flow through the bulkhead, the interface zone, and the surrounding MPZ. As in the small-scale in situ tests, the stresses, strains, and temperatures would be monitored within the bulkhead and at the interface zone.

The large-scale shaft seal and shaft fill tests involve the placement of a concrete seal and rockfill in simulated shafts. The test objectives are similar to those of the Calico Hills bulkhead

test. The seal would be emplaced using a method similar to that of the final sealing procedure of the shaft. A shaft would be developed between two drifts separated by 15 to 20 m and the shaft would be lined. Following an appropriate time for the liner to cure, shaft fill would be placed and hydrologic tests conducted on the shaft fill material. The instrumentation would involve placement of settlement gages, earth pressure cells, tensiometers, and pore pressure cells. The intent of this instrumentation is to assess the coupled settlement and hydrologic properties of the rockfill. Following the testing of the rockfill under simulated conditions, a selected portion of the liner would be removed, a keyway excavated, the cementitious seal placed, contact grouting performed if necessary, and hydrologic testing performed. Because it is anticipated that the hydraulic conductivity of the seal will be low, a transient-pulse technique may be necessary to find the hydraulic conductivity of the shaft seal. While conducting this test, the rockfill placed over the seal would be instrumented with settlement gages, a multiple position borehole extensometer system, earth pressure cells, tensiometers, and pore pressure cells. Instrumentation also would be placed at the interface between the rock and the seal to monitor the changes in stress, strain, and temperature as the concrete hydrates.

The remote borehole sealing tests would determine the hydrologic performance of selected seals placed in exploratory boreholes. This test would implement seal designs for an exploratory borehole with geologic characterization, borehole preparation, seal emplacement, and seal testing performed remotely from the surface. The project proposes that testing be performed in boreholes away from the potential repository boundary. The testing simulates identification of the sealing zone, casing removal, borehole-surface preparation, and emplacement of the seal by remote methods. The repository sealing program proposes three tests within a borehole: one in the Calico Hills unit, one in the Topopah Spring Member, and one in the Paintbrush nonwelded tuff. Knowledge gained from the small-scale in situ and intermediate-scale borehole seal tests would be used to direct the testing of the seals in this borehole.

SCHEDULE

The repository sealing program developed schedules considering time requirements for both pretest activities and the test performance. The pretest activities include performing

numerical analyses, developing engineering work packages, procuring instruments, performing reliability testing, and other site activities. Preliminary time durations were estimated for obtaining and evaluating test data. The tests were sequenced for progressive component evaluation.

Because of the long time durations needed to complete the testing, alternate scheduling strategies were considered in the event that the project requires reducing test durations. These scheduling strategies include:

- Splitting some testing between the pre-LA and post-LA periods. This strategy delays completion of the performance confirmation tests.
- Accelerating the testing program by performing pretest activities of some tests with test activities from other tests. Part of this strategy includes conducting the surface backfill tests in drifts.

Little reduction in time is achieved by implementing the first strategy. It does have the benefits of proceeding cautiously and increasing the likelihood of a successful field testing program. The second strategy reduces an estimated 12-year program to ~6 years. These schedules would be refined after obtaining access to the underground facility.

ACRONYM LIST

10 CFR 60	Title 10, Code of Federal Regulations, Part 60
40 CFR 191	Title 40, Code of Federal Regulations, Part 191
ACD	advanced conceptual design
ASTM	American Society for Testing and Materials
COE	U.S. Army Corps of Engineers
CPU	central processing unit
CRD-C	Concrete Research Division--Concrete
C-S-H	calcium-silicate-hydrate
CSIRO	Commonwealth Scientific and Industrial Research Organization
CSM	Colorado School of Mines
D & B	drill and blast
DOE	U.S. Department of Energy
EPC	expansive portland cement
ESF	Exploratory Studies Facility
ESFAS	Exploratory Studies Facility Alternatives Study
GPR	ground-penetrating radar
ISRM	International Society for Rock Mechanics
L/D	length-to-diameter ratio
LA	License Application
LHD	load haul dump
LL	lower level
MPBX	multiple position borehole extensometer
MPZ	modified permeability zone
NEDA	Non-Explosive Demolition Agents
NRC	U.S. Nuclear Regulatory Commission
NTS	Nevada Test Site
NUREG/CR	U.S. Nuclear Regulatory/Contractor Report
NWTRB	Nuclear Waste Technical Review Board
OPC	ordinary portland cement

PMF	Probable Maximum Flood
RTD	resistance temperature detector
SCP	Site Characterization Plan
SCTF	seal component testing facility
SNL	Sandia National Laboratories
TBD	to be determined
TBM	tunnel boring machine
UL	upper level
USBM	U.S. Bureau of Mines
w/c	water-to-cement
WIPP	Waste Isolation Pilot Plant
YMP	Yucca Mountain Site Characterization Project

1.0 INTRODUCTION

The Yucca Mountain Site Characterization Project (YMP), managed by the U.S. Department of Energy (DOE), is examining the suitability of disposing of high-level radioactive waste in a mined geologic repository at Yucca Mountain. Yucca Mountain is located on and adjacent to the Nevada Test Site (NTS), Nye County, Nevada. The potential repository would be located in an unsaturated tuff formation within Yucca Mountain.

The overall repository system is comprised of many subsystems that must be evaluated to determine the suitability (i.e., acceptable performance) of disposing of waste in Yucca Mountain. One subsystem is the sealing subsystem (Figure 1-1). The logic followed to assess the performance of the sealing subsystem is presented in the Site Characterization Plan (SCP) (DOE, 1988). A potentially important area in this logic is conducting field tests of sealing components that comprise the sealing subsystem. The purpose of these tests is to evaluate the performance of the sealing components so that compliance with regulatory requirements can be demonstrated. Sandia National Laboratories (SNL) is currently responsible for demonstrating that the performance of the sealing system complies with regulatory requirements.

This report contains preliminary definitions of field tests that may be needed to show compliance with the regulatory requirements. Figure 1-2 shows the process for resolving Issue 1-12. Part of the process involves field testing. This philosophy that incorporates field testing as part of the issue resolution process is consistent with the philosophy generally accepted by the International Atomic Energy Agency.

A more detailed description of the philosophy used to design sealing components is given in the SCP. This philosophy involves a four-step process. The first step is an evaluation of the information needed to resolve performance, design, and emplacement issues. The second step is an evaluation of the adequacy of the data needed to show compliance. Specifically, is the information needed currently available at an acceptable confidence level? The third step is definition, evaluation, and selection of in situ tests of sealing components. This step requires an understanding of the geohydrologic conditions at the site, the thermal-hydrological-mechanical

YUCCA MOUNTAIN MINED GEOLOGIC DISPOSAL SYSTEM

Figure 1-1. Sealing Subsystem, Seal Locations, and Seal Components, Modified From the Site Characterization Plan (DOE, 1988)

Figure 1-2. Logic Diagram for Resolution of Issue 1.12 (Sealing Subsystem) From the Site Characterization Plan (DOE, 1988)

conditions at the seal locations, and the performance required from the sealing subsystem. This information currently exists in the SCP. As site characterization proceeds and waste package and repository designs evolve, the details of the field tests will be refined. The fourth step is the detailed design of specific field tests to evaluate the performance of seal components. The first two steps are completed. This report initiates the third step. It is fully expected that the need for and detail of the sealing tests will be refined as more site information is obtained and the required performance of the sealing components is modified as a result of total systems performance analyses that consider the repository and waste package designs.

Although the need for and detail associated with the sealing tests would be refined as new site and design information becomes available, definition of field tests provides a focus toward resolving the sealing issues and concerns.

Chapter 2 of this report presents the regulatory requirements applicable to the sealing program, the performance and emplacement issues, and a compilation of the proposed tests that would resolve the issues and satisfy the regulatory requirements. Chapter 3 describes the techniques that can be used to characterize the rock, including geophysical and hydrological techniques. Because the proposed tests depend on rock locations of interest, potential test locations are presented. Also, available standards for determining properties of sealing material in the laboratory are specified. Instrumentation for measuring in situ conditions are also defined. Chapter 4 defines and describes the proposed tests. These tests are divided into two categories: seal component tests and performance confirmation tests. Chapter 4 presents the 10 individual groups of tests to be performed as part of the program. The presentations of the tests generally include a description, layout, fielding of the experiment, and miscellaneous requirements. For particular tests with material selection or emplacement techniques as significant issues, these concerns are also discussed. Chapter 5 presents the safety and quality assurance considerations of conducting successful tests. Finally, Chapter 6 presents the schedule for performing the field tests.

2.0 BASIS FOR TESTING

The basis for performing seal component testing is the focus of this chapter. Integral to the following discussion of the basis for testing, as well as the remainder of the report, is the description of the potential repository and the sealing concepts. Therefore, a background section (Section 2.1) is included here.

The basis for performing seal component testing is presented in two parts: regulatory requirements (Section 2.2) and technical requirements (Section 2.3). The regulatory requirements are derived primarily from Title 10, Code of Federal Regulations, Part 60 (10 CFR 60) (U.S. Nuclear Regulatory Commission [NRC], 1986a). The technical requirements are defined by the uncertainties associated with seal performance and seal emplacement.

2.1 Background

The following background information is presented in this section: Section 2.1.1 presents a description of the Exploratory Studies Facility (ESF) layout and repository access, Section 2.1.2 presents the sealing concepts, and Section 2.1.3 presents the general technical information needed to support the sealing program.

2.1.1 Exploratory Studies Facility Layout and Repository Access

In the SCP-Conceptual Design Report (MacDougall, 1987), a conceptual repository design was presented. In response to an objection raised by the NRC that consideration of multiple designs was not demonstrated, a study was conducted to evaluate systematically various ESF and repository configurations and construction methods, together called options. The Exploratory Studies Facility Alternatives Study (ESFAS) evaluated 34 options for relative desirability considering a wide range of criteria (Dennis, 1991). Option 30 received the highest ranking. It included two ramps accessing the repository from the east. From each of these ramps, a second ramp was recommended to gain access to the Calico Hills unit below the repository. At both the

Topopah Spring unit (repository level) and the Calico Hills unit, additional drifting would be performed to characterize discrete structural features (see Figure 2-1).

Details of the design were not developed as part of the ESFAS because originally only five principal design features were varied from option to option: (1) means of access, (2) location of access, (3) location of the main test level, (4) excavation methods used, and (5) total number of ESF-repository accesses. This option will be modified in accordance with the design control process. Selected key features that may be considered for change will be subject to engineering trade-off studies during the design phase. The selected option as modified and documented with the Title I design report represents the basis for the locations and types of sealing tests that are proposed.

2.1.2 Sealing Concepts

The primary objective of the sealing subsystem is to contribute to the isolation of radioactive waste. This objective is achieved through the following actions:

- Reducing the amount of water entering the underground facility through vertical shafts, ramps, or other vertical or horizontal penetrations.
- If water enters the vicinity of the waste package, controlling the flow of water away from the waste package and draining it into nonemplacement areas.
- Controlling the release of radionuclides from the repository so that the shaft, ramp, and borehole seals do not become preferred pathways.
- Preserving the structural integrity of the host medium.

Sealing components have been proposed that achieve the primary objective through one of the four means described. In addition to the placement of manmade sealing components, an additional strategy proposed in the sealing program is to isolate and drain water encountered in the repository and at the base of the shafts. Drainage can be accomplished through the highly fractured, densely welded tuff of the Topopah Spring unit.

Note: This is pictorial only and not drawn to scale

TS - Topopah Spring
CH - Calico Hills

Figure 2-1. Modified Option 30 Design Configuration (Dennis, 1991)

In the SCP (DOE, 1988), tentative sealing components were identified. They were placed into one of two subsystems: the shaft and borehole seals subsystem or the repository seals and backfill subsystem. Figure 1-1 identifies the subsystem, the seal location, and the seal component. Sealing components in shafts, ramps, and boreholes are intended to achieve the primary objective by accomplishing the first and third actions given above. Components associated with the shafts include the following:

- A capillary barrier at the surface to restrict surface water from entering the upper portion of the shaft.
- Discrete seals to restrict flow downward into the shaft, to restrict airflow upward, and to reduce settlement effects that could propagate to the surface.
- Shaft and borehole fill to restrict water flow downward and airflow upward.
- The highly fractured, densely welded tuff at the base of the shaft to isolate and drain water in the shaft.

The primary component in the ramps will be the rockfill placed in the majority of the ramp. Periodically, structures such as a dam with a lower hydraulic conductivity than the material upgradient from it can be placed in the ramp to restrict the lateral migration of water flow in the ramps. Drainage can be enhanced by such simple techniques as drilling multiple boreholes in floor areas with a high drainage potential. In exploratory boreholes, seals accomplishing a capillary barrier or fill restricting water flow downward and airflow upward are proposed. While the entire borehole will be sealed, there may be preferred locations to place seals that will accomplish the objective better. Preferred areas may be competent zones within the Paintbrush nonwelded tuff or the upper portions of the Topopah Spring Member, which are not highly fractured.

Within the underground facility sealing subsystem, multiple components are proposed to control various quantities of water inflow. The proposed components would control expected small inflows. More elaborate sealing components, such as the double bulkhead, are proposed in the unlikely event that extremely large inflows occur. These bulkheads are intended to preserve the structural integrity of the host rock. The more probable sealing components that would be

emplaced in the repository are simple: a backfilled channel, the drift fill, and a simple dam constructed by placing low-conductivity material adjacent to higher-conductivity material.

In addition to these sealing components, the design of the repository facility itself could also enhance the repository performance (Fernandez et al., 1987). For example, the grade of the drifts could be constructed so that water in emplacement drifts is redirected into nonemplacement drifts where further drainage into other nonemplacement drifts would not be possible due to their relative grade. Another example is designing the repository with multiple panels so that if isolation from an unusually high water inflow was required, only a small portion of the repository would be affected. Although it is not the intent of this report to present repository layouts that could contribute to performance, the design of the repository, as stated earlier, is important to defining suitable sealing tests. By inference, the design of the repository could reduce the importance of the sealing subsystem in enhancing the overall repository performance or, conversely, could increase the significance of the sealing subsystem if design features are not properly considered.

2.1.3 General Technical Information Needs for the Sealing Program

This section evaluates the general technical information needed to resolve the sealing issues identified in the SCP. Issue 1.12 asks "Have the characteristics and configuration of the shaft and borehole seals been adequately established to (a) show compliance with the postclosure design criterion of 10 CFR 60.134 and (b) provide information for the resolution of the performance issues?" (DOE, 1988). Specific seal performance and seal emplacement information needs, which provide the technical basis for performing sealing tests, are presented in Section 2.3.

In the following discussion, all SCP activities that contribute to the resolution of Issue 1.12 (Figure 1-2) are considered because the sealing program depends upon diverse activities of the repository program to provide the information required to assess the need for sealing and to develop design requirements. Multiple organizations will be responsible for generating data defined in the SCP. To guide other YMP participants, a correlation is made in Appendix A

between the information needed to resolve performance/design and emplacement issues and the study plans that will obtain the information.

The information needs summarized in the tables in Appendix A fall into four major categories. Other activities of the repository program will provide the required data in these categories. These categories include:

- Hydrologic-related site parameters, which are required to estimate the hydrologic influx to the facility and the attendant need for sealing.
- Miscellaneous information related to rock properties, groundwater geochemistry, and shaft fill properties.
- Additional requirements related to hydrologic and rock properties.
- Waste package performance, which describes the physical integrity and chemical stability of the waste package and fuel rods in the presence of the groundwater.

The hydrologic-related site parameters and miscellaneous information categories are derived directly from the SCP, whereas the remaining two categories, additional requirements and waste package performance, are derived from the design assumptions and recommendations made in the technical basis sealing report (Fernandez et al., 1987). The information needs were developed to:

- Determine the allowable amount of water that may reach the waste packages by considering the effects of the water upon the waste packages and contained fuel rods.
- Determine the amount of water that might contact the waste under anticipated and unanticipated conditions.
- Establish performance goals through the adoption of a design approach.
- Establish additional information through the design process.

For each category, the performance or design parameters, their modifiers, and tentative parameter goals are presented in Appendix A. Pertinent sections of the SCP and the title of the

SCP activity are identified, and a series of notes for the specific SCP sections is presented in Appendix A.

2.2 Regulatory Requirements

The primary regulatory requirements that provide a basis for testing sealing components are contained in 10 CFR 60 (NRC, 1986a). The requirements of interest for the sealing program fall into three areas: design testing guidance for the seals, performance criteria for the overall repository system, and general criteria for the seals. The first area of interest deals directly with testing sealing components and is included below.

Guidance on design testing of sealing components is contained in Subpart F, Performance Confirmation Program, of 10 CFR 60. Guidance relevant to seal and backfill testing is provided under the general guidance section, Section 60.140 (a), and the design testing section, Section 60.142. Performance confirmation "means the program of tests, experiments, and analyses which is conducted to evaluate the accuracy and adequacy of the information used to determine with reasonable assurance that the performance objectives for the period after permanent closure will be met."

In the general guidance for the performance confirmation program [10 CFR 60.140(a)], it is stated that:

- The performance confirmation program shall provide data that indicate whether the natural and engineered systems and components required for repository operation, or which are designed or assumed to operate as barriers after permanent closure, are functioning as intended and anticipated.
- The program shall have been started during site characterization and it will continue until permanent closure.
- The program shall include in situ monitoring, laboratory and field testing, and in situ experiments, as may be appropriate to accomplish the objective stated above.

In the design testing section (10 CFR 60.142), it is stated that:

- During the early or developmental stages of construction, a program for in situ testing of such features as borehole and shaft seals, backfill, rock, and the thermal interaction effects of the groundwater shall be conducted.
- The testing shall be initiated as early as practicable.
- A backfill test section shall be constructed to test the effectiveness of backfill placement and compaction procedures against design requirements before permanent backfill placement is begun.
- Test sections shall be established to test the effectiveness of borehole and shaft seals before full-scale operation proceeds to seal boreholes and shafts.

The NRC gives further guidance regarding the requirements for in situ testing and performance confirmation testing in a generic technical position paper on in situ testing (NRC, 1985). In situ testing refers to testing conducted in an underground test facility for purposes of site characterization. In situ tests include geological-geophysical, hydrological, geomechanical, geochemical, and thermal tests. These tests would be conducted on the host rock medium and the engineered components, including backfill and seals. Also, another generic technical position paper on borehole and shaft sealing (NRC, 1986b) states that such testing will start during site characterization and will include in situ monitoring, laboratory and field testing, and in situ experiments as may be appropriate. The NRC (1986b) further stated that " . . . prior to License Application (LA), the DOE should have the capability of verifying, with preliminary results of in situ tests, that the key design measures discussed in this position paper can be met." Alternatively, the DOE may decide to modify the design based on the results of the performance confirmation tests. The sealing program should establish (1) the effectiveness of emplacement equipment and methods; (2) the effectiveness of testing instrumentation; (3) the effectiveness of the seal subsystems; (4) the reliability of the seal subsystems under anticipated and extreme conditions; and (5) the basis for the final design of seal components.

For the second and third areas of interest defined by the NRC, regulations are proposed that address the performance of the sealing system. These are presented below for completeness. The primary performance criterion (10 CFR 60.112) in the second area of interest is that:

The geologic setting shall be selected and the engineered barrier system and the shafts, boreholes and their seals shall be designed to assure that releases of radioactive material to the accessible environment following permanent closure conform to such generally applicable environmental standards for radioactivity as may have been established by the Environmental Protection Agency with respect to the anticipated processes and events and unanticipated processes and events.

The standards given in Title 40, Code of Federal Regulations, Part 191 (40 CFR 191) (NRC, 1986c) are applicable for the entire repository system. Thus, no criteria specific to the sealing subsystem are given.

General criteria for the design of seals for shafts and boreholes are included in the third area. These criteria (10 CFR 60.134) are listed below.

- General design criterion--Seals for shafts and boreholes shall be designed so that following permanent closure they do not become pathways that compromise the geologic repository's ability to meet the performance objectives [for] the period following permanent closure.
- Selection of material and placement methods--Material and placement methods for seals shall be selected to reduce, to the extent practicable, (1) the potential for creating a preferential pathway for groundwater to contact the waste packages or (2) for radionuclide migration through existing pathways.

The second and third areas of performance confirmation defined by the NRC address the performance of the sealing subsystem within the overall repository system. As site information becomes available and repository and waste package design is refined, the performance requirements for the sealing system would evolve. This evolution of requirements would likely require changes in the in situ testing program for the sealing components defined in this report.

2.3 Technical Basis--Seal Performance and Seal Emplacement Uncertainties

Performance goals and emplacement concerns are identified for sealing components in both the SCP and Fernandez et al. (1987). Sealing components include an anchor-to-bedrock plug, single dams (or single bulkheads with no settlement), general shaft fill, drift backfill, station and

shaft plugs, double bulkheads, backfilled sumps, and channels in a backfilled room. The performance goals and emplacement concerns were formulated into questions (Table 2-1, first column) that represent uncertainties in the sealing program. Several questions were asked about each uncertainty.

- What actions are needed to reduce the uncertainties?
- What approach should be adopted to reduce the uncertainty?
- What type of facility is needed to reduce the uncertainty?

Table 2-1 summarizes the answers to these questions. This table, therefore, provides the basis for the field testing program. The basis for the laboratory testing is not presented in this table.

Emplacement properties of interest that can be determined through the laboratory analyses include the following:

- For earthen material (clays and rockfill)
 - Density-water content relation.
 - Particle gradation.
 - Hydraulic conductivity related to density.
 - Swelling capacity related to density.
- For cementitious material
 - Density.
 - Heat of hydration.
 - Slump and viscosity.
 - Set time and working time.
 - Maximum particle size of aggregate for grouting.
 - Hydraulic conductivity.

2.4 Proposed Tests, Test Objectives, and Linkage With Sealing Uncertainties

A number of tests are proposed over the range of expected conditions for specific sealing locations in the underground workings. The sealing tests are summarized in Table 2-2. This

Table 2-1. Correlation Between Seal Performance and Emplacement Uncertainties With Proposed Tests and Facilities

Uncertainty	Proposed Actions	Approach Used Including Proposed Test	Facility Proposed
<p><u>ANCHOR-TO-BEDROCK PLUG</u></p> <p>Can a plug be placed with specific properties to form an effective hydrologic barrier (tentative goal of 10^{-5} to 10^{-4} cm/s saturated hydraulic conductivity)? [1]</p>	<p>Construct a shaft.</p> <p>Construct a plug.</p> <p>Field test the plug hydrologically.</p> <p>Core into the plug.</p> <p>Laboratory test of core to determine hydrologic properties.</p>	<p>Assess the geohydrologic properties of the site prior to conducting the plug test. Use a progressive approach to test a cementitious seal by understanding the thermomechanical response of a cementitious seal in situ and the hydrological performance of the seal through</p> <ul style="list-style-type: none"> • Small-scale borehole tests • Borehole seal test between drifts • Large-scale plug test. <p>For the large-scale tests, the most effective manner to test the MPZ and the interface zone may be to core into those zones. Also, perform hydrological test of plug core after testing and supporting laboratory tests of concrete before placement.</p>	<p>Parallel drifts - one above the other in Tiva Canyon welded tuff.</p> <p>Propose</p> <ul style="list-style-type: none"> • Small borehole test • Borehole between drifts • Large-scale shaft(s).
<p>Can a plug having specified structural design requirements be constructed? [2]</p>	<p>Construct a cementitious seal.</p> <p>Core into plug.</p> <p>Laboratory testing of core after emplacement.</p>	<p>Following proper setting of the plug, core into the plug to test the strength of the samples. Samples should be of various sizes to assess the effect of sample size. Consider the testing of the concrete nondestructively or by hydrofracturing over a portion of the plug.</p>	<p>Large-scale shaft(s) in Tiva Canyon welded tuff.</p>
<p>Can the liner be removed before plug placement without propagating the modified permeability zone (MPZ) outward? [3]</p>	<p>Characterize the MPZ.</p> <p>Emplace simulated liner.</p> <p>Remove liner.</p> <p>Recharacterize the MPZ.</p>	<p>First, characterize the MPZ for the shaft nondestructively. In testing plugs in shafts, the liners will be removed and a keyway constructed. Removal will use drill and blast (D & B) and/or the hydraulic splitter technique.</p>	<p>Large-scale shaft(s) in Tiva Canyon welded tuff.</p>
<p>Can a keyway be excavated before plug construction? [4]</p>	<p>Construct shaft.</p> <p>Emplace liner.</p> <p>Remove liner.</p> <p>Excavate keyway.</p>	<p>Resolution of this uncertainty is obtained in conjunction with removal of the liner.</p>	<p>Large-scale shaft(s) in Tiva Canyon welded tuff.</p>
<p>Can grout be placed effectively by the selection of a suitable grout pattern, the use of grout pipes, and the selection of grout injection pressures? [5]</p>	<p>Select various fracture settings to include welded tuff in the Tiva Canyon unit.</p> <p>Drill grout holes.</p> <p>Characterize fractures.</p> <p>Grout.</p> <p>Test effectiveness of grout operation.</p>	<p>In various fracture settings, grout holes having various patterns should be drilled. The rock is characterized hydrologically. The grout is placed and subsequently drilled out. The hole is then characterized again. The conditions encountered while constructing the ramps will determine where the testing should be performed. At least three environments should be selected. The approach of stage grouting should be implemented and coring into the rock mass to determine the extent of the grout curtain should be performed. Colored grouts can be used for each stage.</p>	<p>Alcoves or drifts in the Tiva Canyon welded tuff.</p>

Table 2-1. Correlation Between Seal Performance and Emplacement Uncertainties With Proposed Tests and Facilities (Continued)

Uncertainty	Proposed Actions	Approach Used Including Proposed Test	Facility Proposed
<p>Can grout workability be assured during placement in the in situ moisture and temperature environment? [6]</p> <p>Can shrinkage and temperature effects due to hydration significantly alter the interface zone? [7]</p>	<p>Measure the workability of grout at the point of placement.</p> <p>Place a cementitious plug.</p> <p>Core into the interface zone.</p> <p>Examine the quality of the interface zone.</p>	<p>Testing of the viscosity at the point of grout placement and laboratory testing of the same property of grout is required. This testing is intended to be a quality-control test.</p> <p>A basic understanding of shrinkage and emplacement techniques will be required. This understanding will be achieved by laboratory and bench-scale testing. The influence of the heat of hydration and the placement technique should be understood before going into the field. Field tests are intended to be confirmatory. For example, after testing of cementitious seals in the following tests, cores along the interface zone can be obtained:</p> <ul style="list-style-type: none"> • Small-scale in situ seal tests • Large-scale shaft seal test. 	<p>No facility is required.</p> <p>Alcoves or drifts in welded tuff.</p>
<p>Can concrete workability be assured during placement to eliminate either premature settling or particle segregation? [8]</p> <p><u>GENERAL SHAFT BACKFILL</u></p> <p>Can the shaft fill be emplaced by either mechanical, pneumatic, or other methods to form an effective hydrologic barrier (tentative goal of 10^{-2} cm/s saturated hydraulic conductivity)? [9]</p>	<p>Measure the workability at the point of emplacement.</p> <p>Construct a shaft.</p> <p>Emplace a liner.</p> <p>Emplace backfill.</p> <p>Conduct a test.</p> <p>Characterize fill after test.</p>	<p>To address this uncertainty, field and laboratory testing of the workability of the concrete would be required. The important concern is maintenance of the properties of the concrete while it is being transported to the point of emplacement. Transport of concrete through a conduit in the underground facility can be performed to simulate the distance of transport at the surface.</p> <p>Instrument the interface zone to measure the stress state in the rock and with the seal itself.</p> <p>There are two basic performance requirements that the shaft fill should achieve. These include the water flow and air flow tests. The following tests are proposed:</p> <ul style="list-style-type: none"> • Laboratory testing to demonstrate the basic physics of flow. (The starting point will be definition of the relationship of gradation to hydraulic conductivity. Understanding the relationship between conductivity and consolidation is necessary.) • Field testing in large-scale shaft(s) to illustrate the ability to emplace backfill at some predetermined state. The testing would first be run using air and then water migrating through the fill. <p>Several different gradations will be used to achieve the desired properties. Furthermore, mixtures of clay and crushed tuff may be tested.</p>	<p>No facility required.</p> <p>Shaft(s) for fielding multiple experiments.</p>

Table 2-1. Correlation Between Seal Performance and Emplacement Uncertainties With Proposed Tests and Facilities (Continued)

Uncertainty	Proposed Actions	Approach Used Including Proposed Test	Facility Proposed
Can an earthen material be placed by mechanical, pneumatic, or other methods to achieve specified properties (tentative goal of 1 to 2% of the effective shaft length of the shaft fill)? [10]	Construct shaft. Emplace a liner. Emplace backfill. Test shaft fill. Monitor shaft fill behavior. Extract samples.	Evaluation of the consolidation of crushed tuff can be performed in multiple ways. The following are the options: <ul style="list-style-type: none"> • Simulation of a shaft segment in the drift floor • Simulation in a large-scale shaft • Simulation of a shaft at the surface. <p>The field testing will simulate the compaction of crushed tuff using actual emplacement techniques. This material will then be extracted and laboratory tests performed. All work is preceded by laboratory testing.</p>	Large-scale shaft in welded or nonwelded tuff. Surface facility to test compaction.
What is the effect of emplacement techniques on the creation of fines? [11]	Same as above.	To simulate the response of the backfill to the operations, it will be necessary to place the backfill under conditions closely simulating the actual emplacement conditions. A facility at the surface that uses realistic techniques is proposed to understand the actual compaction state. These tests also could be performed in large-scale shaft(s). An understanding of the kind of fines created is important when determining the actual gradation for the shaft fill. It will be necessary to determine an acceptable consolidation first, based on literature surveys and "soil" mechanics principles. Then it will be necessary to understand how many fines are created.	Large-scale shaft. Surface facility to test fines creation.
<u>SINGLE EMBANKMENT AND DRIFT FILL</u>			
Can grout be placed effectively by the selection of a suitable grout pattern, the use of grout pipes, and the selection of grout injection pressures? [12]	Select various fracture settings to include welded tuff. (See the similar question for the anchor-to-bedrock plug.)	The proposed tests are similar to those described for the anchor-to-bedrock plug.	Alcoves or drifts in the welded portion of the Topopah Spring Member.
Can grout workability be assured during the emplacement in the in situ moisture and temperature environment? [13]	Measure the workability of grout at the point of placement.	Same as the issue described for the anchor-to-bedrock plug.	No unique facility is required.
Can a continuous grout structure be achieved by redrilling holes many times and grouting without affecting the sump drainage capacity? [14]	Select various fracture settings to include welded tuff. Emplace grout curtain. Determine extent of penetration.	The proposed tests are similar to those described for the anchor-to-bedrock plug. There is an additional concern that the orientation of the fractures should be varied to simulate the conditions of fractures parallel to the drifts and perpendicular to the drifts. Testing of the spread of the grout curtain associated with the dam is necessary as well	A test facility in welded tuff having fractures parallel and perpendicular to the orientation of the drift. Same as the uncertainty associated with the effectiveness of grout placement.

Table 2-1. Correlation Between Seal Performance and Emplacement Uncertainties With Proposed Tests and Facilities (Continued)

Uncertainty	Proposed Actions	Approach Used Including Proposed Test	Facility Proposed
Can drift backfill be placed by mechanical, pneumatic, or other methods to form an effective hydrologic barrier and to achieve specific properties? [15]	Emplace drift fill. Test the drift fill. Extract and test samples.	as the hydraulic testing of the grout curtain after grout emplacement. The holes that are used for verification of the grout extent can be used for the hydraulic conductivity testing. Viscosity and the yield strength are two of the most critical parameters for grout emplacement. Following detailed characterization of the fracture in the area of grouting, ties between their properties, the orientation of the fracture network, and the rate and depth of the penetration will be evaluated. The objective here is to demonstrate that backfill can be emplaced according to some predetermined criteria. Therefore, a large-scale test emplacing the drift fill would be necessary to verify that the criteria is met. Repetitive tests may be required to understand how the placement techniques affect the properties of the drift fill.	Large-scale drift. Surface test facility.
Can cracks due to shrinkage or thermal effects create significant water paths? [16]	Emplace a drift embankment. Conduct tests.	There are several issues associated with this uncertainty: the effect of heat on the separation of the backfill from the rock, the effect of saturation and desaturation on the consolidation properties of the backfill, and the cracking of the fill due to the same effects. Testing will be performed on a single embankment determined through analyses evaluating the potential for piping. A full-scale test, or several tests if rock conditions warrant, will then be performed to address these issues, supported by laboratory compaction tests. This test is an evaluation of the composite system. The flow through the rock, backfill, and the embankment will all be instrumented. A large-scale facility is considered necessary. A facility in welded tuff will be appropriate. Progressive testing is proposed in a surface test facility, in the Tiva Canyon unit, and finally in the Topopah Spring unit.	Large-scale drift in welded tuff. Surface test facility.
Can a single embankment retard the lateral migration of flow to 10% of the drainage capacity behind the embankment? [17]	Emplace a single embankment. Test area and monitor flow. Test area.	An understanding of fines migration is necessary to resolve this issue. Layered systems will be constructed in the drifts and subjected to water bench-scale testing. The tests will be performed to understand the migration of fines in selected fills. Filter designs will also be tested. Testing will saturate the rock below. Laboratory analyses will be performed first to refine filter criteria.	Drift in Tiva Canyon unit. Drift in the Topopah Spring unit. Surface test facility.
Can the migration of fines behind the single embankment clog the fractures? [18]	Emplace a layered fill. Flood the layered fill. Extract samples. Perform gradational analyses.		Drift in welded tuff. Surface test facility.

Table 2-1. Correlation Between Seal Performance and Emplacement Uncertainties With Proposed Tests and Facilities (Continued)

Uncertainty	Proposed Actions	Approach Used Including Proposed Test	Facility Proposed
Can fines be removed from the floor or controlled from entering the fracture system to reduce the silting potential? [19]	See similar uncertainty above.	See above. Resolution of this issue may depend in part on the ability to reexcavate the welded tuff at the floor of the drift. This is not a field test requiring an instrumented area. The resolution is achieved by removing the welded tuff. Any restoration of the floor should consider the use of Non-Explosive Demolition Agents or other techniques that do not introduce an unacceptable amount of fines.	See above.
<u>STATION AND SHAFT/RAMP PLUGS AND DOUBLE BULKHEAD-NO SETTLEMENT</u>			
Can the liner be removed prior to plug placement without propagating the MPZ outward at the plug location? [20]	See similar uncertainty above.	The following uncertainties can be resolved by looking at similar questions asked for the single embankment or the anchor-to-bedrock plug. Separate facilities are not considered necessary.	Large-scale shafts or lined ramp segments in welded and nonwelded tuff.
Can a keyway in the welded and nonwelded tuff be excavated? [21]	See similar uncertainty above.	See above. Because of the potential need to remove shaft and ramp liners in nonwelded and welded zones, it may be necessary to evaluate this uncertainty in both rock types.	Alcoves or drifts in welded and nonwelded Paintbrush tuff.
Can grout be placed effectively by the selection of a suitable grout pattern, the use of grout pipes, and the selection of grout injection pressures? [22]	See similar uncertainty above.	See similar uncertainty above.	Drifts in welded tuff.
Can grout workability be assured during placement in the in situ moisture and temperature environment? [23]	See similar uncertainty above.	See similar uncertainty above.	No facility required.
Can the material be placed with specified properties to form an effective hydrologic barrier? [24]	See similar uncertainty above.	See above. Because the station plug may be subjected to a hydrostatic head load from water accumulated in the shaft, a modification of the anchor-to-bedrock plug test may be appropriate. Shaft plugs would require study in nonwelded tuff.	Locations in welded and nonwelded tuff.
Can shrinkage and temperature effects due to hydration significantly alter the interface zone? [25]	See similar uncertainty above.	See above.	Same as the anchor-to-bedrock plug.
Can workability be assured during placement to eliminate either premature settling or particle segregation? [26]	See similar uncertainty above.	See above.	No facility required.

Table 2-1. Correlation Between Seal Performance and Emplacement Uncertainties With Proposed Tests and Facilities (Continued)

Uncertainty	Proposed Actions	Approach Used Including Proposed Test	Facility Proposed
<p><u>BACKFILLED SUMP</u></p> <p>Can fines be removed from the floor to reduce silting potential? [27]</p>	<p>Control potential by effective design and operational practices.</p>	<p>This is not a testing issue but rather an operational concern. For example, fines created during the use of the tunnel boring machine (TBM) should be removed from the face in as effective a manner as possible. The entire issue of fines clogging the fracture system and reducing the hydraulic conductivity of the rock is dependent on the fracture system itself. Prevalent fracture having a large aperture should be protected so that even sand-sized particles or larger will not enter the fracture. This concept argues in favor of a filter near the fracture itself. All fines migration problems also require the characterization of fines in flows entering the underground facility.</p>	<p>No facility required.</p>
<p><u>CHANNEL IN BACKFILLED ROOM</u></p> <p>Can backfill be placed in a channel subjected to quality-control testing of specified gradations to reduce the silting potential and achieve the performance objective? [28]</p>	<p>Laboratory and bench-scale testing.</p>	<p>Similar to the emplacement of a filter around a large fracture, the placement of drift fill should be done around the channel fill so that the channel does not become clogged. Laboratory testing using bench-scale testing should be used to satisfy this uncertainty.</p>	<p>No facility required.</p>
<p><u>EXPLORATORY BOREHOLES</u></p> <p>Can the casing be removed and a plug emplaced in an exploratory borehole at the Calico Hills level and in the Paintbrush tuff horizon? [29]</p> <p>Can a plug be installed in the Tiva Canyon, Paintbrush nonwelded, and Calico Hills units with specific conductivities? [30]</p>	<p>Remove casing. Install plug.</p> <p>Characterize borehole geology. Emplace seal. Test seal.</p>	<p>This uncertainty is strictly a feasibility question. The answer will be resolved by demonstrating the removal of casing and the installation of a plug.</p> <p>The installation of a cementitious seal can be demonstrated by testing easily accessible seals as in the small-scale in situ tests. Next, seals should be tested in boreholes drilled between drifts in welded tuff and nonwelded tuff. Finally, a surface-scale operation must be performed to illustrate that an acceptable seal can be emplaced from the surface. Part of this effort will include the placement of bentonite pellets in a highly fractured zone in the welded devitrified zone. This will necessitate small-scale testing at the repository level to effectively place bentonite pellets.</p>	<p>Any exploratory borehole extending through the Calico Hills unit. Borehole should be outside the repository area.</p> <p>Parallel drifting with boreholes, both shallow and connecting the two drifts in the Paintbrush nonwelded and Tiva Canyon. An exploratory borehole penetrating the Calico Hills unit.</p>

Table 2-1. Correlation Between Seal Performance and Emplacement Uncertainties With Proposed Tests and Facilities (Continued)

Uncertainty	Proposed Actions	Approach Used Including Proposed Test	Facility Proposed
<u>CALICO HILLS DRIFT SEAL</u> Can the drifts in the Calico Hills unit be adequately sealed? [31]	Construct a drift. Create isolated areas for testing. Seal portion of drift. Extract samples.	The current justification for testing in the Calico Hills unit is to understand the basic properties of the recompacted nonwelded tuff that is extracted from this unit. These properties will include the consolidation properties, the creation of fines, the hydraulic conductivity of the recompacted material, and the strength properties of the fill. If the fracturing is different from the units above, fracture grouting experiments may be necessary. Additionally, a second test, such as a bulkhead test, may be required to seal environments where special problems may occur.	Drifting in the Calico Hills unit.
<u>ENGINEERED DRAINAGE ENHANCEMENT</u> Can the drainage of the fractured densely welded tuff be enhanced through the use of engineered features? [32]	Map fractures in drifts. Drill exploratory boreholes. Characterize the fractures system. Test for engineering enhancement.	The purpose of this test is to characterize the impact of thermal loads on the drainage capacity of a fractured rock mass. It is further intended to experiment with several design concepts that enhance the drainage capacity of the rock mass. Because several fractured systems will be characterized and tested with water, isolation of this test facility from other tests and the repository may be prudent.	Multiple drainage facilities in welded tuff.
<u>MISCELLANEOUS</u> Can unusual features be sealed if they are encountered? [33]	To Be Determined (TBD)	TBD	TBD

Table 2-2. Summary of Sealing Tests and Associated Objectives

Test	Objective
<u>SEAL COMPONENT TESTS</u>	
Small-Scale In Situ Test	Characterize the thermal and stress response of the hydration of the grout or concrete placed in nonwelded and welded tuff environments.
Intermediate-Scale Borehole Seal Test	Characterize the hydrologic performance of borehole seals in the nonwelded Paintbrush tuff and the upper portion of the Topopah Spring Member.
Fracture Grouting Test	Understand the hydrologic effectiveness of grout penetration in a number of different fractured environments, including the Tiva Canyon, possibly the Paintbrush nonwelded tuff, and the Topopah Spring Member.
Surface Backfill Test	Establish the consolidation behavior and the preliminary hydrologic performance of drift and shaft fill, including nonwelded and welded rockfill, due to emplacement techniques.
Grouted Rock Mass Test	Assess the thermal effects on the permeability of a fractured rock mass, as well as the thermal effects on a fractured rock mass that is grouted.
<u>PERFORMANCE CONFIRMATION TESTS</u>	
Seepage Control Test	Understand the hydrologic performance of drift fill and filter designs together with the drainage through the underlying fractured rock, the potential migration of fines and subsequent clogging of the underlying fractures, the drainage enhancement provided by various drainage designs, the effects of saturation and desaturation on the rockfill properties, and the thermal effects on the rockfill/rock interface and the rockfill itself. This testing should be done in the Tiva Canyon and Topopah Spring Members.

Table 2-2. Summary of Sealing Tests and Associated Objectives (Continued)

Test	Objective
<u>PERFORMANCE CONFIRMATION TESTS</u>	
Backfill Test	Characterize the structural and hydrologic performance of various rockfill material as emplaced in the Calico Hills unit.
Bulkhead Test in the Calico Hills Unit	Characterize the conductivity and structural performance of a full-scale bulkhead in the Calico Hills unit.
Large-Scale Shaft Seal and Shaft Fill Test	Characterize the hydrologic behavior of rigid shaft seals (including the interface zone, the MPZ, and the seal itself) in the Tiva Canyon welded tuff, the Paintbrush tuff, and the Topopah Spring Member. Characterize the hydrologic performance and the consolidation behavior of rockfill comprised of nonwelded and welded tuff with and without additives.
Remote Borehole Sealing Test	Understand the hydrologic performance of borehole seals in the nonwelded Paintbrush, the welded Topopah Spring, and the nonwelded Calico Hills units.

summary considers the types of tests proposed in Table 2-1 and the geologic environment of the sealing component. The tests discussed in Table 2-2 divide into two categories: seal component tests, which are less complex in nature, and performance confirmation tests, which are typically more complex. The seal component tests could be performed prior to LA and the performance confirmation tests could follow LA. The need for completing these tests prior to LA depends on the additional site characterization information obtained prior to LA, the repository design, and the required performance of the seal components. While the tests are intended to meet specific seal performance objectives, they are also intended to meet other common objectives. These common objectives are

- To demonstrate emplacement of sealing components using existing technologies.
- To determine effectiveness of testing instrumentation, including the range, sensitivity, and accuracy of the instruments. (This is particularly true for the surface backfill tests.)
- To develop quality-control procedures for emplacing seal components.
- To establish the reliability of the seal system under anticipated and extreme conditions.

To verify that the uncertainties defined in Table 2-1 are addressed by at least one test, a matrix of uncertainties and tests was prepared (see Table 2-3). As indicated by the matrix, each uncertainty is addressed by at least one test. In some cases, many tests address a single uncertainty.

Table 2-3. Correlation Between Uncertainty and Uncertainty Reduced by Test

Uncertainty	Uncertainty Reduced by Test ^a									
	A	B	C	D	E	F	G	H	I	J
<u>ANCHOR-TO-BEDROCK PLUG</u>										
Hydrologic Performance-1	x	x	x						x	
Structural Requirements-2	x								x	
Liner Removal and MPZ-3									x	
Keyway Excavation-4									x	
Fracture Grouting-5			x						x	
Grout Workability-6			x						x	
Shrinkage and Interface-7	x	x							x	
Workability and Settling-8									x	
<u>GENERAL SHAFT FILL</u>										
Hydrologic Performance-9				x					x	
Settlement-10				x					x	
Fines Creation-11				x					x	
<u>SEEPAGE CONTROL AND DRIFT FILL</u>										
Fracture Grouting-12			x		x	x				
Grout Workability-13			x		x	x				
Grout Plugging Sump-14			x			x				
Placing Drift Fill-15				x		x				
Shrinkage Along Fill-16						x				
Flow Control of Dam-17			x			x				
Fines Clogging Fracture-18				x		x				
Fines Control-19				x		x				
^a Sealing Tests: A = Small-Scale In Situ Tests B = Intermediate-Scale Borehole Seal Tests C = Fracture Grouting Tests D = Surface Backfill Tests E = Grouted Rock Mass Tests F = Seepage Control Tests G = Backfill Tests H = Bulkhead Test in the Calico Hills Unit I = Large-Scale Shaft Seal and Shaft Fill Tests J = Remote Borehole Sealing Tests										

Table 2-3. Correlation Between Uncertainty and Uncertainty Reduced by Test (Continued)

Uncertainty	Uncertainty Reduced by Test ^a									
	A	B	C	D	E	F	G	H	I	J
<u>STATION AND SHAFT PLUGS</u>										
<u>DOUBLE BULKHEAD-</u>										
<u>NO SETTLEMENT</u>										
Liner Removal and MPZ-20									X	
Keyway Excavation-21									X	
Fracture Grouting-22			X						X	
Grout Workability-23			X						X	
Hydrologic Performance-24									X	
Shrinkage and Interface-25	X	X							X	
Workability and Settling-26									X	
<u>BACKFILLED SUMP</u>										
Siltation of Sump-27			X			X				
<u>CHANNEL IN BACKFILLED</u>										
<u>ROOM</u>										
Siltation of Backfilled Room-28			X			X				
<u>EXPLORATORY BOREHOLE</u>										
Casing Removal-29										X
Hydrologic Performance-30	X	X								X
<u>CALICO HILLS DRIFT SEAL</u>										
Adequacy in Sealing-31	X	X	X	X				X	X	
<u>ENGINEERED DRAINAGE</u>										
<u>ENHANCEMENT</u>										
Enhance Drainage-32						X				
<u>MISCELLANEOUS</u>										
Unusual Features-33	X	X	X	X	X	X	X	X	X	X
^a Sealing Tests: A = Small-Scale In Situ Tests B = Intermediate-Scale Borehole Seal Tests C = Fracture Grouting Tests D = Surface Backfill Tests E = Grouted Rock Mass Tests F = Seepage Control Tests G = Backfill Tests H = Bulkhead Test in the Calico Hills Unit I = Large-Scale Shaft Seal and Shaft Fill Tests J = Remote Borehole Sealing Tests										

3.0 TEST AREA LOCATIONS AND ROCK AND HYDROLOGIC CHARACTERIZATION TECHNIQUES

This chapter first presents a brief description of the facilities necessary for conducting tests and then presents information on individual tests. There are two categories of testing required to support the sealing program:

Category A - Intrinsic permeability characterization of discrete, geologic features to evaluate water inflow and air outflow.

Category B - Evaluation of specific sealing concepts and sealing components.

If a test area involves only the geophysical or conductivity (hydraulic and air) characterization of a discrete geologic feature and does not involve a Category B test, the test is not considered to be a sealing test but is considered necessary to support the sealing program. It is anticipated that other organizations will be interested in this geohydrologic information and may have proposed similar tests. Acquisition of site information will be coordinated with other project participants to increase data acquisition and to decrease data acquisition costs. If appropriate, testing areas will be used by more than one project participant to reduce costs.

Category B tests include the seal component and performance confirmation tests defined in Table 2-2 that are the primary focus of this report. There are ten sealing tests to be conducted in the ESF. The five seal component tests include:

- Small-scale in situ tests.
- Intermediate-scale borehole seal tests.
- Surface backfill tests.
- Fracture grouting tests.
- Grouted rock mass tests.

The five performance confirmation tests include:

- Seepage control tests.
- Backfill tests.
- The bulkhead test in the Calico Hills unit.
- Large-scale shaft seal and shaft fill tests.
- Remote borehole sealing tests.

The tests reduce the uncertainties associated with the performance of sealing components. A number of tests are proposed over the range of expected conditions for specific locations in the underground workings.

3.1 Test Area Locations

Category A testing ties directly to specific geologic and hydrologic features. Because the variety and frequency of these features are currently unknown, the locations listed below are subject to change as the north and south ramps are characterized. The following lists present tentative locations for a complete suite of Category A tests.

For the north ramp, the locations are:

- Bow Ridge Fault.
- A characteristic fault or fracture zone beneath alluvium downgrade from the Bow Ridge Fault.
- A characteristic fault or fracture zone downgrade from the alluvial area.
- A characteristic fault or fracture zones in the Topopah Spring Member (two areas).

For the south ramp, the locations are:

- A characteristic fault or fracture zone from the surface through the Tiva Canyon to the ramp.

- A characteristic fault or fracture zone from an alluvial area, through the Tiva Canyon, the nonwelded Paintbrush tuff, and the Topopah Spring tuff, to the ramp.
- A characteristic fault or fracture zone near the repository boundary.
- Ghost Dance Fault.

Within the Topopah Spring and Calico Hills horizon, representative testing of discrete, geologic features will include major geologic features such as the Ghost Dance Fault and the imbricate fault zone.

The following list presents tentative facilities for a complete suite of seal component and performance confirmation (Category B) tests. These facilities include:

- A facility in a portion of the Tiva Canyon unit in the north ramp.
- A facility in a portion of the nonwelded Paintbrush tuff in the north ramp.
- A drift in the Calico Hills unit parallel to the drift that penetrates the Ghost Dance Fault.
- A second drift in the Calico Hills unit at the north or the south end of the facility to test the performance of a bulkhead. (This drift location depends on the hydrologic conditions encountered in the ramp that exits from the north and south ramps and enters the Calico Hills unit.)
- An alcove within the Topopah Spring main test area for a grouted rock mass test.
- A drift within the Topopah Spring main test area for a seepage control test.

For both categories of testing, standard facilities support will be necessary. In addition, there are the following requirements:

- For Category A Testing - Discrete Geologic Feature Testing--An alcove will be needed to characterize each fault and fracture zone. Each alcove should be large enough for a drilling machine that can core a distance of ~20 m. Two horizontal holes will penetrate the fault or fractured zone. The first hole will be drilled perpendicular to the discrete geologic feature and the other will be drilled to intercept the features at a currently unspecified angle. The intrinsic permeability of the zone upgrate from the fault zone, the fault zone, and the zone downgrade from the fault zone will be

characterized through multiple testing in these zones. This testing will involve a data acquisition system and a system to measure flow rates and pressures and calculate the intrinsic permeability of these zones, possibly a guarded straddle packer system. To determine the permeability through the use of air injection, a uniform supply of compressed air will be necessary.

- For Category B Testing - Seal Component and Performance Confirmation Testing-- There are six potential areas (see Figure 3-1) currently defined for seal component and performance confirmation testing. Their locations are defined below.

In presenting test facilities, the following discussion provides general guidance for the types of facilities and construction methods required in field testing. The DOE or DOE subcontractors hold responsibility for designing and constructing facilities to meet space, power, and ventilation requirements for the facility under appropriate regulations. Note also that locations for testing were selected on the basis of performing tests outside the potential repository block. Locations are subject to change.

The first potential test area is a major seal component testing facility (SCTF) in the densely welded portion of the Tiva Canyon Member in the north ramp. The facility is shown off the north ramp because the steeper grade (up to 10%) of the north ramp reduces the amount of drilling required. The tests conducted in this facility include the small-scale in situ tests, the intermediate-scale borehole seal tests, large-scale shaft seal and shaft fill tests, seepage control tests, and fracture grouting tests (Figure 3-2). This facility, as currently configured, would involve ~300 m of drifting. The drifts should be 3.7 m and 4.6 m in height and width, respectively, and should be developed using smooth wall blasting to simulate the conditions in the main ramps as closely as possible. Development of this facility using a TBM or a mobile miner could also be considered. The facility itself will include two drifts coming off the main ramp. The drift further to the east will incline at an approximate grade of 10%. Two drifts will come off this incline and connect at the far end. The second drift coming off the main ramp will be ~78 m from the drift further to the east. This drift will decline approximately 20%. Two drifts will come off this decline and both drifts will be connected at the far end. The appearance of the final facility will be four drifts parallel to the main drift; two of the drifts will be at an upper level (UL) and two at a lower level (LL). The minimum separation of the two levels is estimated as 15 m. Two shafts 3.7 m in diameter will be developed between the drifts furthest from the main

Figure 3-1. Potential Testing Areas

Figure 3-2. Three-Dimensional Perspectives of a Major Seal Component Testing Facility in Welded Tuff

ramp. The liners in these shafts will have a design similar to the final design of the repository shafts. Two 31-cm intermediate-scale boreholes will be drilled between the two levels. Fracture grouting will take place on the UL at three potential locations and on the LL at three locations not shown in Figure 3-2. Within the lower drifts, the following boreholes will be developed: two 0.91-m-diameter x 0.91-m-length holes, two 0.91-m-diameter x 1.8-m-length holes, three 0.46-m-diameter x 0.46-m-length holes, and three 0.46-m-diameter x 0.91-m-length holes.

The second potential test area is a major SCTF at the north ramp located in the nonwelded Paintbrush tuff. The north ramp was selected because the nonwelded units may be thicker in this area. The design concept for this facility will be similar to that presented above for the facility in welded tuff. However, because of the lower degree of fracturing in the nonwelded tuff as compared to the densely welded tuff, the seepage control test would not be performed. Also, depending on the nature and extent of fracturing, the number of fracture grouting tests may be modified. These changes to the SCTF would probably reduce the size of the facility.

In both facilities discussed above, borehole drilling and coring will be required to characterize the natural fracturing and excavation-induced fracturing. The number of boreholes depends on the nature of the fracture system. A compressed air supply will be necessary to determine the permeability through the use of air injection within the borehole. The exact amount of drilling is not currently defined. Air conductivity at the drift scale will also be measured. This will require the pressurization of a segment of the drift and injection of a tracer. Because of the scale of this test, a larger compressed air supply will be necessary. The size requirement of the air supply depends on the quality of the rock being evaluated. Large-scale drift testing will require temporary bulkheads possibly constructed of cinder blocks and sealed with gunite.

The third potential test area for backfill tests of the Calico Hills unit is a drift within this unit. This drift could be perpendicular to the main drift and ~ 45 m long. It would be the same design as the remainder of drifting in the Calico Hills unit. Two drifts will be associated with this drift. The first drift (Figure 3-1) would house the instrumentation for the backfill test in the Calico Hills unit. The second drift would be used to store rockfill for the backfilling operation.

Some drilling may be required to core into the backfill after placement. Excavation of the backfill may also be performed to extract samples.

The fourth potential test area is a drift in the Calico Hills unit. Because the test location depends on the geohydrologic conditions encountered in the ramps gaining access to the Calico Hills, only a generic bulkhead test is proposed here. The drift in this area will have the same design as the other drifts in the Calico Hills unit. It could be >25 m in length. Testing would involve the construction of a large-scale seal and subsequent testing behind the seal. Coring of the seal may be performed to obtain strength and hydrologic properties of the seal.

The fifth potential test area is within an alcove in the Topopah Spring main test area (Area 5a, see Figure 3-1) or in a drift off the Calico Hills access ramp (Area 5b, see Figure 3-1) for a grouted rock mass test. If the heated block test, which is currently proposed by other YMP participants, is not suitable for grouting, the grouted rock mass test will need to be performed separately. In this case, a drift similar to that developed for the heated block test will be required. The same construction requirements would apply to both tests.

The sixth potential test area is within a drift ~25 m long in the Topopah Spring main test level (Area 6a, see Figure 3-1) or in a drift off the Calico Hills access ramp (Area 6b, see Figure 3-1). The seepage control test will involve the emplacement of rockfill and will require a storage area for the rockfill. There are no specific requirements for this storage area except a reasonable storage capacity.

3.1.1 Influence of Proposed Facilities

A review of the proposed test facilities is required to determine the influence that facility design and construction methods have on the test plan and the sequence of the tests. The following sections discuss operational and design aspects of the SCTF that would be located in potential test areas 1 and 2.

3.1.2 Seal Component Testing Facility Key Equipment and Services

Rubber-tired diesel equipment may be used in the facility during the construction, testing, and demobilizing (i.e., all phases) to negotiate the grades shown in Figure 3-3. This equipment requires rounded corners, preferably flat-bottomed roadways, and an increased volume of ventilation. Equipment should be able to pass all other equipment--a most important factor in the plan. The load haul dump (LHD) units may be 1.5 to 2.3 m³ in size and would require 4.6-m (15-ft)-wide roadways to meet the passing requirement. All fetching and carrying would be done by the LHD and forklift units.

Other units needed are a combined drill and rock bolt unit, a compact combined concrete and shotcrete mixer/pump unit, a shotcrete boom-mounted unit, a pneumatic stowing unit, a compact grout-preparation plant, and man-transporter units. Compressed air and water could be supplied from the main supply lines in the north and south access ramps.

An electrical load center could be located in an alcove off the north and south access ramps. From the load center, mine-type cables would lead to the various locations requiring electric power and isolating switch/starting control units. Some of these may be portable.

Underground trucks may not be required to go beyond the loading and unloading stations at the entrance of the facility adjacent to the main access ramps, thus reducing the headroom required for loading and unloading within the facility-roadway system. Higher headroom at the shaft-collar areas would be required for the raise bore operations and for the proposed overhead crane discussed subsequently.

3.1.3 Proposed Second Means of Egress/Ventilation Maintenance in the Major Seal Component Testing Facilities

There is no connection between the UL and LL of the facility except via the main access ramp. A connection would provide a second means of egress and a quicker access between levels

Figure 3-3. Proposed Layout for a Large Seal Component Testing Facility

and could help to maintain the ventilation during tests. The raise boring equipment could be used to make a third vertical connection between the levels at intersections UL 3 and LL 3 (Figure 3-3) while the equipment is available.

3.1.4 Proposed Hoisting Means and Other Means of Handling Bulk Material

In place of the standard mine hoists for men, material, and tools and hoists for the working-stage support and movement, it is proposed to use an overhead crane for all shaft hoisting work and to hang the working stage on chains (Figure 3-4). The stage could be raised or lowered to the next working position using the chains shackled to the overhead crane. A second overhead crane will be required if test operations are conducted in both shafts at the same time. A vertical ladder with back protection will suffice for the second means of egress from each shaft. The Mine Safety and Health Administration may require an emergency hoist close by or a second overhead crane powered independently of the first.

3.1.5 Stub Ends/Turn Outs

The proposed underground test facilities include stub ends (turn outs) (Figure 3-3) at junctions LL 2, 3, 6, and 7 and UL 2, 3, 4, and 5 to provide for turning, parking, muck bays, and storing grout or shotcrete stations, etc. This is particularly necessary when the large-scale shaft seal and shaft fill and seepage control tests are in progress. A muck bay has been added near the LL main access ramp junction (LL 1) and a storage area at the UL main access junction (UL 1). Trucks can be loaded from the side, if necessary, with an overhead loader. The loading facility will be held to a reasonable size to avoid over-enlargement of the main access drift. Fill material will be dumped at the unloading station, the corner-cutting roadway ULs 6 and 7, and taken into the SCTF by the LHD units to the UL.

3.2 Rock Characterization Techniques

The rock characterization techniques include (1) direct sampling techniques such as rock core recovery and associated laboratory testing and (2) noninvasive methods such as geophysical

Figure 3-4. Large-Scale Shaft Seal Testing

techniques. Both methods will allow some correlation of properties and will be used jointly in project decision-making. The direct sampling and testing techniques, summarized in Section 3.4, will be used to select sealing locations. The properties tested for depend on specific areas of investigation; e.g., structural testing depends on the thermal and thermomechanical rock properties, and hydrological testing depends on hydrologic properties.

Geophysical characterization provides a cost-effective and noninvasive method for characterizing fracture systems and void space. In welded tuff, existing sets of fractures orient vertically with fracture apertures ranging from a few to hundreds of microns. Lithophysal zones comprise other voids encountered at selected depths.

Geophysical methods will be considered for the following purposes:

- Selection of the sealing test intervals in welded and nonwelded tuff,
- Pretest characterization of specific test intervals, and
- Monitoring of test performance.

Evaluations of existing borehole information show that the intensity of fracturing and presence of lithophysal zones vary between units. During excavation of the north ramp, geophysical mapping will be combined with fracture mapping of the tunnel surface to select the location of the two major underground facilities in welded and nonwelded tuff.

Pretest characterization emphasizes the properties of the sealing zones (borehole-scale to tunnel/shaft-scale). Actual drilling or tunnel/shaft excavation results in stress relief and possible induced fracturing at selected sealing locations (Case and Kelsall, 1987). Geophysical characterization defines the extent of the zone of loosening caused by stress relief and may provide correlation of geophysical index parameters with fracture frequency (measured by mapping) and porosity (measured in the laboratory from core samples). The laboratory measurements might include both primary and secondary porosity.

The final application of geophysical techniques is monitoring individual test performance. This application includes qualitative effectiveness of fracture grouting in reducing or eliminating void space and monitoring the movements of saturated plumes during testing.

The investigation methods selected to accomplish these objectives include both borehole (cross-borehole seismic tomography and borehole induction logging) and surface techniques [resistivity surveys and ground-penetrating radar (GPR) surveys]. The borehole-logging techniques will be used in combination as appropriate for a given test and in conjunction with other geologic drilling and sampling activities. In this way, drilled boreholes will satisfy multiple purposes. The surface-survey techniques will be used to identify suitable test locations or to monitor specific test performances.

The following sections discuss the four geophysical techniques suggested for use in the field testing activities. The final determination of which technique should be used depends on the final test configuration and specific geohydrologic setting.

3.2.1 Cross-Borehole Seismic Tomography

Cross-borehole seismic tomography at Yucca Mountain may be best suited to characterizing the geologic regime in the welded tuffs such as the Topopah Spring Member. This method images the rock mass between small-scale test boreholes or other preliminary pilot holes used in tests. The structural information developed by the surveys supports selection of seal zones. The method may be used as a supplement to verify that undesirable features do not exist in test locations requiring a relatively undisturbed rock mass, as in some nonwelded tuff tests. Configuring the survey to image the rock between the test boreholes will provide the information necessary to select the seal locations or test intervals in both welded and nonwelded tuff. Further, the U.S. Bureau of Mines (USBM) showed that the method applies to monitoring the process of a saturation front during in situ mining of dry strata (Davis and Heinrichs, 1991). The cross-borehole seismic technique places a series of receivers down one borehole and records the transmission from a seismic-energy source placed at various positions in an adjacent borehole. Activation of the energy source at different positions in the transmitting borehole produces a dense

network of seismic ray paths between the two holes (Figure 3-5). A high density of ray paths allows resolution of the features between the boreholes. The ray paths will curve as they cross various interfaces (i.e., discontinuities or saturation fronts) in the tuff.

Both seismic travel times and sine-wave amplitudes can be measured by the receivers. Travel times are used to determine the velocity of the tuff between the holes. Amplitudes show the distribution of energy-transmission characteristics. These parameters define occurrences of fracture zones, lenticular sequences, or other geologic contrasts (Davis and Heinrichs, 1991) and can be applied to mapping the joint discontinuities in welded tuff. Fracture zones in the nonwelded tuff that are too narrow to detect directly using analysis of arrival times may be revealed by anomalously low amplitudes between some of the source-receiver points.

Seismic tomography interprets the seismic data generated from the cross-borehole survey. Seismic tomography may employ two different methods of reflector reconstruction: direct, in which a set of simultaneous equations are solved by matrix inversion, or iterative, in which a trial solution is found to agree within allowable limits to the observed data values.

The advantage of using seismic tomography techniques is that it provides a relatively complete image of a given volume of tuff with minimum disturbance from borehole penetrations. Boreholes limited to the outer region of the block can be imaged without disturbing the intended test area. Boreholes located within the block are required to perform the in situ tests and may also be used for the structural image development. Several underground facility tests will require drilling pilot boreholes. The pilot boreholes may be used in conjunction with other cross boreholes to provide adequate spatial coverage.

The technique may have some limitations in its application to the field testing activities. If the tuff formations exhibit layers with significantly different seismic velocities (elastic properties and density), the seismic refraction along these contact interfaces will tend to underrepresent certain volumes of the rock mass due to preferential travel of the rays. This is not expected to be a significant issue unless the depth intervals intended for seismic imaging becomes excessive or the potential for multiple layers increases.

Figure 3-5. Straight Line Ray Paths Generated in Cross-Borehole Seismic Tomography (simplified)

Cross-borehole seismic tomography in any geologic media has inherent limitations. The more numerous the ray paths and the more divergent the angles of the rays contacting the interfaces, the more accurately the properties can be determined through tomography techniques. Restrictions in achieving thorough coverage may result from limited access through boreholes in shafts and drifts.

Seismic tomography may be applied to monitor the migration of the saturation front in certain hydrologic testing. Seismic profiling of the saturation front is a function of the contrast between the in situ rock mass saturation and the advancing wetting front. Successful observation will be limited where the contrast is small.

3.2.2 Borehole Induction Logging

Electromagnetic induction logging measures the electrical conductivity of a surrounding formation by inducing an electrical potential in the material (Figure 3-6). Electrical conductivity depends on the type of rock, its porosity and permeability, and the fluids that fill the pores. Generally, the specific conductance of the fluids will dominate the measurement. The method may apply to mapping saturation plumes generated in falling head or infiltration tests.

The instrument probe consists of both transmitting and receiving coils that run the length of a borehole. The instrument energizes the surrounding formation by induction. Current is excited in the ground around a borehole by a time-varying magnetic field with an induction transmitting coil. The receiving coil in the probe measures the response induced in the surrounding formation. Less-conductive adjacent beds do not affect the receiver readings appreciably in contrast to more-conductive zones. These measurement differences allow interpretation of surrounding formations. Note that electrical contact between the logging-system probe and the rock wall is not required. The logging tools may be run in dry or fluid-filled holes.

The effectiveness of the technique in monitoring plume development depends on the electrical conductivity contrast that in turn depends on porosity, initial degree of saturation, and

Legend

Tx Transmitting coil

Rx Receiving coil

Figure 3-6. Relation of Magnetic Field and Induced Electric Eddy Currents in Vertical Borehole

salts. The technique is being considered as a method to monitor plume propagation by surveying specifically located monitoring boreholes over time. Some field tests involving falling head observations will be configured with various access boreholes and will be surveyed at specified periods to track plume development. If contrasts in fluid conductivity are not great enough, it may be feasible to use waters with additives (i.e., salts) that render the solutions more suitable for induction of current.

3.2.3 Surface Resistivity Survey

Resistivity surveying directly introduces a current into the ground using galvanic contact points. Current is applied through one pair of electrodes and the potential established in the tuff is measured with a second pair of electrodes. The resistivity is calculated knowing the electrode spacing and geometry, the applied current, and the measured voltage (Benson et al., 1982) (Figure 3-7). Resistivity is the reciprocal of conductivity, which is measured in the induction logging technique.

Kelsall et al. (1982) described the use of this technique for determining the extent of the MPZ through vertical sounding. By varying the electrode spacing, configuration, and possibly the current level, the electrical resistivity of the tuff in the tested interval is determined.

A resistivity survey may be suitable for monitoring the groundwater plume in specific field tests. Test locations that are unsaturated prior to seal testing may be suitable for monitoring using resistivity techniques. Resistivity may be used to monitor the advance of the saturation front in the field test in which hydrologic testing is also performed. A more conductive hydrologic fluid (i.e., salts added) could be incorporated to create a greater conductivity contrast between the dry formation rock matrix and the testing fluid. The larger the contrast, the more successful the imaging of the saturation front will be.

The properties of the water contained within the pores and fractures controls electrical conductivity. In general, the tuff matrix is considered an electrical insulator. The resistance will decrease as the moisture or water content increases, and the porosity and permeability of the

Figure 3-7. Diagram Showing Basic Concept of Resistivity Measurement (Benson et al., 1982)

formation increase. Some clayey minerals may also reduce the resistivity, but such minerals should not have a significant influence in the Yucca Mountain tuff units. The actual measured resistivity will be a function of all these formation characteristics.

Resistivity methods can be divided into three classes:

- Arrangements in which the potential difference between two widely spaced measuring electrodes is recorded.
- Arrangements in which a potential gradient or electric field intensity is measured using a closely spaced pair of measuring electrodes.
- Arrangements in which the curvature of the potential function is measured using a closely spaced current electrode pair as well as a closely spaced measuring electrode pair.

The actual resistivity method employed will be determined in the field for the site-specific conditions encountered.

Vertical sounding determines the vertical changes in electrical conductivity and is accomplished by varying the electrode spacing. The larger the spacing, the greater the effect material at depth will have on the measured response. By successively increasing electrode spacing for a given survey location, variations in resistivity with depth are studied. Horizontal profiling is performed when investigating the lateral changes in the resistivity of a particular horizon. This is accomplished by holding the spacing between the electrodes constant and moving the array as a whole along a traverse line.

The advantage of performing a resistivity survey is that it is primarily a noninvasive approach to determining the nature of the rock mass. It could be performed along the upper and lower drifts. Beyond the small electrode contact points required for the earth coupling, no other penetration is necessary. This limits the disturbance to the test location. Resistivity sounding may be used to determine the depth to the water table or other conductive layers (i.e., saturation plume). It could be conducted at predetermined periodic intervals to monitor the progression of a saturated front.

Limitations in the resistivity method require that sounding be performed in a laterally homogeneous site condition, which should be possible in the bedded tuff conditions of the test facilities. As the electrode spacings are increased to achieve greater depth imaging, a lateral facies change could interfere with the depth penetration. Secondly, as with many electrical methods, cultural features can influence the electrical response and resistivity may not be a suitable method in areas of the underground where significant utilities are routed.

3.2.4 Ground-Penetrating Radar

GPR is an electromagnetic geophysical technique that uses high-frequency radio waves to acquire subsurface information. It allows the detection of structures such as major voids, fractures, and faults. The GPR method successfully mapped the development of fractures in anhydride marker beds in bedded salt at the Waste Isolation Pilot Plant (WIPP) (Westinghouse Electric Corporation, 1992). To aid in selecting the sealing horizons, the method may be applied to detect void spaces. It may also be applied to gain useful information for effective grouting of larger void zones.

The GPR survey technique will be used in a manner similar to the resistivity survey approach. It will be incorporated in the specific test location selection process to identify suitable rock mass intervals to satisfy the test objectives and in the test performance monitoring. Profiling the tuff from the underground surface provides an efficient precursory view of the material interval to be tested. When combined with other surveys (resistivity) and information, an assessment of the ground conditions should be possible. The survey method will allow performance monitoring surveys to be conducted in the most efficient manner. GPR surveys will not be influenced by the electrical or other cultural features that influence electrical resistivity survey techniques in the underground.

The GPR survey uses a transmitting antenna that emits an energy pulse over the 0.3- to 300-cm wavelength of the electromagnetic spectrum. The energy may either be transmitted through the rock mass or reflected back from subsurface structures to the surface from which it was transmitted. The reflected or transmitted energy is monitored in a receiving antenna

(Figure 3-8). The received signals contain both amplitude and temporal information. The time delay from signal transmission may be used to provide imaging of the subsurface structure because it is proportional to the depth of the interface of the void space. Both the amplitude and time delay are affected by attenuation.

The material properties include the dielectric constants of the subsurface material. The dielectric properties of the media that affect the wave velocity are a function of the composition and moisture content of the subsurface material.

Attenuation refers to the degree that the signal is reduced with increasing distance. Attenuation depends on the conductivity of the material investigated and radar frequency. An increase in either results in greater attenuation. The radar frequency is controlled and may be selected to give the optimum depth and resolution of the features of interest. Nevertheless, the depth of the signal penetration is site-specific.

The advantages of a GPR survey are listed below (Benson et al., 1982):

- The radar energy will respond to any dielectric discontinuity, which may show fractures or saturation boundaries in the tuff.
- It provides good resolution when used under suitable saturation and contrast conditions.
- Interpretation of radar data is relatively straightforward if site conditions are simple and a strong dielectric contrast exists between the features of interest.
- The survey provides a relatively continuous image of the traverse line in real time. Interpretation of the graphic record is possible in the field.
- The system may be optimized to the site-specific conditions by varying the frequency and energy sources.
- Processing techniques may be applied to the data to enhance the graphic image and aid in the interpretation.

Figure 3-8. Schematic of Ground-Penetrating Radar System

The methods of application may be limited by the following (Benson et al., 1982):

- The signal attenuates in highly conductive material and will have a reduced penetration in material with high saturation. (Note that a material with a higher degree of saturation will result in higher conductivity and therefore higher attenuation.)
- Depth of penetration is site-specific, depending on the electrical conductivity of the fluids and rock, and is commonly less than 10 m.
- In complex geologic regimes, interpretation of the raw data may be difficult.

3.3 Hydrologic Characterization Techniques

There are three basic techniques for measuring flow rate and calculating in situ permeability: (1) the constant-pressure flow test, (2) the shut-in or pressure decay test, and (3) the tracer test. These methods are applicable to either gas or liquid, but for these applications, only the gas-testing analysis will be considered. There are two types of analysis that can be applied to these techniques to evaluate gas permeability: steady-state and transient analysis. Both methods allow the calculation of air permeability for given experimental setups.

The steady-state analysis assumes steady-state flow in a homogeneous isotropic porous medium. This assumption allows the use of simple, closed-form solutions, but the application of these solutions is less general due to the assumptions used in deriving them. A disadvantage is the difficulty in attaining steady-state flow over the durations of most tests. As a result, this type of analysis often leads to approximate or order-of-magnitude permeability values from the experimental data.

Transient experimental techniques also have advantages and disadvantages. An advantage of these techniques is their ability to measure permeability of low permeability rock. The transient techniques often require substantial test durations to obtain the amount of data needed. This is a result of the numerical solution techniques that are applied to match the experimental data to the associated rock permeability.

Both the steady-state and transient analyses are applicable to in situ testing. In general, a specific type of testing only lends itself to one type of analysis; however, multiple tests are often run in a single borehole using different experimental techniques, resulting in both types of analysis being used.

3.3.1 Constant-Pressure Flow Test

Constant-pressure flow tests will be used when in situ permeability of the surrounding rock is being measured using a single borehole. This method uses a single borehole with a single packed-off section for the flow measurements. This packed-off area is maintained at a fixed pressure by the addition of gas as the pressure declines (because of gas flowing into the surrounding rock). This continues until a steady-state condition is reached between the gas flow out of the packed area and the gas flow into the packed area. The flow rate into the test area is then used to determine the permeability of the rock.

The permeability can be calculated by assuming steady-state radial or axial flow in a uniform homogeneous isotropic media. Assuming Darcy flow conditions, the permeability is given by the following equation (Stormont et al., 1987):

$$k = \frac{\partial v}{\partial t} \frac{\mu P_a \ln \left(\frac{r_a}{r} \right)}{\pi L (P^2 - P_a^2)}$$

where

k = Permeability,

μ = Fluid viscosity,

r = Radius of borehole,

r_a = Assumed depth of fluid penetration into the formation,

P = Initial measured pressure,

P_a = Ambient pressure,

L = Length of test interval, and

$\frac{\partial v}{\partial t}$ = Steady-state flow rate.

In fractured material such as welded tuff, the constant-pressure flow test might be used to determine effective fracture permeability through interpretation of fracture data as part of pretest activities. The test uses the same experimental setup.

During drilling of the borehole, a core must be taken from the proposed test interval. This core would be analyzed for the number of fractures present and their average aperture width. The information could then be used in the analysis to determine the effective permeability of the fractures.

The analysis assumes that fractures are oriented normal to the borehole axis over an infinite radial distance from the borehole. The analysis also assumes that the tuff is impermeable and flow takes place in the fractures.

In the above equation for test interval length L , the number of fractures multiplied by the average fracture aperture is used in place of the test interval length:

$$k = \frac{\partial v}{\partial t} \frac{\mu P_a \ln \left[\frac{r_a}{r} \right]}{\pi n e (P^2 - P_a^2)}$$

where

n = Number of fractures in the test interval,

e = Average aperture width of the fractures, and other terms are as defined previously.

The experimental setup can be improved through the use of a multipacker system with guard zones that minimize vertical flow and permit measurement of pressure response in the borehole above the test area. This is accomplished by placing a packer above and below the area being measured. These guarded or packed-off areas should be larger than the area being measured and kept at nearly the same pressure (Figure 3-9). Essentially, this provides radial flow out of the test area to comply with the assumption of radial flow used to derive the closed form solution shown above.

Fractures parallel to the borehole axis can also be tested in a guarded straddle packer system. Rather than pressurizing the outer packed areas to minimize flow along fractures normal to the borehole axis out of the test area, the pressure in the outer packed areas is monitored for buildup. This pressure buildup is predominantly a factor of the vertical permeability and can be analyzed using more complex numerical modeling techniques (Peterson et al., 1985) accounting for anisotropic hydrologic properties.

3.3.2 Shut-In or Pressure Decay Test

The shut-in or pressure decay test uses the same basic physical setup as the constant-pressure flow test to measure the permeability of the rock surrounding a single borehole. The difference between these two methods lies in the collection and analysis of the data. The pressure decay test examines the decrease of pressure in the packed-off section of the borehole rather than the constant flow rate that is generated by maintaining a constant pressure in that section. Because the pressure is monitored for changes over a specified period of time, transient techniques are required for the analysis.

The pressure change over time is plotted to obtain a pressure-versus-time curve for the experiment. An example of a pressure-versus-time curve is shown in Figure 3-10. This curve is evaluated using a numerical solution to the following flow equation (Peterson et al., 1985):

Figure 3-9. Guarded Straddle Packer Setup

Figure 3-10. Typical Pressure-Versus-Time Plot for Shut-In or Pressure Decay Test

$$\phi \frac{\partial (P^{1/\gamma})}{\partial t} = \nabla \frac{kP\gamma}{\mu} \nabla (P^{1/\gamma}) + w(t) \frac{(P_a^{1/\gamma})}{\rho}$$

where

ϕ = Formation porosity,

γ = Ratio of specific heats,

$w(t)$ = Mass flow rate, and

ρ = Fluid density, and other terms are as defined previously.

Type curves can be generated using the numerical solution to the previous equation to allow for a preliminary match to be made. Because the model simulates the experimental setup for varied permeabilities and porosities, both of these parameters may be identified. Once the type curves are used to find a close match to the experimental curves, the numerical model can be used to further refine the values using iterative techniques based on the measured pressure.

3.3.3 Tracer Test

Tracer tests are used when permeability is to be measured across a given area. This test involves the use of two or more measuring points (i.e., across a concrete borehole seal, between two boreholes, etc.). To evaluate tracer test results, steady-state flow conditions are required between the two points (Stormont et al., 1987).

The injection zone is pressurized to the desired level and held at that constant pressure, inducing a pressure gradient to the withdrawal zone. Once attaining steady-state flow, the tracer test commences.

The flow of tracer breaks through from the injection zone to the withdrawal zone and shows the velocity of gas flow. This data can then be used to evaluate the permeability of the host rock formation.

In cases in which a tracer test might be performed in conjunction with the constant-pressure flow test, a direct analysis estimates the fracture aperture of the test zone. The test assumes a uniform series of fractures. This analysis provides a confirmation of the average fracture aperture used in the constant-pressure flow test analysis.

This analysis is based upon the arrival time of the tracer to the second borehole. The arrival time determines the ratio of the flow zone permeability to the porosity. If the flow occurs along the fractures, the arrival time can be used to determine a rough measure of fracture aperture using the following equation (Stormont et al., 1987):

$$e = d \left(\frac{12\mu}{T(P - P_a)} \right)^{1/2}$$

where

T = Tracer transmit time,

d = Distance to the measuring point,

P = Pressure of the injection zone,

P_a = Pressure of the withdrawal zone, and other terms are as defined previously.

3.4 Laboratory Testing of Seal Material

This section presents the standard laboratory tests that may be employed to determine and verify specific seal properties. Many of the laboratory tests will be performed prior to conducting the field tests in the ESF. The results of laboratory tests will support the seal material specifications for each field test. The objectives of the standard laboratory tests are to provide characteristic properties for the seal material emplacement and to assure that the material meets material and design specifications.

Laboratory test descriptions are provided for specific tests on earthen and cementitious material in Tables 3-1 and 3-2. The tables summarize information on test description, specimen size, and test standards for earthen and cementitious material. The standard laboratory tests for earthen material include hydraulic conductivity, moisture-density relations, sieve analysis, and shrinkage and swelling potential. The standard laboratory tests for cementitious material include cement-particle size, hydraulic conductivity, setting time, viscosity, and workability.

The tests identified in Tables 3-1 and 3-2 may be considered typical index property tests that can be used in the field testing program. They will be used to identify the primary performance properties of the seal material (i.e., hydraulic conductivity and unconfined compressive strength), but the list should not be considered comprehensive.

Laboratory and field confirmation tests require various standard and specialized equipment. The suggested testing presented in the tables also indicates the American Society for Testing and Materials (ASTM) standard methods where appropriate. When no appropriate ASTM method applies, the next most significant method is referenced (U.S. Army Corps of Engineers [COE] Concrete Research Division--Concrete [CRD-C]; International Society for Rock Mechanics [ISRM]; or U.S. Nuclear Regulatory Commission/Contractor Report [NUREG/CR]).

3.5 Instrumentation for Field Testing

The testing described in Chapter 4 requires structural, hydraulic, and thermal instrumentation, as given in Table 3-3. Specific instrumentation requirements are described in Chapter 4.

Table 3-1. Summary of Laboratory Testing on Earthen Seal Material

Property	Test Description	Specimen Size	Test Standard ^a	Remarks
Density	Laboratory moisture/density test (Proctor test) to determine optimum compacted density and moisture content required to obtain that density	3,000 - 5,000 g	ASTM D 698-78 or ASTM D 1557-78	
Moisture content	Laboratory test for determination of moisture content of a field soil sample	3,000 g	ASTM D 2216-80	
Particle gradation	Laboratory sieve analysis of field samples; performed prior to and after placement	3,000 g minimum	ASTM D 422-63	
Swelling capacity	Laboratory test for determination of the swelling pressure under volumetric confinement; applied where clay mix fills are evaluated	Compacted cylinder	ISRM Suggested Methods	Test may have to be performed at elevated temperatures
Shrinkage due to drying	Laboratory test for shrinkage factor of soil; applied where clay mix fills are evaluated	30 g minimum	ASTM D 427-83	
Hydraulic conductivity of seal material	Laboratory test for hydraulic conductivity (constant head or constant flow, or falling head or pressure decay)	Compacted cylinder	ASTM D 2434-68 or similar	Test may have to be performed at elevated temperatures
^a See spec...c standards: ASTM, 1990; ISRM, 1981; CRD-C, 1988; or NUREG/CR (Daemen et al., 1983).				

Table 3-1. Summary of Laboratory Testing on Earthen Seal Material (Continued)

Property	Test Description	Specimen Size	Test Standard ^a	Remarks
Hydraulic conductivity of the interface zone	Laboratory hydraulic conductivity test of a seal emplaced in a block of rock	Dependent on test apparatus	NUREG/CR-3473	Test may have to be performed at elevated temperatures
Particle gradation of fines	Laboratory sieve analysis of field samples (including hydrometer analysis)	3,000 g minimum	ASTM D 422-63	
Hydraulic conductivity of the welded tuff floor	Laboratory test for hydraulic conductivity of rock (constant head or constant flow, or falling head or pressure decay)	10 cm high x 10 cm diameter or 15 cm high x 15 cm diameter	CRD-C 48-73 or similar	The standard test method would need to be modified by testing with a permeant fluid containing suspended solids to simulate silting

^aSee specific standards: ASTM, 1990; ISRM, 1981; CRD-C, 1988; or NUREG/CR (Daemen et al., 1983).

Table 3-2. Summary of Laboratory Testing on Cementitious Seal Material

Property	Test Description	Specimen Size	Test Standard ^a	Remarks
Cement particle size	Laboratory wet or dry sieve analysis of cement	50 g minimum	ASTM C 430-89, C 184-83, or C 786-83	
Hydraulic conductivity of grout	Laboratory test for hydraulic conductivity of low-permeability material (constant head or high pressure, constant flow, or falling head or pressure decay)	Dependent on test apparatus	CRD-C 48-73 or similar	
Setting time	Laboratory or field test of time of set by penetration resistance	15 cm x 15 cm minimum	ASTM C 403-90	Test should be performed on samples in simulated expected environment
Grout viscosity, workability	Laboratory or field test of grout flow using flow cone	1,721 cm ³ wet grout	ASTM C 939-87	
Density/entrained air	Laboratory or field test of unit weight, yield, and air content of concrete	11,327 cm ³ wet concrete	ASTM C 138-81	
Hydraulic conductivity of seal material	Laboratory test for hydraulic conductivity of low-permeability material (constant head or high pressure, constant flow, or falling head or pressure decay)	Dependent on test apparatus	CRD-C 48-73 or similar	Test may have to be performed at elevated temperatures
^a See specific standards: ASTM, 1990; ISRM, 1981; CRD-C, 1988; or NUREG/CR (Daemen et al., 1983).				

Table 3-2. Summary of Laboratory Testing on Cementitious Seal Material (Continued)

Property	Test Description	Specimen Size	Test Standard ^a	Remarks
Hydraulic conductivity of seal material	Laboratory test for hydraulic conductivity of low-permeability material (constant head or high pressure, constant flow, or falling head or pressure decay)	Dependent on test apparatus	CRD-C 48-73 or similar	Test may have to be performed at elevated temperatures
Hydraulic conductivity of the interface zone in a block or rock	Laboratory hydraulic conductivity test of a seal emplaced in a block of rock		NUREG/CR-3473	Test may have to be performed at elevated temperatures
Proper mix proportions (water-to-cement [w/c] ratio)	Surveillance of proportioning during mixing procedure at batch plant and general acceptance inspection of premixed concrete	N/A	ASTM C 94-90	
Concrete aggregate gradation	Sieve analysis of concrete aggregate prior to mixing at batch plant	15 kg minimum	ASTM C 136-84a	Sample should be taken from aggregate hopper shoot to be representative of aggregate being mixed on concrete
Unconfined compressive strength	Field sampling and laboratory unconfined compression testing of concrete cylinders	7.5 cm diameter x 15 cm high cast concrete cylinders (minimum)	ASTM C 39-86	
Concrete viscosity, workability	Field slump test of concrete as it is being emplaced	5,600 cm ³ wet concrete	ASTM C 143-90	Test should be performed immediately before other concrete samples are prepared
^a See specific standards: ASTM, 1990; ISRM, 1981; CRD-C, 1988; or NUREG/CR (Daemen et al., 1983).				

Table 3-2. Summary of Laboratory Testing on Cementitious Seal Material (Continued)

Property	Test Description	Specimen Size	Test Standard ^a	Remarks
Heat of hydration	Laboratory or field measurements of heat of hydration in concrete seal	91 cm diameter x 91 cm high cast concrete cylinder (minimum)	CRD-C 38-73 or related method: ASTM C-186-86	
Thermal diffusivity	Laboratory test for thermal diffusivity of concrete	15 cm diameter x 30 cm high	CRD-C 36-73	
Thermal expansion	Laboratory test for coefficient of linear thermal expansion of concrete	7.5 cm x 7.5 cm x 28.5 cm prism	CRD-C 39-81	
^a See specific standards: ASTM, 1990; ISRM, 1981; CRD-C, 1988; or NUREG/CR (Daemen et al., 1983).				

Table 3-3. Summary of Potential Instrumentation for Field Testing

Instrument	Function
Rock Displacement Gage (USBM or Ames)	Measures diametrical changes in small-diameter drillhole; allows determination of the in situ stress by overcoring instrument and measuring subsequent deformations.
Commonwealth Scientific and Industrial Research Organization (CSIRO) Hollow Inclusion Stress Cell	Measures the strain response in three orientations of a borehole from overcoring; allows determination of the three-dimensional state of stress.
Hydraulic Pressure Cell (or Flatjack)	Measures total pressures or supports the collection of deformability data in combination with strain gages. (Note these pressure cells may be monitored for hydraulic pressure directly or with pressure transducers that convert to electrical signals.)
Concrete Stress Cell (Carlson)	Measures compressive stresses in the concrete independent of the changes in Young's modulus.
Strain Gage	Measures strain while embedded in concrete. Strain is measured and related to stress through an empirically developed stress-strain relationship. (Meters may use electrical resistance, vibrating wire, or elastic wire depending on the sensitivity and durability required.)
Earth Pressure Cell	Measures the total pressure in an earth fill. Cells are designed for the direct measurement of the pressure of a granular material against a solid structure.
Piezometer	Measures pore fluid pressure in geologic material. Piezometers may be simple open tubes to monitor water elevation or electrical or pneumatic pressure transducers to allow remote monitoring of pore pressures.
Pore Pressure Cell	Measures pore pressure within an earth fill. Specialized pore pressure transducer monitoring device for embedment within backfill. Allows direct monitoring of pore water or air pressure.
Soil Moisture Tensiometer or Suction Lysimeter	Directly measures vacuum pressure in the geologic material. At 0 bars of pressure, the observation zone is saturated.
Thermocouple Temperature Sensor	Measures temperature in rock mass, concrete, or test fluids. Operates on the principle that two dissimilar metals produce a small thermoelectric voltage when heated. The voltage change represents the temperature change. Rugged construction. Accuracy normally $\pm 1^{\circ}\text{C}$.
Resistance Temperature Detector (RTD)	Measures temperature in rock mass, concrete, or test fluids. Operates on the principle of change in electrical resistance in a wire with temperature. Accuracy $\pm 0.1^{\circ}\text{C}$.

Table 3-3. Summary of Potential Instrumentation for Field Testing (Continued)

Instrument	Function
Vibrating Wire Temperature Sensor	Measures temperature in rock mass, concrete, or test fluids. Monitors frequency change of vibrating wire as a result of temperature. Accuracy $\pm 1^{\circ}\text{C}$.
Thermistor	Measures temperature in rock mass, concrete, or test fluids. Very accurate temperature probe.
Anemometer	Measures air flow velocity. Available for air flow monitoring over a wide range air flow velocities.
Moisture Density Nuclear Gage	Measures the moisture content or placed density of the earth fill by correlation to a known standard. May be employed to monitor moisture changes in fill material.
Flowmeter	Measures liquid flow rates. The differential devices include orifices, venturi tubes, flow nozzles, pitot tubes, and variable area meters. Positive displacement meters include piston, oval-gear, rotating disk, and rotary-vane meters.
Settlement Gage	Measures the differential movement between two points based upon fluid pressure concepts.
Strain Relief Pin	Measures the displacement between two fixed points on the rock mass, typically by the mechanical caliper method.
Multiple Position Borehole Extensometer (MPBX)	Measures the relative displacement between several anchor points within a borehole.

4.0 SEAL COMPONENT AND PERFORMANCE CONFIRMATION TESTING

In the following sections, ten sets of tests are described. These include seal component tests that examine special issues and concentrate on combined office, field, and laboratory studies that lead to the selection of seal material and field quality-control methods and the subsequent performance confirmation tests. The performance confirmation tests take the selected material and field quality-control methods and scale up to larger tests that measure seal system performance. The performance confirmation tests validate the processes of analyzing, designing, and constructing large-scale seal components. These tests will be coordinated with other project participants to increase data acquisition at the site and to decrease data acquisition costs.

4.1 Small-Scale In Situ Tests

This test series includes shallow tests (<2 to 3 m) instrumented with stress and strain gages in the seals and surrounding rock that evaluate cement-hydration effects at various scales. Various scales are selected because hydration effects in the placement of cementitious material are scale-dependent.

4.1.1 Description of the Test

Objective of the Test--The objective of these tests is to evaluate the structural performance of the interface zone that is dependent on the moisture and thermal environments during the hydration of cementitious material. The degree of restraint offered by the surrounding rock mass also affects interface-stress development and represents a condition not easily evaluated in the laboratory. Previous investigations in basalt (Kelsall et al., 1982) showed the relative importance of the interface zone. To place seals of low conductivity with a high level of hydrologic performance, it is necessary to evaluate effects of shrinkage, hydration, and temperature on the interface stress. The current design locates the primary sealing components in exploratory boreholes, shafts, and ramps away from high-temperature zones; it is therefore unnecessary to test these components in elevated temperature conditions. If the Calico Hills bulkhead test discussed

in Section 4.8 is required, then the test objectives of small-scale testing would be expanded to evaluate structural performance of seals in a higher-temperature environment.

Structural Hydration Analysis--During emplacement of the plug, the heat of hydration produces increases in temperature and thermal gradients resulting in thermal stress. Initially, the plug is in a plastic state as thermal stresses increase. Finally, the plug is in a more rigid elastic state when the thermal stresses reduce.

Design mix development for cementitious seals considers such factors as water-to-cement ratio, aggregate size, and temperature and moisture environments at depth. The design mix attains workability partially through control of these factors. The geothermal conditions at depth control temperature of the surrounding rock. The placement temperature controls the initial temperature of the mix; exothermic chemical reactions during cement hydration then elevate the temperature. The water-to-cement ratio, humidity, and containment (or addition) of moisture during the curing process controls the design-mix moisture content.

The plug expands thermally during curing and contracts during subsequent cooling. Residual compressive or tensile thermal stresses develop within the plug and may result in separation at the interface zone. The permanent effect, shown by modeling, depends on the heat evolution due to hydration and thermal diffusion to the surrounding rock mass. It also depends on the thermal expansion coefficient of the plug and the evolution of thermomechanical properties over time.

In general, the formation of expansive constituents leads to the volumetric expansion of cement (Hoff, 1974) and the development of interface stress. Effectiveness depends on temperature, moisture environments, evolution of thermomechanical properties, and degree of external restraint. Moisture and temperature environments affect the rate of exothermic reaction and, therefore, the rates of expansive-constituent formation and hardening (Beirute and Tragesser, 1973). Clearly, a reduction in water available for cement hydration reduces the rate and amount of expansive constituent formation. Expansivity may be optimized only if moisture is continually available during curing.

Analysis of the development of interface stress during curing requires knowledge of expansivity and stiffness as functions of time and restraint in representative environmental conditions (see Appendix D, Structural Hydration Calculations). Both volumetric expansion and stiffness properties change with time during curing. The relationship between the plug-material stiffness and the volumetric expansivity can be characterized as early stiffening (i.e., the material stiffens before the main expansive phase) or late stiffening (i.e., the material stiffens after the main expansive phase). An early-stiffening material develops a higher interface stress. A late-stiffening material develops a lower interface stress because expansion precedes hardening or stiffening of the material. The degree of restraint offered by the surrounding rock mass and formwork also affects the development of interface stress. In principle, ettringite grows preferentially toward regions where lower restraining forces are acting. In tests on cement performed by Ish-Shalom and Bentur (1975), a stress value of 0.4 MPa was developed under uniaxial restraint, whereas values exceeding 9.8 MPa were developed under isotropic triaxial restraint.

Placement temperatures also affect the development of interface stress. As discussed in Appendix D, an increased placement temperature increases the peak temperature in the plug due to the rapid release of the heat of hydration, resulting in heat diffusing away from the plug. In contrast, a reduced placement temperature (4°C) might result in a smaller initial temperature rise such that the peak temperature never exceeds the ambient rock temperature. Only the plug would then be subject to heating.

In presenting supporting analyses in the following discussion and in Appendix D, an important objective will be predictions of temperature with space and time. These predictions are necessary for this series of tests for proper placement of instrumentation to measure thermal gradients and their thermomechanical effects and to identify high curing temperatures that could damage instrumentation or render their response nonlinear.

Performance Objective and Supporting Analysis--A one-dimensional numerical model, SHAFT.SEAL, solves for temperatures and stresses as a function of radial position, either in the plug or the surrounding host rock. Heat transfer occurs by conduction, and the model uses the

implicit finite-difference method to analyze the temperature rise and subsequent fall following completion of the hydration (Carnahan et al., 1990). The model performs thermal and volumetric stress analysis using closed-form solutions that account for thermal or volumetric initial strains.

In the heat transfer analysis, the cementitious plug is long in comparison to its diameter and heat transfer is by conduction to a surrounding rock mass of infinite extent. The governing partial differential equation is the diffusion equation. Approximate first- and second-order finite difference expressions for the plug, the surrounding rock, and the interface, coupled with appropriate boundary conditions, solve the governing equation.

The structural model uses the theory of linear elasticity to calculate thermal or volumetric stresses (Timoshenko and Goodier, 1970). The solution assumes the plug-interface stress equals the rock-interface stress and the plug radial displacement equals the rock radial displacement at the boundary of the plug.

The analysis uses input parameters and the range of values used in parametric analyses presented in Appendix D. Three categories comprise the model input parameters: the thermal and thermomechanical properties of the host rock, the thermal and thermomechanical properties of concrete, and the environmental conditions at depth. The analysis uses these factors to evaluate plugs of different diameters under different injection pressures and placement temperatures. These preliminary analyses show that the peak seal temperatures from an ambient rock temperature of 22.7°C ranges between 20°C to 40°C with interface stresses ranging from ~ -0.4 MPa to ~ 1.5 MPa.

4.1.2 Layout and Important Parameters

The present layout consists of conducting separate small-scale in situ tests in welded and nonwelded tuff. The tests consist of evaluating two different-sized seals in welded and nonwelded tuff, as illustrated in Figures 4-1 through 4-4. The first tests examine a 0.46-m-diameter seal instrumented for temperature, interface stress, and interface circumferential and radial strains. Additional temperature and stress instrumentation is placed in the surrounding rock. The tests are

Figure 4-1. Small-Scale In Situ Test (46 cm)--Weided

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Figure 4-2. Small-Scale In Situ Test (91 cm)--Welded

- Notes:**
- Tests to be performed on L/D = 1, 2
 - In situ stress measured at test location prior to excavation of borehole

Figure 4-3. Small-Scale In Situ Test (46 cm)--Nonwelded

Figure 4-4. Small-Scale In Situ Test (91 cm)--Nonwelded

conducted on a plug with the length equal to the diameter and on a plug with the length equal to twice the diameter. The instrumentation is placed near the center of the seal under the largest confinement and near the ends of the seal where less constraint is provided. Circumferential and radial strains are measured in directions of 0, 60, and 120 degrees, relative to the direction of the minor principal stress, to evaluate the stress conditions in a plane perpendicular to the seal axis. This requires characterization of the state of in situ stress prior to performing the test. This pretest characterization is especially important for determining the safe injection pressure.

A second series of tests examines the development of interface stress for a larger 0.91-m-diameter plug. The larger plug has a lower surface-area-to-volume ratio; therefore hydration temperatures are expected to exceed those in the 0.46-m-diameter seal test.

The emplacement of the seal may require an injection pressure. An injection test determines the safe injection pressure. The safe injection pressure is the pressure that develops radial interface stress along the periphery of the seal, but does not degrade the surrounding formation. Information on in situ stress at depth in the sealing zone is necessary to estimate the safe injection pressure. The plug can then be emplaced either by injection or by the dump-bailer method. Using either method, it is necessary to complete emplacement before the cement gels.

Parameters to be Measured--The parameters to be measured during the small-scale in situ tests performance will concentrate on the seal material in the 0.46-m-diameter tests and on both the seal material and rock mass in the larger 0.91-m tests. The following is a list of parameters to be measured in the small-scale in situ tests.

- Geomechanical measurements.
 - Test-induced stress measurements in the seal material and the surrounding disturbed zone.
 - Thermomechanical behavior (stress and strain) of the seal and surrounding rock mass.
- Thermal measurements.
 - Concrete hydration temperatures.

Measurement Methods--Measurements in the 0.46-m test are restricted primarily to the cementitious material, due to the limiting size of the borehole. Instruments and cable routing will intentionally be limited so that test performance is not influenced. In the 0.91-m test, the number of seal horizons instrumented will be increased and the rock mass also will be instrumented.

Anticipated instrument/gage locations are shown in Figures 4-1 through 4-4. The 0.46-m-diameter tests will have instruments at depths of 15%, 50%, and 85% of the length of the test boreholes. The 0.91-m boreholes will be instrumented at additional depths of 33% and 67% of the length of the test boreholes. These intervals identify the locations of configurations of thermocouples, pressure cells, strain gages, and displacement gages for the given horizon in the seal. The larger-diameter holes will have instrumentation installed at the edge of the seal and also bonded on the rock.

Many types and designs of thermocouples are suitable to measure temperatures in the rock mass and seal component. Temperatures expected for the small-scale in situ tests are on the order of 22.7°C (ambient rock temperature) to 50°C. Sensors with a range of 0°C to 100°C will be sufficient for application in the test. The required number of temperature-sensing elements are summarized in Table 4-1.

Stresses will develop in the concrete as a result of temperature changes associated with cement hydration, volumetric expansion or contraction due to hydration, and closure of existing joints in the adjacent rock. The small-diameter in situ tests will be configured with an embedded pressure cell near the seal and borehole interface to permit observation of the radial stress changes at this interface during hydration of the cementitious seal. An earth pressure cell was chosen for emplacement because the limited size of the instrument reduces the impact of the cell on seal performance. Earth pressure cells may be several centimeters in diameter, but they are thin discs much less than a half-centimeter thick. Pressure is transmitted through hydraulic fluid in the cell to a pneumatic or electrical transducer in the system. The data is then recorded. Embedded concrete strain gages are frequently used in this application, but their large dimensions, 15 cm to 20 cm, are considered excessive in a 0.46-m in situ test. One pressure cell located at the seal midpoint is planned for each of the 0.46-m in situ tests, for a total of 12 instruments.

Table 4-1. Thermocouples for the Small-Scale In Situ Tests

Temperature Gage Summary			
Test	Number of Tests ^a	Gages per Test	Total Gages
0.46-m nonwelded tuff	6	13	78
0.91-m nonwelded tuff	4	21	84
0.46-m welded tuff	6	13	78
0.91-m welded tuff	4	21	84
^a Note the tests include variations in the seal length-to-diameter ratio (L/D), different material, and different emplacement conditions.			

The larger-diameter small-scale in situ tests will be configured with a stress cell embedded in the seal at midheight and located at the seal and borehole interface (Figures 4-2 and 4-4), to observe the radial stresses created during the expansion and contraction cycles. Stresses for the 0.91-m in situ tests will be designed to be compressive. The stress cell is designed to measure stress independent of the changes in the elastic modulus of the concrete during curing. The instrument will be placed at the borehole surface interface prior to concrete seal emplacement. The instrument face will be epoxied to the borehole surface. One stress cell will be installed for each of the 0.91-m in situ tests, for a total of eight instruments.

A rock displacement gage will be located in a nearby parallel borehole. The small-diameter borehole will be drilled prior to construction of the larger-scale borehole seal test. A displacement gage (e.g., USBM or Ames) will be placed in the borehole to observe the stress relaxation due to excavation of the larger hole. Once the seal is emplaced and expansion occurs, the borehole displacements will be converted to borehole closures and associated stress changes as calculated.¹ The rock displacement gage and stress cell instrumentation is summarized in Table 4-2 below.

¹Obert and Duvall (1967) provide a method for calculating changes in stress in two dimensions from changes in diametral deformation of a circular hole. This technique can be used to calculate changes in stress induced by seal hydration.

Table 4-2. Rock Displacement Gages and Stress Cells for the Small-Scale In Situ Tests

Rock Displacement Gage and Stress Cell Summary ^a			
Test	Number of Tests	Gages per Test	Total Gages
0.46-m nonwelded tuff	6	2	12
0.91-m nonwelded tuff	4	2	8
0.46-m welded tuff	6	2	12
0.91-m welded tuff	4	2	8
^a Note that this table includes both rock displacement gage and stress cell requirements.			

It is anticipated that one instrument will be emplaced in a monitoring borehole near each of the small-scale in situ tests, for a total of 20 instruments, i.e., 20 tests. Depending on the number of tests performed concurrently, some of these instruments may be reused as they are not bonded to the borehole surface. The combined number of rock displacement gages and stress cells is 40.

Strain measurements will be obtained in conjunction with the other direct stress and borehole displacement measurements. Radial and circumferential strains will be measured in the cementitious seals of the 0.46-m in situ tests and in both the seals and at the borehole interface surface in the 0.91-m tests. Strains will be measured using mostly strain gages, which operate on the basis of changes in electrical resistance across a finite interval. Straining the wire in the element changes its cross-sectional area and therefore its resistance, which is measured directly. It is anticipated that strains in the cementitious seals will be measured with encapsulated gages embedded in the seal and rock surface strains will be measured using a bonded thin-wire element gage in the 0.91-m tests. The bonded gages require meticulous care in preparing the surface and cementing the gages in place to ensure protection from moisture.

The embedded strain gages will be configured to measure the radial and circumferential strains in the seal component near the interface. Those gages bonded to the rock surface

commonly use an elastic wire as the electrical resistance sensing element. Two coils of steel wire are wound around a ceramic spool; one coil increases in length as the other decreases. The ratio of the changes in resistance is a result of deformation of the instrument alone. The gages provide a temperature measurement to be used in the temperature correction of the readings. The strain gage instrumentation is summarized in Table 4-3 below.

Table 4-3. Strain Gages for the Small-Scale In Situ Tests

Radial/Circumferential Strain Gage Summary			
Test	Number of Tests	Gages per Test	Total Gages
0.46-m nonwelded tuff	6	9 ^a	54
0.91-m nonwelded tuff	4	30	120
0.46-m welded tuff	6	9	54
0.91-m welded tuff	4	30	120
^a Radial and circumferential strains measured at each location.			

Embedded strain gages will be located as shown in Figures 4-1 through 4-4. Nine sets of strain gages will be located in the concrete seal in the 0.46-m in situ tests, representing 18 measurement points. Thirty sets of gages will be located in the 0.91-m in situ test, for 60 measurement points. Circumferential and vertical strains will be measured at the borehole rock surface, as it will not be possible to measure the radial component of strain in the rock mass.

Very-early-time concrete-strain data will be difficult to interpret, because the concrete is changing from a viscous liquid to a relatively stiff solid and the properties will be changing rapidly during this period (Stormont, 1987). Detailed laboratory testing is expected to verify the appropriate start time for meaningful data collection. Preliminary modeling of the seal performance using the SHAFT.SEAL code indicates thermal strains should be on the order of 0.001.

A data acquisition and recording system will be required for performance of the in situ measurements. This data acquisition system will be capable of automatic timed readings of all instruments. It will perform any necessary preprocessing or signal conversion prior to recording the digital information. It will be capable of storing and transferring data for analysis and reporting. The polling devices may be a combination of specific devices that monitor one test instrument or centralized systems that concurrently poll and record a number of tests. Duplication will be limited to that required for efficient collection of the measurements and as necessary for system redundancy and backup.

The system is likely to include the following components (after Roberds et al., 1982):

- A central processing unit (CPU) that will control the logic of the system and the measurement and recording devices.
- An analog to digital converter that will convert voltages to digital signals.
- A multiplexer that will allow coupling of numerous input and output channels to the CPU.
- Data storage devices to maintain the data temporarily.

The data acquisition system will need to support the following minimum channel requirements² for each test:

- 13 or 21 channels for temperature output for 0.46-m and 0.91-m tests, respectively.
- 1 channel for pressure transducer output.
- 18 or 60 channels for strain gage output for 0.46-m and 0.91-m tests, respectively.
- 6 channels for deformation gage output for the 0.46-m tests (3 channels per instrument) and 3 channels for deformation gage output for the 0.91-m tests.
- 1 channel for the extensometer used in the 0.91-m tests.

²For all tests described in this report, additional channels would be required to correct for variations in signal output and to allow for redundancy in measurements.

Monitoring frequency will be refined during the detailed laboratory testing for specific seal mixtures. In general, the frequency will be greatest in early time and decrease as the concrete curing proceeds.

4.1.3 Fielding of the Experiment

The nature of the small-scale in situ tests requires that they be conducted in a portion of a dedicated drift or alcove in the welded and nonwelded tuff facilities. The tests may continue for long periods and disturbance should be minimized. The dedicated drift should be selected so that unrelated activities do not interfere with the drift stability or the stress regime in the immediate test area. Much of the equipment required to construct the test is normally available in underground mining or construction operations. Drills capable of drilling grout, instrumentation holes, and large-diameter boreholes for the test seals will be required. A batch-mixing facility for the cementitious material may be needed near the test drifts.

4.1.4 Miscellaneous Requirements

Standard and nonstandard laboratory testing will provide for the selection of cementitious material used in this series of tests. These tests include cement particle size, setting time, viscosity, and workability. Measurements of unconfined compressive strength, slump, density, and moisture content will be used to develop design specifications for the emplacement of material. Should measured properties show that material do not meet design specifications, modifications to the material or the emplacement techniques may be made.

Laboratory Testing--Laboratory testing of grout material is needed prior to fielding the fracture grouting tests. In a laboratory testing program, it is fundamentally important to define:

- Material used.
- Mixing, casting, and curing properties.
- Concrete properties.

The cementitious material proposed for use in these experiments contain cement, reactive silica, nonreactive silica, superplasticizer, defoamer, and water.

Several different cementitious material for seals are proposed, due to the potential for different geochemical environments. The properties that will be determined include:

- Fluid properties.
 - Setting time.
 - Workability.
 - Density.
 - Viscosity.
- Physical properties.
 - Porosity.
 - Density in the hardened state.
 - Cement particle size.
- Strength properties.
 - Unconfined compressive strength.
 - Poisson's ratio.
 - Triaxial compressive strength.
- Chemical characteristics.
 - Mineral composition.
 - Bulk chemistry of the constituents.
 - Resistance to dissolution.

In addition to determining these properties, the effects of wet curing, high-humidity curing, and curing time on properties will also be evaluated.

Field Quality-Control Measures--To assure proper quality control of seal material emplacement, quality-control testing may be performed prior to and during emplacement of the cementitious material to assure that the design specifications are met. The tests use quality-control testing specific to cementitious material. The recommended quality-control laboratory and field tests for cementitious material on selected samples can include:

- Surveillance of proportioning during mixing procedure (at batch plant) as well as general acceptance inspection of premixed concrete prior to emplacement (ASTM C 94-90, "Standard Specification for Ready-Mixed Concrete").
- Sieve analysis of concrete aggregate prior to addition of water into the mix (performed at the batch plant) to verify that grain-size distribution is within allowable design specifications (ASTM C 136-84a, "Standard Method for Sieve Analysis of Fine and Coarse Aggregates").
- Slump tests of the mixed concrete periodically during the pour to reverify frequently that the concrete has the design-specified workability and water content for proper emplacement (ASTM C 143-90, "Standard Test Method for Slump of Hydraulic Cement Concrete").
- Setting time of mixed concrete to reverify concrete workability (ASTM C 403-90, "Standard Test Method for Time of Setting of Concrete Mixtures by Penetration Resistance").
- Making and curing concrete test specimens (ASTM C 31-90, "Making and Curing Concrete Test Specimens in the Field") for use in determining the unconfined compressive strength of the concrete (ASTM C 39-86, "Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens") and comparing that strength with the design-specified compressive strength.
- Determination of the unit weight and entrained air of the concrete during emplacement (ASTM C 138-81, "Standard Test Method for Unit Weight, Yield, and Air Content [Gravimetric] of Concrete") to verify these properties are within design specifications.

Special Facility Requirements--The special facility requirements were discussed previously in Chapter 3. In addition, a grout batch plant will be necessary for blending material.

Constraints on Testing--The drilling should not introduce water into the underground facility.

4.2 Intermediate-Scale Borehole Seal Tests

This series of tests includes 0.3-m-diameter holes drilled from the upper drift to the lower drift and instrumented with flowmeters and pore pressure cells to determine the flow resistance of the seals in nonwelded and welded tuff (Figures 4-5 and 4-6).

Figure 4-5. Intermediate-Scale Borehole Seal Tests in Nonwelded Tuff (steady state or falling head)

Sequencing of Test

1. Characterize fractures and permeability around borehole.
2. Grout fractured rock.
3. Drill out grout in borehole.
4. Characterize open hole in the grouted zone for permeability changes.
5. Place plug and bentonite at bottom of hole.
6. Conduct steady-state and falling-head tests.
7. Core into rock mass to determine extent of fracturing.

Figure 4-6. Intermediate-Scale Borehole Seal Tests in Welded Tuff

4.2.1 Description of the Test

The intermediate-scale borehole seal tests examine the hydrologic performance of borehole seals surrounding interfaces and MPZs located in welded and nonwelded tuff environments.

Objective of the Test--The objective of these tests is to use geophysical and hydrological methods to characterize the geologic and hydrologic properties of the rock mass at sealing locations, to emplace a seal, and then to test the seal for hydrologic performance. The geologic and hydrologic characterization is necessary because seal performance relates to the performance of the surrounding interface and MPZ. The seal performance objective is to reduce flow under certain hydrologic conditions.

Seal Hydrologic Analysis--The intermediate-scale borehole seal tests entail similar considerations as those for testing in situ permeability, discussed previously. Testing could be performed using either transient or steady-state methods. The objectives of the test may be met if the flow satisfies a hydrologic performance goal established for the plugs. In performing an air- or water-flow test, it may be necessary to conduct the test at higher pressure gradients than would exist in the potential repository because the pore pressure cells should operate in a range of good responsiveness. The test performance objective for borehole seals to be satisfied under steady-state conditions might be

$$\frac{Q_t}{\Delta H} \leq C_r$$

where

Q_t = Total flow rate through a seal system parallel to the seal axis (MPZ, interface zone, and seal) during the test,

ΔH = Change in potential or head loss through the seal system during the test, and

C_r = Required conductance as determined from repository or seal design studies. Note that the required conductance is proportional to the product of the hydraulic conductivity and the seal area and inversely proportional to plug length.

Steady-State Flow--There are several potential flow paths for plugs emplaced in boreholes or shafts (Figure 4-7): the plug, the interface zone between the plug and the rock, the MPZ, and the surrounding undisturbed zone. The following steady-state and transient flow models assume parallel flow through the plug, the interface zone, and surrounding MPZ (in which the specific discharge is given by Darcy's law).

For steady-state flow, the total flow rate is given by

$$Q = \frac{(\sum K_i A_i) \Delta H}{L}$$

where

Q = Flow rate,

A_i = Cross-sectional areas for the plug, interface zone, and MPZ,

K_i = Conductivities for the plug, interface zone, and MPZ,

ΔH = Change in potential or head loss, and

L = Length of plug.

The same principle used for tracer tests may be applied to seals. If sections of the borehole above and below the seal are isolated (Figure 4-8) and steady-state flow is established, the minimum breakthrough time can be evaluated.

The approximate minimum travel time for a tracer to traverse the plug is given by:

$$\Delta t = \min \left[\frac{\phi_i}{K_i} \right] \left[\frac{L^2}{\Delta H} \right]$$

Figure 4-7. Flow Through Seal Components for Shafts and Boreholes

Figure 4-8. Assumed Tracer Test Setup

where

Δt = Minimum travel time, and

ϕ_i = Porosities for the plug, interface zone, and MPZ, and other terms are as defined previously.

The above relationships assume that the plug and MPZ are modeled as equivalent porous media. The interface zone is modeled by the cubic law for an equivalent smooth wall aperture in that the porosity equals 1.0. The above calculation of travel time assumes plug flow without hydrodynamic dispersion or diffusion effects.

For a very tight, unfractured tuff, it might take time for steady-state flow to develop or, if steady-state flow did develop, flow rates could be very small. Under these circumstances, a transient seal test may be performed (Figure 4-9). There are several alternative methods for conducting transient tests; the formula presented by Sutherland and Cave (1980) for pressure decay tests, originally developed by Brace et al. (1968) for laboratory testing of low-permeability granite, is the simplest:

$$\frac{\Delta P}{\Delta P_o} = e^{-\alpha t_d}$$

$$\alpha = \sum (K_i A_i) \frac{1}{LV_s \mu \beta}$$

where

t_d = Time to decay 90% of the pressure pulse,

ΔP = Difference in pressure between two reservoirs ($P_L - P_S$),

P_L = Pressure of upper larger reservoir,

P_S = Pressure of lower smaller reservoir,

ΔP_o = Initial difference in pressure between the reservoirs,

V_s = Volume of the smaller reservoir,

Figure 4-9. Transient Testing of Seals

- μ = Absolute viscosity of the fluid (air),
- β = Compressibility of the fluid (air),
- α = An exponential constant, and other terms are as defined previously.

Testing using the pressure decay method assumes the following major conditions prevail (Sutherland and Cave, 1980): (1) viscous one-dimensional flow with a linear pressure drop along the flow path length; (2) no capacity change in the flow regime during testing, i.e., the connected porosity of the specimen; and (3) no slip at the airflow channel interface. The experimental design of a transient test must consider many factors that need to be characterized to prevent experimental error. These factors include potential leakage paths away from the test area or to the formation, the compliance of the piping system, and temperature effects during pressurization. Assuming that experimental errors can be controlled, the above relationship indicates the conductance of the plug system could be quantified without measuring flow rates. For purposes of preliminary analysis, assuming further that the volume of the downstream reservoir is 10% of the volume of the plug, the time required to decay 90% of the pressure pulse is given by

$$t_d = \frac{-\ln(0.10)(.1L^2A\mu\beta)}{\sum (K_iA_i)}$$

where all terms are as defined previously.

4.2.2 Layout and Important Parameters

The layout is a central borehole surrounded by a series of site-characterization holes. Electrical resistivity and GPR methods are used to select and survey the test area of lower fracture intensity from the upper and lower drifts. Following these surveys, the series of site-characterization boreholes are located based on seismic tomographic and hydrological considerations.

Many electrode-array configurations are commonly available that can perform the vertical and horizontal resistivity surveys. The specific electrode arrays will be determined as part of the

actual test design. Periodic surveys for a given area may provide qualitative information with which to evaluate test performance.

Following drilling, the site-characterization holes will be logged using induction methods and crosshole seismic surveys will be performed. After the geophysical surveys, the central hole will be instrumented with a guarded straddle packer and tested using steady-state methods.³ Pressure response will be recorded in the surrounding observation holes using techniques suggested by Roberds et al. (1982). In welded tuff, the tests will account for anisotropic flow caused by the dominantly vertical jointing.

The experimental design tailors the small-scale hydrological test concept to the conditions encountered in welded and nonwelded tuff. In welded tuff, a bentonitic seal (Figure 4-6) is placed near the bottom of the hole. Above the zone, the grout bulb is placed extending into the rock. The extent of the grout bulb depends upon the degree of fracturing and the degree of overbreak occurring in the seal zone. After completion of grouting, the borehole is drilled out and characterized. Steady-state or falling-head tests are then conducted to measure the flow resistance of the grouted zone. The test may be conducted in progressive lifts to evaluate lengthening the seal.

In nonwelded tuff, the degree of fracturing is not as significant, and simple plugs are emplaced and evaluated in progressive lifts (Figure 4-5). Either transient or steady-state flow testing will be performed, depending upon the initial site characterization and expected seal tightness. A transient test requires the upper chamber to be pressurized and the pressure decay with time to be measured. Alternatively, a steady-state or falling-head test with either injection at a constant flow rate or injection of a constant volume determines the conductance of the seal system. After completing the testing for one lift, an additional lift is emplaced to lengthen the plug and increase flow resistance.

³Appendix B presents specific analyses for transient and steady-state flow. The nature of the welded tuff suggests that steady-state tests could be conducted within a reasonable time frame.

Measurement Methods--Measurements in the intermediate-scale borehole seal test are restricted primarily to the fluid-flow rates, fluid pressures, and tracer arrival times. The nature of the tests and the small operating diameter of the boreholes will not allow secondary seal performance measurements to be conducted without potentially influencing the test performance.

Pressure decay measurements in the transient tests and flow-volume rate measurements in the steady-state tests are expected to be monitored at the water inflow lines to the individual test boreholes. In the transient tests, a pressurization chamber will be configured to allow pressurization of the borehole seal. The requirements of the chamber will be defined during the final test design activities.

System pressures will be monitored in the chamber and water or air supply lines using pore pressure cells. Pressure transducers are available in a wide choice of operating ranges and sensitivities that could be employed for the tests. A high-accuracy, current-loop-output pressure transducer could be utilized. These particular transducers have sensitivities to 0.005% and very fast response times (milliseconds) and may also have wire runs of 300 m with no signal degradation. The pressure-sensor device in this instrument is a noncontacting optical pressure sensor that measures motion of an elastic member diaphragm. The instrument has an accurate and dependable design.

The steady-state test could be conducted either with a constant injection flow rate or as a falling-head test with the pressure changes monitored over time. Fluid volumes injected into the system will require accurate measurement over time. These flow rates could be measured with in-line or noninvasive flowmeters. Scoping calculations indicate flow rates of $1 \text{ m}^3/\text{s}$ to $10^{-7} \text{ m}^3/\text{s}$ ($\sim 16,000 \text{ gpm}$ to $\sim 10^{-3} \text{ gpm}$) could be experienced. The scoping calculations are presented in Appendix B, Modeling of Hydrological Performance of Shaft and Borehole Field Tests.

The flow rate of liquids is determined inferentially by measuring the liquid's velocity or the change in kinetic energy. Measuring a pipe's cross-sectional area and the average fluid velocity allows determination of the flow rate using the relation:

$$Q = VA$$

where

Q = Flow rate,

V = Fluid flow velocity, and

A = Cross-sectional area of the pipe.

Numerous flowmeters are available to measure liquid flow rates. Flowmeters considered applicable to the closed piping system anticipated for this test can be classified as differential pressure, positive displacement, velocity, and mass meters. The differential pressure devices include orifices, venturi tubes, flow nozzles, pitot tubes, and variable area meters. Positive displacement meters include piston, oval-gear, rotating-disk, and rotary-vane meters (O'Higgins, 1966). The positive displacement meters may be either mechanical or electrical. Velocity flowmeters can be represented by a turbine-type design. Mass meters are very accurate and specific to mass-related processes.

Flow rates in the upper estimated range ($1 \text{ m}^3/\text{s}$) for these tests would be best monitored by turbine flowmeters with multiple-blade, free-spinning impellers. These instruments are capable of monitoring at high-volume flow rates. Flow rates in the lower estimated range ($10^{-7} \text{ m}^3/\text{s}$) would be best monitored using accurate rotameter flowmeters, a differential pressure device capable of measuring rates at less than 1 cc/min . Flow rates in the intermediate range could be measured with a variety of instruments including the turbine or rotameter flowmeters. Instruments used in this intermediate range would have to be specified for the particular flow range expected. One other meter also appropriate for the intermediate flow ranges might be the orifice flowmeter. The orifice meters are commonly available and operate on the principle that the flow in the pipe is constricted; the pressure differential across the restriction is measured and then converted to an actual flow rate.

Piezometers, suction lysimeters, or tensiometers may be utilized in the test to monitor the advance of the saturation front. Figure 4-5 shows the anticipated configuration of the test and observation boreholes for nonwelded tuff. All instruments will be configured in observation boreholes to monitor a specific interval in each observation hole. The sampling points will be isolated and packed off from adjoining points.

Piezometers will be used near the test borehole and seal because flow in this area is expected to be saturated or reaching saturation over shorter intervals. At greater distances from the test borehole, flow conditions are expected to be unsaturated and suction lysimeters and tensiometers will be used.

Suction lysimeters are devices that consist of porous cups that can be evacuated to negative pressures. The device, suited to collecting moisture samples from unsaturated intervals, is commonly used in moisture studies in agricultural science. The tensiometers will be used to measure moisture available in the rock mass and indicate when saturation has been reached. Tracers may be included with the inflow water to indicate travel times and perhaps migration paths of the fluids. Tracers would be effective in conjunction with the piezometers and suction lysimeters.

Table 4-4 summarizes the instrumentation requirements for this test.

A data acquisition/recording system will be needed for the in situ measurements to be performed. The data acquisition system will be capable of automatic timed readings of all instruments. The data acquisition system must support the following minimum channel requirements for each test:

- 1 channel for flowmeter output.
- 2 channels for pressure transducer output.
- 24 channels for piezometer/suction lysimeter/tensiometer output.

Table 4-4. Summary of Gage Requirements--Intermediate-Scale Borehole Seal Tests

Instrument	Number of Tests	Gages per Test	Total Gages
Flowmeters	4	1	4
Pore Pressure Cells	4	2	8
Piezometers	4	8 ^{a,b}	32
Suction Lysimeters	4	8 ^{a,b}	32
Tensiometers	4	8 ^{a,b}	32
^a Piezometers, suction lysimeters, and tensiometers would be used in combination to total 24 instrument monitoring locations in each test. ^b These instruments may not be used in the fractured tuff environment depending on the degree of fracturing.			

4.2.3 Fielding of the Experiment

The nature of the intermediate-scale borehole seal tests requires that they be conducted in a portion of the test area that is free of influence from other tests and facility construction. Prior to fielding this test, the spacing of the test areas will be defined to avoid test-to-test interference. Proper spacing is required to restrict introduction of water from other seal testing that could influence observation of water flow paths in the intermediate-scale borehole seal tests.

Drift disturbance should be kept to a minimum prior to and during the test. Drift excavation, blasting, and drilling activity should be limited to reduce the potential of test-to-test interference and subsequently to enhance data interpretation. Stress redistributions in the area of the boreholes could have deleterious impacts on the seals, open otherwise tight fractures, or create new uncharacterized fractures. When the test area is characterized, no additional excavation activities should be conducted within the test envelope. The location of the drift should be selected so that unrelated activities do not interfere with the drift stability or the stress regime in the immediate area of the test.

Fielding of the experiment will include:

- Potential test location characterization through drift mapping, geophysical surveys, and observation boreholes. The observation boreholes will be located to be used as test observation holes if the site is selected for the intermediate-scale borehole seal tests.
- Stress characterization of the test location.
- Enlargement of the stress characterization borehole to be used as the seal test borehole.
- Placement of the piezometers, tensiometers, or suction lysimeters to provide for test observation.
- Installation of the seal at the selected interval.
- Test monitoring.

4.2.4 Miscellaneous Requirements

Field Quality-Control Measures--The miscellaneous requirements include laboratory testing, field quality-control testing, special facilities, and constraints on testing. The laboratory testing includes index properties similar to the small-scale in situ tests, with the addition of hydraulic conductivity determinations of the seal material and host rock. To assure quality control of the seal material emplacement, quality-control testing may be performed prior to and during emplacement of the cementitious material to ensure that the design specifications from the previous testing results are met. The quality-control tests were specified in Section 4.1.4 for the small-scale in situ test. The recommended quality-control laboratory and field tests for the cementitious material can include those tests defined for the small-scale in situ tests or those tests defined for the fracture grouting tests.

Special Facility Requirements--The special facilities were discussed previously. In addition, a grout batch plant will be necessary for blending material.

Constraints on Testing--The special facilities selected should reduce constraints on testing. A constraint on introducing water may be identified during site characterization activities.

4.3 Fracture Grouting Tests

Fracture grouting is the process of injecting fluids into fractures. These fluids (or grouts) typically involve cementitious, clay, and chemical grouts. Several major aspects of grouting include:

- Development of performance criteria.
- Selection of grout material.
- Design of the grout pattern.
 - Single versus multiple rows.
 - Layout of the grout curtain.
 - Hole depth.
 - Hole size and curtain size.
 - Spacing.
- Emplacement of the grout slurry into the fractured rock mass.
 - Pressures.
 - Injection times.
- Evaluation of grout performance.

These aspects of grouting are discussed below.

4.3.1 Development of Performance Criteria

Fluids can be categorized into two general areas: Newtonian and non-Newtonian fluids. If the shear stress is proportional to the rate of shear strain, the fluid is termed a Newtonian fluid. Fluids that do not obey this linear relationship are termed non-Newtonian fluids. In this latter category, there are the Bingham plastic and visco-plastic fluids. Figure 4-10 shows conceptual depictions of these fluids. The Bingham fluid model (Craft et al., 1962) can be described as

Figure 4-10. Types of Fluid Behavior

$$\tau = \tau_o + \nu \frac{d\nu}{dy}$$

where

$$\begin{aligned} \tau &= \text{Shear stress,} \\ \tau_o &= \text{Bingham yield stress (or cohesion in a plastic state),} \\ \nu &= \text{Dynamic viscosity, and} \\ \frac{d\nu}{dy} &= \text{shear rate.} \end{aligned}$$

For visco-plastic fluids, the viscosity of the fluid is dependent on the shearing rate. Stable grout slurries typically do not behave as Newtonian fluids, but as visco-plastic or Bingham fluids with well-defined yield points.

Deere and Lombardi (1985) suggested that viscosity is related to the rate at which the grout travels away from the grout hole and cohesion to the final distance of penetration. The following equations were presented:

$$R_{\max} = \frac{p_{\max} \cdot t}{C}$$

$$V_{\max} = \frac{2\pi p_{\max} \cdot t^3}{C^2}$$

$$F_{\max} = \frac{\pi p_{\max}^3 \cdot t^2}{3C^2}$$

where

- $R_{\max}$ = Maximum radius of penetration,
- $V_{\max}$ = Maximum volume of injected grout,
- $F_{\max}$ = Maximum total uplift force,
- $P_{\max}$ = Final applied pressure,
- t = Half-thickness of the joint, and
- C = Cohesion (Bingham yield point) of the grout slurry.

These equations show that greater grout penetration can be achieved by increasing the time and/or the pressure associated with the grouting operation.

General objectives in fracture grouting include emplacing the grout to achieve a reduction in fluid flow through selected geologic media and/or to achieve a strengthening of the rock mass. To ensure the grout can maintain acceptable durability over a specified time, the grout should be resistant to physical and chemical deterioration processes. As discussed by Deere (1982), the objectives of the grouting process are:

- To get the thickest possible grout into the cracks as easily as possible so as to reach the limit of penetration while the grout is still able to move freely and transmit pressure effectively.
- To restrict the volume of grout entering the job to only the quantity needed to fill any voids plus that subsequently needed to compensate for bleeding and leakage losses.

The benefits of a dense, stable grout are:

- Complete filling of the voids and joints.
- High mechanical strength.
- Good bond to rock.
- Resistance to physical and chemical deterioration.

- Predictability of the grouting process.
- Limited travel of the grout, thus avoiding useless losses (Deere and Lombardi, 1985).

Completely filling the fracture reduces the potential for dissolution of the grout as water enters between the grout and the fracture wall (Figure 4-11). This presumes that free-flowing water is present to dissolve the grout. When the intent is to restrict the airflow in the fractured rock, any reduction of the aperture will reduce the potential for flow.

A high-strength material in the fractures strengthens the rock mass and results in a greater resistance to deformation and failure. In some instances, a material that can deform without failing may be appropriate. To determine the required strength of grout placed in fractures in various rock openings (assuming the placement of a rigid material), several calculations were performed and are presented in Appendix C. The results of these calculations indicate that a conservative value for fracture cohesion (hardened state) for resisting thermal and in situ loads is 10 MPa.

A dense, more stable mixture would also be effective in resisting chemical deterioration. Typically, a dense mixture has a low porosity and hence a low permeability. This reduced permeability restricts the movement of permeants through the grout and restricts the potential for dissolution. Designing the grout from a geochemical viewpoint will also enhance the durability of the grout. Selection of suitable grouts is discussed subsequently in Section 4.3.2.

Although a dense grout is generally preferred, in some instances a thinner grout will probably be required to penetrate fine fissures. Initial criteria proposed for both thin and thick grouts include:

- Fluid properties.
 - Bleed: Less than 5% at 2 hours.
 - Viscosity: 30s (thin grout) to 60s (thick grout) using the Marsh funnel (10 to 40 cP [Deere, 1982]).
 - Set characteristics: Sets up rapidly.

Figure 4-11. Potential Separation Between Grout and Rock

- High cohesion (Bingham fluid yield point)⁴ for thick grouts (20 Pa) and lower cohesion for thinner grouts.
- Workability.
- Strength properties.
 - Acceptable minimum 28-day compressive strength of greater than 35 MPa.
- Composition.
 - Water-to-cement ratio by volume, 3:1 to 1:1 (by weight, 2:1 to 0.67:1).

4.3.2 Selection of Grout Material

Material Considered for Fracture Grouting--The three major types of grout are cement, clay, and chemical grouts. Cementitious grout has the advantage of imparting strength to the rock mass and can be erosion-resistant in its hardened condition. Because of its fluid nature immediately after mixing, it can be emplaced relatively easily before setting occurs. However, after hardening it can become brittle and can crack. Cementitious grout is considered a material with low self-healing capability.

Clay-based grouts usually have a higher self-healing capability than cementitious grouts. However, they are more susceptible to erosion and are not as strong as cement grouts. They can have a high swelling potential and high chemical instability.

Clays can be categorized in four major groups: the kandite group (especially kaolinite), the smectite group, the chlorite group, and the illites. These groups represent most of the extremes of the properties important for sealing (Meyer and Howard, 1983). The smectite-rich clays (especially bentonite, which contains largely the clay minerals montmorillonite and beidellite) have been used in grouting. Often they are used in small amounts (up to 8% by weight of the mixture) in cementitious grouts to control the bleeding of grouts. Smectite-rich clays are

⁴As illustrated in Figure 4-10, the Bingham fluid yield point is the shear stress at which the shear rate becomes large and the fluid yields.

also used as bentonite slurries in cutoff walls. Some recent research has been done on bentonite grouts (Pusch and Borgesson, 1989; Ran and Daemen, 1991). Ukrainian technology uses kaolinite clays in grouting operations.

Unlike the cementitious and clay-based grouts, chemical grouts do not contain particulates. Because of this characteristic, they are able to penetrate apertures smaller than those penetrated by clay or cementitious grouts. Chemical grouts can strengthen the geologic media into which they have penetrated and they can reduce the permeability of the grouted mass. Gnirk (1987) found that chemical grouts tend to shrink and the grouted zones tend to increase in permeability with time.

The YMP repository sealing program at SNL concentrates on selected cementitious and earthen-based material for sealing applications (Fernandez et al., 1987; Hinkebein and Fernandez, 1989). Specific material selection is discussed below.

Grouts Proposed for the Fracture Grouting Tests--In selecting potentially suitable grouts, it was necessary to achieve the performance objectives stated earlier. An additional criterion deals with the chemical durability of the grout. The development of a chemical criterion is discussed below.

Cementitious material are the primary material that have been considered for fracture grouting and borehole seal emplacement. The use of smectite clays (bentonite) is also being considered because it is a ubiquitous alteration product at Yucca Mountain, although it occurs in small quantities (Bish, 1988). There are two zones of abundant smectite: one is at the top of the vitric nonwelded base of the Tiva Canyon and contains 7% to 35% smectite, and the other is at the top of the basal vitrophyre of the Topopah Spring Member and contains 5% to 45% smectite (Bish and Vaniman, 1985).

Unlike the smectite clay, the cementitious material, including the grouts, are not indigenous to the volcanic tuffs. The SNL YMP repository sealing program developed cementitious material that are similar in bulk chemistry to the tuffs and contain enough reactive

silica present in the mixture to react with the calcium hydroxide to form a more stable calcium-silica-hydrate (C-S-H) (Licastro et al., 1990). Laboratory analyses conducted under the SNL YMP repository sealing program show the instability of ettringite, an expansive agent, above $\sim 100^{\circ}\text{C}$, but that tobermorite, a C-S-H, is present up to as high as 300°C (Scheetz and Roy, 1989). Based on these results, the strategy is to minimize the presence of ettringite and reduce the presence of calcium hydroxide by including enough silica to form a C-S-H such as tobermorite.

To gain a greater understanding of the chemical alteration of cementitious material, the computer code EQ3NR/EQ6 (Wolery, 1983) was used to simulate the geochemical alterations of three cementitious material in the presence of water and tuff (Hinkebein and Gardiner, 1991). Two of these material were variations of an ordinary portland cement (OPC)-based concrete and one was a variation of an ettringite-rich concrete. Chemically, the three concretes were described as an OPC-B concrete with a balance of silica and calcium, an OPC-C concrete that is calcium-rich, and an expansive portland cement (EPC)-S concrete that contains silica and ettringite. Hinkebein and Gardiner (1991) concluded that ettringite or excess portlandite decomposes to open the concrete structure and that excess silica reacts with hydrogarnet to tighten the concrete structure. Also, the analysis showed that OPC-B containing a balance of calcium and silica produces the least volume change. The analysis suggests the least modification to the overall structure and the resulting permeability for the OPC-B grout.

In these calculations, the C-S-H gel was modeled as 14 Å tobermorite, considered a reasonable substitute for the C-S-H gel because the thermodynamic differences between the behavior of C-S-H gel and tobermorite may be less than 1% (Atkinson et al., 1987). The information from the studies cited can be applied as follows:

- The development of a C-S-H gel was preferred to development of a more thermodynamically stable (i.e., less reactive) cementitious material.
- Tobermorite could be used to approximate the behavior of C-S-H gel.

- Geochemical analyses showed that balancing the calcium and the silica composition of a cementitious sample results in a material undergoing the least volumetric change.
- Material containing excess ettringite and calcium resulted in the most volumetric change.
- Tobermorite was observed at temperatures as high as 300°C.

Geochemical criteria for the formulation of cementitious material were developed for use in the sealing program. These criteria are presented below:

- Minimize the development of ettringite by selecting a cement that has a low sulfate content. (Ettringite is a sulfate-bearing cement phase in concrete.)
- Achieve a Ca/Si molar ratio of 0.83 or a CaO/SiO₂ mass ratio of 0.77 consistent with the composition of tobermorite, Ca₅Si₆O₁₇•10.5H₂O, by adding reactive silica.

An additional consideration in selecting grouting material is the aperture of the fractures that may need to be sealed. Cementitious grouts contain cement particles that could clog the fractures, resulting in limited or no penetration into the fractures.

Because of the potential for encountering fractures of several different apertures, four grouts are proposed for usage. For fine fissures with apertures of ~ 10 to ~ 100 μm, a microfine cement (MC-500) having an initial w/c ratio of 2:1 by weight is recommended. MC-500 is an inorganic grouting mixture composed of microfine particles, with 50% of the particles < 4 μm and the largest ~ 13 μm (Geochemical Corporation, 1990). The Ca/Si ratio for the commercial MC-500 microfine cement is ~ 1.53. To reduce this ratio to 0.83 as specified above, silica fines could be added to the mixture, but this would reduce the strength significantly. At this stage of the field testing, it is not proposed to add reactive silica. For apertures 100 μm to 1 mm, the w/c ratio should be halved to achieve a higher strength. For fissures with apertures between 1 mm and 1 cm, the use of a neat Class A portland cement grout is proposed. For fractures with apertures greater than 1 cm, the addition of silica sand is proposed. A small amount of aqua gel (a fine bentonite clay) is recommended to reduce the settling of the cement grains and the sand particles. The Ca/Si ratio can be adjusted by adding silica flour or slag.

To lower the cohesion (i.e., the Bingham fluid yield point) and thereby increase the final distance of penetration of the grout in the fracture, a dispersant (superplasticizer) should be added to the mixtures. The dispersant enhances the wetting ability of the water by reducing the interparticle attractive forces so that full wetting and mixing can occur.

4.3.3 Layout and Important Parameters

Two important aspects of the fracture grouting test layout are the general orientation of the test and the design of the grout pattern. Both aspects depend on the sealing component being considered and the following geotechnical considerations defined by Houlsby (1982):

- Spacing of the joints.
- Size of the joints.
- Direction of the open joints.
- Rock strength.
- Rock soundness.
- Rock stresses.
- Uniformity.
- Proneness to piping.

At this stage of field test planning, the most important geotechnical considerations are the spacing (and orientation) of the joints and the aperture of the joints. While there is a broad variation in the strike and dip of the joints at Yucca Mountain, there are two predominant joint sets that strike: N12°W and N34°E. Their fractures dip predominantly vertically or near vertically. These strikes and dips are assumed for planning purposes. Apertures are assumed to range from tens of microns to >1 cm. Because welded tuff is highly fractured, the grout is expected to be fairly uniformly emplaced around the injection hole, with some preferential emplacement along the direction of the strike if the fracture sets.

To develop the proper orientation for the field tests, types of sealing components will be considered. The current sealing data indicate emplacement of grout in fractured, welded tuff

(Tiva Canyon and Topopah Spring Members) and possibly in nonwelded tuff. Fracture grouting could be associated with the following specific sealing components: shaft and ramp seals, borehole seals, single embankments, and bulkheads at the repository horizon. Grout curtains associated with these sealing components can be vertically oriented, such as below a single embankment, or horizontally oriented, such as around a shaft seal. The field test program shall consider both orientations. To determine the effectiveness of the grouting operation, the following designs shall be evaluated: a single borehole, a single grout line, and a multiple-line grout curtain. The standard method for emplacing a grout line is proposed: first primary grout holes are grouted, then secondary grout holes are placed between the primary holes and grouted, and tertiary and quaternary holes are placed and grouted until the closure criteria are achieved (see Figure 4-12a). A multiple-line curtain is similar in concept, except multiple grout lines are proposed. A wide curtain will reduce the hydraulic gradients and piping potential (Deere, 1982). For a vertical grout curtain, the downstream line is placed first, the upstream line is placed next, and finally, the central line is placed. For a horizontal grout curtain, the grouting is completed downward in stages (see Figure 4-12b).

In the case of the multiple-line grout curtain, two designs are under consideration: a perpendicular-hole pattern, with holes perpendicular to the face of the shaft (Figure 4-13a), and a spiral-hole pattern, with holes slightly inclined to the face of the shaft (Figure 4-13b). The spiral design provides greater assurance that all fractures will be intercepted by a grout hole.

Several fracture grouting tests are proposed in the underground facility discussed in Section 3.1. Because grouts may migrate large distances depending upon the injection pressure, the aperture of the fracture, and the cohesion of the grout, the test locations should be carefully considered so that other tests areas are not impacted. Using the relationships developed by Lombardi and presented in Deere and Lombardi (1985), the maximum penetration distance can be determined as a function of injection pressure and cohesion, as illustrated in Figure 4-14.

The maximum injection pressure for fracture grouting depends on the state of in situ stress. The injection pressure should be less than that stress that would cause hydrofracturing. The rule-of-thumb for the maximum injection pressure is ~ 1 psi per foot of depth. If the test area

	<u>Order in Grouting</u>
P - Primary Holes	1st
S - Secondary Holes	2nd
T - Tertiary Holes	3rd
Q - Quaternary Holes	4th

(a)

(b)

Figure 4-12. Grouting Patterns: (a) Single Line, Cross-Sectional View and (b) Multiple-Line Grout Curtain in Shaft or Drift, Plan View

Figure 4-13. Plan Views of Grouting Pattern Around a Shaft: (a) Holes Perpendicular to Shaft Wall and (b) Spiral-Hole Pattern

Figure 4-14. Maximum Penetrations Into a Fracture as a Function of Injection Pressure, Cohesion, and Fracture Aperture

depth for fracture grouting is ~ 90 m, as currently proposed, then the estimated maximum penetration for an injection pressure of 300 psi (2 MPa) in a 10μ and a 100μ fracture would be 2 m and 20 m, respectively (Figure 4-14). This suggests the necessity to characterize fracture characteristics such as spacing, aperture, and orientation prior to field testing and the need to separate the field tests to avoid test-to-test interference.

Three types of fracture tests are proposed in the test facility described earlier. Locations in the underground test drift are indicated in Figure 4-15. These tests are listed below:

- Single borehole tests (Area 1).
- Single- and multiple-line grout tests to be located in the floor and ribs (Area 2 and optional Area 1).
- Multiple-line grouting shaft tests (Areas 3 and 4 and optional Area 5).
 - Holes perpendicular to the simulated shaft wall.
 - Spiral holes.

Performing these tests in multiple orientations will provide a good demonstration of the effectiveness of grouting rock from different orientations. While these tests are proposed for inclusion in the facility discussed in Section 3.1, these tests could easily be performed in other selected drifts if:

- Based on the site characterization process, a comprehensive testing program is deemed unnecessary, or
- Grout migration could affect other tests in the proposed test facility.

4.3.4 Fielding of the Experiment

Fielding of the experiment includes characterization of the grout and sequencing the tests. The characterization of the grout (i.e., the laboratory testing program and the field testing controls) is discussed in Section 4.3.5. The sequencing of the tests and the equipment setup is discussed below.

Plan View Upper Level

Plan View Lower Level

- Represents a 3.7-m-diameter shaft connecting UL and LL
- Represents LL relative position
- All dimensions reflect horizontal distances

Figure 4-15. Tentative Locations for Fracture Grouting Tests

An understanding of the geology and the hydrology involved is necessary to interpret the effectiveness of the grouting operation. Permeability testing will be carried out prior to any testing. Current practice is to use Lugeon water pressure tests to assess the permeability of the rock mass. However, the presence of water in the fractures could diminish the quality of the grout; therefore, most permeability testing will be performed with air. In selected circumstances, testing will be performed with water to establish a correlation between both methods of measuring permeability.

Sequencing of activities for each test shall be similar, as follows:

- Dry drill (HQ-size) the primary holes using rotary diamond drilling to maintain a smooth wall for future permeability testing and grouting.
- Conduct geophysical logs of the hole, including a detailed video log, to determine the location and frequency of fractures and the quality of the rock.
- Measure the permeability over the entire length of the holes using the guarded straddle packer system. Repetitive measurements (at least three) shall be made. These measurements will allow a qualitative estimate of the average fracture aperture if multiple fractures occur in a given test length.
- Perform packer grouting in ascending stages.
- Dry drill the secondary holes, conduct geophysical and video logs, measure the permeability over the entire length, and grout the holes.
- Repeat the process with tertiary and quaternary holes, if needed.
- Allow the grout to cure at least one month and carefully remove the grout from the holes.
- Visually inspect the borehole walls to assess the effectiveness of the grouting operation.
- Remeasure the permeability along the axes of the grout holes.
- Carefully extract grouted samples around the grouted holes to determine the effectiveness and extent of the grouting operation and measure the properties of the grouted rock mass.
- Determine the permeability along the axes of the newly drilled holes.

One standard setup for a grouting operation is illustrated in Figure 4-16. This setup includes (from Houlsby, 1990):

- A high-speed colloidal mixer and agitation tank, needed to maintain suspension and avoid settling and separation prior to grout emplacement.
- A positive-displacement grout pump in which the pressure can be maintained. In the U.S., the Moyno slurry system transport is often used because of its smooth and rapid delivery.
- A grout-circulation line to transport the grout to the grout hole and back to the agitator.

Grout is usually injected using a static-injection pressure that depends upon the extent of penetration desired. For single grout lines, one possible injection strategy is using a thick mixture for the primary and secondary holes and thinner mixtures for the tertiary and quaternary holes. Another possible strategy for single grout lines is starting with thin mixtures in all holes and thickening up when the pressure drops and the rate of pumping increases. For multiple grout lines, one possible injection strategy is using thicker mixes on the outer rows and a thinner grout on the internal row.

The STRIPA Project used another technique for emplacing grout. This "dynamic injection" technique applied vibrations of optimum amplitude and frequency to the grout in addition to the conventional static pressure (Borgesson and Pusch, 1989; Pusch et al., 1991). The benefit of this technique is its potential to penetrate fractures with apertures as low as 20 μm with a w/c ratio of 0.40 to 0.45. If it is determined that penetration into fine fissures is required, then this technique will be evaluated further.

4.3.5 Miscellaneous Requirements

Laboratory Testing--Laboratory testing of grout material is required prior to fielding the fracture grouting experiments. In a laboratory testing program, it is fundamentally important to define:

*For measuring water to the mixer

** For controlling the quantity of grout fed into the circulation line

Figure 4-16. Typical Equipment Layout for a Grouting Operation (Houlsby, 1990)

- Material used.
- Mixing, casting, and curing properties.
- Grout properties.

The cementitious material proposed for use in these experiments contain cement, reactive silica, nonreactive silica, superplasticizer, defoamer, and water.

As indicated earlier, several different cementitious grouts are proposed based on the potential for different apertures. Both fluid and hardened properties of these grouts should be determined. The properties that will be determined include:

- Fluid properties.
 - Dynamic viscosity.
 - Cohesion.
 - Bleed.
 - Setting time.
 - Density.
- Physical properties.
 - Porosity.
 - Density in hardened state.
 - Cement particle size.
- Strength properties.
 - Unconfined compressive strength (which represents a fundamental strength index).
 - Poisson's ratio.
 - Triaxial strength under a range of confining loads.
 - Shear strength of the grout.
 - Shear strength of the grouted fracture.
- Hydrologic properties.
 - Hydraulic conductivity.
 - Saturation state.

- Chemical characteristics.
 - Mineral composition.
 - Bulk chemistry.
 - Resistance to dissolution.

In addition to determining these properties, the effects of wet curing, high-humidity curing, and curing time on the properties could also be recorded. For destructive testing, the mode of failure shall be recorded.

Field Quality-Control Measures--Two areas require careful monitoring: the material properties as emplaced and the emplacement techniques. Three critical measurements would be performed for the grout slurry: temperature, density, and specific gravity, using the mud balance; and the apparent viscosity, using the Marsh and/or the ASTM (ASTM C 939-87, 1990) flow cone. As indicated by Deere (1982), measuring slurry density verifies cement content and provides a basis for mix duplication. Apparent viscosity is a good indicator of dynamic viscosity. The relationship between the dynamic viscosity and the apparent viscosity can be determined in the laboratory prior to fielding the experiment.

Because pressure is critical in determining the extent of penetration in the fractures and reaching closure, pressure monitoring is essential. Monitoring could be made at the point of injection and a record kept of time versus injection pressure. Accurate pressure measurements will require calibration of the flowmeter for each w/c ratio.

Special Facility Requirements--None.

Constraints for Testing--The grout holes would be drilled dry if it is determined that water intrusion into the fractures is detrimental to the quality of the "as-placed" grout. Assuming that each HQ-grout hole is 6 m in length and 27 grout holes are associated with each multiple-line grout hole, ~ 160 m of drilling would be required for each multiple-hole grout line. Additional post-characterization drilling would also be required.

4.4 Surface Backfill Tests

4.4.1 Objectives of the Test

The objective of the surface backfill tests is to support underground sealing tests by determining selected properties of rockfill placed in the repository drifts, understanding the extent of particle breakage, and evaluating the hydrologic performance of rockfill.

4.4.2 Regulatory Background

The current design basis for the YMP repository sealing program is to backfill the entire underground facility with general rockfill and use specialized sealing components in selected areas. Backfill in an underground facility is part of the engineered barrier system as defined in 10 CFR 60 (NRC, 1986a). In 10 CFR 60, general guidance is provided on the role that backfill takes in the underground facility. 10 CFR 60, Section 60.112 (NRC, 1986a) states that:

The geologic setting shall be selected and the engineered barrier system and the shafts, boreholes and their seals shall be designed to assure that releases of radioactive material to the accessible environment following permanent closure conform to such generally applicable environmental standards for radioactivity as may have been established by the Environmental Protection Agency with respect to both anticipated processes and events and unanticipated processes and events.

10 CFR 60, Section 60.133 (NRC, 1986a) addresses "Additional design criteria for the underground facility" and lists four criteria that can impact drift performance and, hence, backfill specification. Those criteria are:

- a). General criteria for the underground facility. (1) The orientation, geometry, layout, and depth of the underground facility, and the design of any engineered barriers that are part of the underground facility shall contribute to the containment and isolation of radionuclides.
- e). Underground openings in the underground facility shall be designed so that operations can be carried out safely and the retrievability option maintained.

- f). Rock excavation. The design of the underground facility shall incorporate excavation methods that will limit the potential for creating a preferential pathway for groundwater or radioactive waste migration to the accessible environment.
- h). Engineered barriers. Engineered barriers shall be designed to assist the geologic setting in meeting the performance objectives for the period following permanent closure.

In 40 CFR 191.12 (a) and 40 CFR 191.13 (NRC, 1986c), the following statements are made:

Disposal system means "any combination of engineered and natural barriers that isolate spent nuclear fuel or radioactive waste after disposal," and

Disposal system shall be designed to limit "cumulative releases of radionuclides to accessible environment for 10,000 years . . . "

The two important aspects of these criteria are that engineered barriers shall contribute to the containment and isolation, and that they shall assist the geologic setting in meeting the performance objectives. As additional geologic and hydrologic characteristics of the site are obtained, specific performance goals for the backfill will evolve. The following considerations will guide the development of specific performance goals:

- 10 CFR 60 indicates that backfill shall contribute to containment and isolation of radionuclides (NRC, 1986a).
- The Nuclear Waste Technical Review Board (NWTRB) believes that "well-engineered structures are less variable and more predictable than rock formations. Therefore, it should be possible to reduce overall uncertainty concerning the repository's long-term performance by relying on geologic barrier systems designed to retain waste material for thousands of years (NWTRB, 1990, 1991a, and 1991b)."
- The National Research Council recommended that flexibility be maintained in design of the repository features recognizing that variability in the geologic and hydrologic characteristics of the site would warrant the need to refine designs (National Research Council, 1990).

A broad field test program is proposed to evaluate the properties of backfill. This would allow flexibility in designing backfill for the variable conditions that may be encountered at Yucca Mountain.

4.4.3 Rockfill Placement Techniques and Properties

Background--Currently, information on the engineered properties of rockfill placed in underground openings is limited. In the mining industry, backfill is placed to attain underground structural stability to continue ore extraction. A common technique for backfilling in the mining industry is placement of hydraulic fill that contains a high water content. Use of hydraulic fill in the proposed repository is unacceptable due to the high water content. Therefore, experience gained from hydraulic fill emplacement is generally not useful to the YMP repository sealing program. Mechanical placement of rockfill mixed with cement by modified LHD units is also sometimes used (Brechtel et al., 1989) to attain high backfill strength. Use of cement in the backfill placed at the repository may be inappropriate due to the potential for elevating the pH of ground waters that could then contact waste packages. However, placement of cemented backfill at other locations, such as the Calico Hills unit, may be appropriate. The Brechtel (1989) study indicates the feasibility of emplacing backfill using mechanical placement techniques. Another technique used to backfill mines is pneumatic stowing (Reynolds, 1972). This technique may be appropriate for backfilling the potential repository due to the placement equipment mobility, its ability to transport material through a lengthy pipeline, its safety and speed of operation, and its ability to transport dry material (Maksimovic and Lipscomb, 1982; Roberts and Masullo, 1985). However, the potential for excessive wear inside the transport pipes due to the abrasive nature of the welded tuff and questions about its ability to achieve the necessary engineered properties are additional concerns. While other techniques such as hydraulic filling are available to fill mines, in almost all cases only the very basic engineering properties have been determined.

In civil applications, the predominant role of backfill is to develop earthen and rockfill dams. While the properties of the material used to build dams have been determined through large-scale laboratory testing and some field test programs, the placement techniques and material used in unconfined space for dams differ from a repository application. However, information

obtained from evaluating rockfill in large-scale laboratory testing apparatus and the limited information on placement techniques for essentially dry material used in the mining industry will be useful in defining a field test program.

Therefore, the intent of the backfill field test program is to obtain sufficient information on the properties of rockfill utilizing existing placement technologies to enable engineering of backfill for specific repository conditions.

General Rockfill Properties--Shear strength and compressibility are the two most important factors in designing rockfill structures (Marachi et al., 1972). Most investigations of rockfill consider variables, such as relative density, gradation of the rockfill, dominant size of rock particles, particle shape, degree of saturation, and confining pressure, that can affect these factors. Results of some of these investigations are presented below.

Through laboratory studies (Leps, 1970; Marachi et al., 1972; Marsal, 1973; Donaghe and Cohen, 1978; Charles and Watts, 1980; and Charles and Soares, 1984), fundamental properties relationships were established for numerous rock types. These studies provide direction for some of the more important variables to be considered in evaluating rock strength. Some of the results are summarized below.

- In general, compacting rockfill increases shear strength and decreases compressibility and permeability.
- The angle of internal friction provides a measure of shear strength. The angle of internal friction shows a decrease at a decreasing rate with increasing confining pressure. Shear strength increases with increasing coefficient of uniformity, increasing relative density, and increasing compressive strength of the intact rock.
- For large hard rock (similar to welded tuff), particle breakage occurs by splitting and breaking off edges and corners. Softer rock is reduced mainly by crushing.
- Clean, hard, angular, quarried basalt possesses higher strength than well-rounded gravel (Marsal, 1973).
- Saturation of the rockfill generally results in greater consolidation and greater strains.

- At very low confining pressures, dilation of rockfill occurs; at higher confining pressures, compression increases to some limiting value.

The results above are cited to show some of the relationships established in the past for rockfill. Additional investigations are needed to determine the properties of crushed tuff to support analyses evaluating the response of the tuff in the confined environment of the potential repository.

Particle Breakage and Placement Techniques--As indicated above, two important objectives of this testing are an understanding of the material properties of the as-placed rockfill and the hydrologic response of the material. Surface backfill testing will address the movement of water and fines through the backfill. Both water and fines movement depend on the gradation of the material, which depends in turn on the rock type and the manner in which the material is handled and emplaced. The gradation as mined from the facility may be within the range shown in Figure 4-17. Modifications to the material can occur as part of the material handling and emplacement process. This process is shown in Figure 4-18. During the material handling and emplacement process, particle breakage that can affect the properties of the rockfill occurs. The degree of this breakage cannot be accurately predicted without performing placement tests and in situ measurements. Incorporating actual handling procedures in the test will allow an accurate determination of the fines that can be created. Knowing the nature and the degree of the fines development will allow definition of the initial gradation at an appropriately coarser level to compensate for particle breakdown. Quantifying the effect of this particle breakage is therefore necessary.

To determine the influence of particle breakage, it will be important to utilize large-scale operations for the placement of rockfill. Because backfilling is a large-scale operation, determinations of particular importance are sizing of material and the requirements of a backfill plant to properly grade and mix the material and the fill-placement operation.

Because the emplacement operation will influence the particle breakage and the properties of the rockfill through the compactive force used, several emplacement methods are proposed for testing. These techniques include:

Figure 4-17. Gradation of Crushed and Mined Tuff--A Comparison

Figure 4-18. Material Flow Diagram for Material Preparation and Handling of Backfill

- Mechanical emplacement and compaction by rubber-tired vehicles (see Figure 4-19a).
- Throwing methods, such as pneumatic stowing (see Figure 4-19b) and the slinger method (see Figure 4-19c).
- A combination of these methods (Figure 4-19d).

In addition to the emplacement techniques, rockfill properties also depend on the rock itself. Therefore, both nonwelded and welded tuff will be considered in the testing program.

4.4.4 Description of the Test

The proposed testing program will address the following factors that can affect the gradation and, subsequently, the properties of the rockfill:

- Emplacement techniques.
 - Mechanical compaction with rockfill placement in lifts and with the rockfill at the crown pushed into place.
 - Pneumatic stowing in the entire drift.
 - Combination of mechanical compaction and pneumatic stowing.
- Rock types.
 - Welded tuff.
 - Nonwelded tuff.

Categorization of Tests--Three factors, emplacement technique, rock type, and the hydrologic response of backfill, provide the framework for testing in the surface backfill facility. The three categories of testing proposed in the surface facility include:

Category 1: Partial Drift Emplacement (unsaturated)

Variables considered:

- Lift thickness.

A. Mechanical Compaction

B. Pneumatic Stowing

C. Truck Mounted Belt Slinger

D. Combination of Mechanical Compaction and Throwing Methods

Figure 4-19. Techniques for Placing Backfill

- Emplacement technique.
 - Rubber-tire compaction.
 - Vibratory compaction.
- Rock types.
 - Nonwelded tuff.
 - Welded tuff.

Category 2: Full Drift Emplacement (unsaturated)

Variables considered:

- Emplacement technique.
 - Mechanical compaction.
 - Pneumatic compaction.
 - Combination of mechanical and pneumatic compaction.
- Rock type.
 - Nonwelded tuff.
 - Welded tuff.

Category 3: Hydrologic Testing

Variable considered:

- Permeability of rockfill.

Because of the repetitive testing of this material, a convenient surface facility to conduct this work is proposed. This surface test area can be a tunnel excavated in rock or a facility especially constructed for repetitive testing. Such a facility presents advantages and disadvantages, as listed below.

- Advantages.
 - Monitoring ports with easy access.
 - Minimizing the impact of instrument presence on the placement operations by diverting leads to monitoring ports.
 - Ability to simulate varying hydraulic conductivities at the base of the emplacement area.

- Disadvantages.
 - May be more costly to construct than developing a full-scale tunnel.
 - May not replicate the surface of a mined tunnel in welded tuff.

Further considerations include the comparative cost to construct a surface facility versus a tunnel complex and the availability of a tunnel complex.

4.4.5 Layout and Important Parameters

Testing can be performed either in a tunnel or a specially constructed facility. A testing area ~25 m long is proposed for conducting the tests.

Category 1 Testing--For category 1 testing, material would be placed in lifts of variable thicknesses. This category of testing is intended to determine the rockfill density as a function of compactive force used to emplace the rockfill, the moisture content, the thicknesses of the lifts, and the gradation of the material. Therefore, the parameters to be measured include the moisture content, density, gradation, and pressure in the backfill. The instruments to be used in the tests are hydraulic pressure cells (or flatjacks) and earth pressure cells to measure the pressure in the fill during the placement process. Extensive samples would be taken to measure the moisture content and the gradation and to compute density after all layers were placed. The in situ density and moisture content would be measured using portable field instrumentation. The proposed location of the instruments and the sampling locations are indicated on Figure 4-20.

Category 2 Testing--For category 2 testing, material would be placed within the entire drift. Category 1 testing results would be used to determine the preferred mechanical compaction technique. The effectiveness of both mechanical compaction, pneumatic stowing, and a combination of both would be investigated. The parameters of interest are similar to those proposed for category 1 testing. In addition to the instruments used in category 1 testing, settlement gages would also be placed into drifts to ascertain the settlement at the roof of the drift. The instrumentation and sampling locations for this category of proposed tests are illustrated in Figures 4-21a and 21b.

Sampling Grid for In Situ Moisture/Density Measurements and Laboratory Measurement of Moisture Content and Gradation (4 layers assumed)

Figure 4-20. Instrument and Sampling Locations to Evaluate Particle Breakage Due to Mechanical Compaction--Partial Drift
Emplacement (category 1 testing)

(A) Station Location for Instrument and Sampling Locations

★ Preferred Number and Thickness of Lifts Selected From Category 1 Testing (5 layers assumed for field density and moisture content measurements)

(a) Mechanical Compaction

α - Additional Sample Extraction and In Situ Moisture/Density Determination (2 sampling points per level)

(b) Pneumatic Stowing

Figure 4-21. Instrument and Sampling Locations for (a) Mechanical Compaction and (b) Pneumatic Stowing--Full Drift Emplacement (category 2 testing)

Instrument and Sampling Locations at Cross-Section B-B' for Mechanical Compaction

+ - Hydraulic Pressure Cell and Sampling for Gradation and Moisture Content

□ - Earth Pressure Cell

T - Settlement Gage

Instrument and Sampling Locations at Cross-Section B-B' for Pneumatic Stowing

Category 3 Testing (Optional)--One of the preferred sealing concepts is the placement of a seepage control structure (see Section 4.6) in a drift to control lateral movement of water in drifts. If a significant amount of water is encountered in the repository, water control may be necessary. If a seepage control structure is determined to be an important part of the repository postclosure design, this sealing concept will be tested.

A prototype test is proposed in a surface facility to understand the hydrologic response of rockfill and to develop procedures for the placement of instruments. Using the preferred technique for compacting rockfill from categories 1 and 2 testing, two different rockfills and the associated instrumentation would be placed as illustrated in Figure 4-22. Water would be introduced into the left portion of the modeled area and the saturation front monitored.

Table 4-5 lists the surface backfill tests proposed. The parameters of interest include the pressure in the backfill, the moisture content, the density, the gradation, and the settlement at the roof for the category 2 and category 3 tests. The parameters of interest can be measured as indicated in Table 4-6. The number of channels and the samples required for each test are presented in Table 4-7.

4.4.6 Fielding of the Experiment

As indicated above, a facility of ~25 m in length is proposed for the surface backfill tests. Prior to initiating the field tests, a laboratory testing program should be initiated to determine fundamental relationships, such as compactive force versus density for a given gradation of welded and nonwelded tuff. Several tests are proposed to obtain an understanding of the as-placed properties of rockfill. These tests will be conducted in essentially the same sequencing of operations as given below.

Fielding of the experiment includes:

- Particle preparation including washing, sieving, and blending the crushed material to a suitable gradation.

Legend

- Horizontal Hole for Gamma-Gamma Log, Neutron Log, and Settlement Profile
- Vertical Hole for Piezometer
- Pore Pressure Cell

Scale

Figure 4-22. Geophysical Logging and Instrument Ports for the Surface Backfill Test (category 3 testing)

Table 4-5. Proposed Surface Backfill Tests to Evaluate Rockfill Performance at the Surface Facility

Test No.	Description of Test ^a
	<u>Category 1</u>
1	Lift thickness "a" Mechanical compaction Nonwelded tuff
2	Lift thickness "b" Mechanical compaction Welded tuff
	<u>Category 2</u>
3	Lift thickness "a" Mechanical compaction Nonwelded tuff
4	Lift thickness "b" Mechanical compaction Welded tuff
5	Pneumatic stowing Nonwelded tuff
6	Pneumatic stowing Welded tuff
7	Combination--mechanical and pneumatic stowing Lift thickness "a" Nonwelded tuff
8	Combination--mechanical and pneumatic stowing Lift thickness "b" Welded tuff
	<u>Category 3</u>
9	Welded tuff--hydraulic conductivity "x" Welded tuff--hydraulic conductivity "y"
^a Note that "a" and "b" refer to variable thicknesses and "x" and "y" refer to two different conductivities that will be determined by future planning efforts.	

Table 4-6. Parameters to be Measured and Techniques Employed

Parameter	Description
Earth Pressure	Direct monitoring during emplacement operations by earth pressure cells and hydraulic pressure cells.
Laboratory Gradation	Laboratory measurements from samples extracted from the field following placement operations (ASTM D 422-63, "Standard Method for Particle Size Analysis of Soils," or equivalent method for a coarse material).
Laboratory Moisture Content	Laboratory measurements from samples extracted from the field following placement operations (ASTM D 2216-80, "Standard Method for Laboratory Determination of Water [Moisture] Content of Soil, Rock, and Soil-Aggregate Mixtures").
Field Density and Moisture Content	In situ measurement at time of placement and during extraction of samples used for laboratory analyses (ASTM D 2922-81, "Standard Test Methods for Density of Soil and Soil-Aggregate in Place By Nuclear Methods [Shallow Depth]," and ASTM D 3017-88, "Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods [Shallow Depth]").
Settlement	Direct monitoring following placement operations.

Table 4-7. Channels and Rockfill Samples Required

Test No.	Parameter of Interest	Channels or Samples Required (per test)
1 and 2	Earth pressure	30
	Laboratory gradation	39
	Laboratory moisture content	39
	Field density and moisture content	60
3 and 4	Earth pressure	52
	Laboratory gradation	33
	Laboratory moisture content	33
	Field density and moisture content	75
	Settlement	3
5 and 6	Earth pressure	39
	Laboratory gradation	24
	Laboratory moisture content	24
	Field density and moisture content	42
	Settlement	3
7 and 8	Earth pressure	52
	Laboratory gradation	33
	Laboratory moisture content	33
	Field density and moisture content	75
	Settlement	3
9	Density, moisture, and settlement profiles	24
	Piezometer measurements	27
	Pore water pressure	8

- Transporting the blended material to the surface facility.
- Placing the fill in the drift using a LHD, a modified LHD, and/or pneumatic stowing. A pneumatic conveying system consists of a power supply, an air supply, an infeed arrangement, a pipeline, and a discharge pipe (Maksimovic and Lipscomb, 1982; Powell, 1982). Placement equipment is available from commercial vendors.
- Emplacing the instrumentation concurrently with the placement of the fill.
- Monitoring performance.
- After the test, characterizing the backfill following excavation.

4.4.7 Miscellaneous Requirements

Laboratory Testing--To support the field tests, laboratory analyses will be performed prior to and following the tests. Analyses performed prior to the test will provide welded and nonwelded tuff rockfill properties. Strength and compressibility are the most important factors in the design of the rockfill. Therefore, variables that affect these factors should be measured. In addition, saturated-unsaturated versus suction head properties should also be determined. The properties to be measured prior to testing tuff particles and several gradations of rockfill include:

- Strength.
 - Internal friction angles over a range of σ_1/σ_3 values.
 - Unconfined compressive strength, which represents a fundamental property of the intact rock.
 - Particle breakage due to a change in the state of stress.
 - Shear strength over a range of confining pressures.
 - Confined comprehensive strength over the range of stress (σ_1/σ_3).
- Compressibility.
 - Volumetric and axial strain over a range of σ_1/σ_3 values.
 - "e-log p" curve including rebound effects.
 - Compaction due to single and repeated vibratory motions.
- Physical properties.
 - Gradational analyses.
 - Specific gravity.
 - Scratch hardness.
 - Shape factor.

- Particle breakage.
- Porosity.
- Density of the rockfill.
- Dry density and moisture content of the minus 4 sieve material.
- Hydrologic properties.
 - Saturated.
 - Unsaturated versus pressure head.

Following the placement of the rockfill, the primary laboratory analyses will measure gradations, moisture contents, and densities.

Field Quality-Control Measures--Correlations will be made of in situ and laboratory measurements of the density and the moisture contents of the rockfill. The checks will be performed at the same locations. The in situ measurements can be made with a portable device according to the standards indicated in Table A-6.

Special Facility Requirements--The placement of large quantities of rockfill will require the ability to sieve and mix the rockfill. Therefore, a processing plant will be required to handle the quantities of rockfill being placed.

To characterize the large number of samples extracted following the tests and to provide quality control on the mixture of the rockfill at the site, a laboratory testing room is recommended to measure the gradations, moisture contents, and the weight. The quantity of large volume samples will require a storage area for the samples while they are being processed. Samples will be handled and the chain-of-custody documented according to DOE Quality Assurance requirements.

Constraints on Testing--The primary constraint before testing can begin is a thorough laboratory testing program that will better concentrate the field testing efforts in selecting the appropriate gradation and the suitable thickness for the lifts. In addition, preliminary performance criteria should be developed before field testing and laboratory testing is performed.

4.5 Grouted Rock Mass Tests

The grouted rock mass tests includes a series of tests on an isolated block of tuff instrumented with various stress-strain, temperature, and deformation gages to evaluate the mechanical hydrologic properties of a typical planar joint. The properties of the joint will be evaluated under grouted and ungrouted conditions at ambient and elevated temperatures and stress, as in previous studies (Case and Kelsall, 1987). Hydrologic performance of the joint will also be observed during heated and unheated phases of the test. The tests are designed to control the stress and temperature boundary conditions of the rock block. The tests will be closely fashioned after similar tests performed at the NTS, G-Tunnel facility by Zimmerman et al. (1984). Fracture properties at Yucca Mountain are assumed to be similar to those at G-Tunnel. The fractures will be identified through detailed mapping and the four flatjacks will be grouted at the edges of the test block to simulate an impermeable flow boundary condition.

4.5.1 Description of the Test

Objective of the Test--The objective of the grouted rock mass tests is to evaluate the thermomechanical and hydrologic properties of a typical joint in welded tuff. Hydraulic conductivity measurements will be performed along the joint surface at various temperatures when the joint is both ungrouted and grouted. The measurements will help to characterize the effectiveness of the grouting, as in previous jointed block testing. The test layout is provided in Figure 4-23.

Welded Tuff Block Isolation--The grouted rock mass test will be performed in a dedicated alcove in the SCTF area of the ESF. The alcove will be excavated to limit the extent of disturbance around the excavation. Once excavated, the floor will be cleared and the area will be fracture mapped to identify suitable planar fractures for testing. If deemed suitable for the geologic medium, cross-borehole seismic tomography will be incorporated to define the fracture patterns in the alcove. Based on these investigations, a single fracture that is nearly vertical and continuous over the area of the potential block will be selected. The block will be defined based on the location of a fracture for the permeability test. Necessary slots will then be cut.

Figure 4-23. Grouted Rock Mass Test Layout

Stress Conditions--Three-m-deep slots cut into the alcove floor will define the horizontal dimensions of the block. These slots are expected to be ~2 m on a side. Surface deformations due to slot cutting and stress relaxation will be measured to determine the stress relief. Flatjacks will be grouted into the slots to reload the block to the previous state of in situ normal stress. The flatjacks will be hydraulically operated and maintained at a constant pressure when evaluating the thermal expansion of the rock mass.

Thermal Loading--Heater holes in two opposing rows parallel to the flatjacks will provide directional heat fluxes on the isolated block. It has been shown by Zimmerman et al. (1985b) that a nearly unidirectional heat flux is created with this heater arrangement. Approximately ten heater elements will be incorporated in the test. Depending on test conditions, continuous power and separate diesel generators may be necessary.

Hydraulic Parameters--Boreholes for hydrologic data collection will be drilled along the trace of the selected fracture. They will provide access for collecting data on the thermomechanical conditions created in the test and on the hydrologic performance. This information will be used to assess the performance of the grouting system by measuring the permeability changes due to grout and thermal loads. The hydrologic data holes will consist of three boreholes along the trace of the fracture; one will be packed off and used for the injection interval and the remaining two will be observation holes for fluid arrival detection.

Fracture permeability will be determined for the temperature and pressure conditions created in the test. It will be determined whether the permeabilities are sensitive to the effects of stress and temperature and if that sensitivity becomes significant in reducing fluid flow in the fractures.

Grouting Effects--The grouted rock mass tests will be performed in two identical phases: the first phase under elevated temperatures in an untreated fracture and a second phase with the fracture grouted. The fracture will be grouted through a diagonal borehole penetrating the fracture at some depth. Grouting will be conducted under the pressures and methods determined

in other sealing tests. Permeability tests will be conducted in the same borehole set created for the first phase, but may require redrilling of the observation boreholes if grout has intercepted the borehole surfaces.

4.5.2 Layout and Important Parameters

The present proposed layout conducts the two phases of testing in a single isolated block. The first phase will examine the hydrologic performance of the joint under successively higher temperatures and stresses. From this information, a relationship for permeability versus temperature or pressure will be developed for a single fracture. The second phase will reevaluate the same fracture after grouting under similar temperature and stress conditions.

The range of variation will be used to estimate sensitivities of the fractured welded tuff to flow scenarios in performance assessment studies. Work conducted by Zimmerman et al. (1985a) suggests single-fracture permeabilities are relatively insensitive to stress and slightly sensitive to temperature increases under representative in situ conditions. They measured a range of fracture permeability from 49 to $956 \times 10^{-8} \text{ cm}^2$; however, grouting of the fracture was not examined in previous studies.

Performance Objective and Supporting Analysis--Zimmerman et al. (1985b) and Case and Kelsall (1987) determined that the permeabilities along the two flow paths measured in the heated block test varied considerably between loaded and unloaded conditions. It appeared the fractures did not open uniformly and permeability was relatively insensitive to stress changes. The fracture permeability was slightly reduced at higher temperatures. Those studies determined that, at the higher temperatures, the fracture asperities were more compatible and therefore a reduction in permeability was observed. This is similar to the conclusion arrived at by Hardin et al. (1982) from a block test performed in granite.

The current examination expands the tests to a grouted phase to evaluate the addition of grout on the performance of the fracture under controlled thermal and stress conditions. The

performance of the grouted fracture may be affected by the rise in temperature, development of thermal stress, and potential differences in thermal expansion between the grout and the rock.

Permeability measurements are taken by injecting known amounts of water into the middle borehole; the amounts collected in the observation boreholes are then a function of the permeability of the discrete fracture. Smooth-wall fracture aperture theory will be used in the calculation. The intrinsic fracture permeability is defined by the equation (Zimmerman et al., 1985b):

$$k = \frac{e^2}{12}$$

where

k = Intrinsic permeability, and

e = Smooth-wall fracture aperture.

The flow rate occurring over fracture length will be plotted against the pressure gradient. The slope of this line is the product ke . If the slope is constant, the fracture is not deforming with pressure and the aperture and fracture permeability can be determined. Examination of these plots versus temperature, flatjack pressures, and grout application may provide the necessary input for the test performance assessment.

Parameters to be Measured--The following parameters are to be measured in the heated block performance assessment test:

- Geomechanical measurements.
 - Strain relief in the block resulting from slot cutting.
 - Thermomechanical behavior of the block during heating phases.
 - Flatjack pressures.

- Thermal measurements.
 - Heater temperatures.
 - Block temperatures at the fracture location.
 - Injection fluid temperatures.
- Hydrologic measurements.
 - Injection fluid flow rates.
 - Injection interval fluid pressures.
 - Fluid accumulation at the observation boreholes.

Measurement Methods--The parameters to be measured during the grouted rock mass tests will concentrate primarily on the hydrologic performance of the fracture under ungrouted and grouted conditions. Strain measurements will be observed in the rock mass using rigid metal pins located at oblique orientations on the surface of the block. Using a caliper, the exact pin distances will be measured before and after slot excavation. The resulting strain deformation will provide data to calculate the stress relaxation that occurred.

Thermocouple locations are planned at the heater elements and at various locations within the isolated block. Temperatures of the fluids used in the hydrologic testing will also be monitored. Thermocouples identified in the small-scale in situ tests (Section 4.1.2) will be used. The temperature range of the tests will be selected to represent the expected repository conditions. It is anticipated that 13 thermocouples will be employed in the test.

Stresses will be measured directly in the heated block test using a borehole deformation gage. One borehole gage will be located within the isolated block itself and a second may be employed in the undisturbed rock mass but within the heated field. Strains will be measured at the block surface with the previously emplaced rigid pins and correlated to the measured stresses. Strains will be manually read during the test performance. Pressures maintained in the flatjacks will also be monitored. A summary of instrumentation requirements is presented in Table 4-8.

Table 4-8. Summary of Gage Requirements--Grouted Rock Mass Test

Instrument	Number of Tests	Gages per Test	Total Gages
Strain Relief Pins	2	1	2 ^a
Borehole Deformation Gage	2	2	2 ^b
Thermocouples	2	13	13 ^b
Flowmeters	2	1	1 ^b
Pore Pressure Cells	2	2	2 ^b
^a One set of strain relief pins will be used. They will measure relaxation of the isolated block after slot cutting and possibly for verification a second time when flatjacks are used to restress the block. ^b One instrument set may be used before and after grout injection. Separate instruments are not required.			

A data acquisition/recording system will be needed in some instances for the in situ measurements to be performed. The system will be capable of automatic timed readings of all instruments. It will perform any necessary preprocessing or conversion of signals prior to recording the digital information. It will be capable of storing and transferring data as required for analysis and reporting. The polling devices may be a combination of specific devices that monitor one test instrument or centralized systems that will concurrently poll and record a number of tests. Duplication will be limited to that required for efficient collection of the measurements and as necessary for system redundancy and backup.

The system is likely to include the following components (Roberds et al., 1983):

- A CPU that will control the logic of the system and the measurement and recording devices.
- An analog to digital converter that will convert voltages to digital signals.

- A multiplexer that will allow coupling of numerous input and output channels to the central unit.
- Data storage devices to maintain the data temporarily.

The data acquisition and recording system will need to support the following minimum channel requirements (see Table 4-8) for each test:

- 13 channels for temperature output.
- 6 channels for borehole deformation gage (3 per gage) output.
- 2 channels for pressure transducer output.
- 1 channel for flowmeter output.

Monitoring frequency will be refined during the detailed laboratory testing for specific seal mixtures. In general, the monitoring frequency will be consistent throughout the duration of the tests.

4.5.3 Fielding of the Experiment

The nature of the grouted rock mass tests does not require that they be conducted in a dedicated drift or alcove. However, if performed with other tests, it would be useful to be sequenced as the first series of tests in a given alcove. The stress field should be measured in the initial stages and an effort should be made to reduce the perturbation of the natural in situ stresses to those created in the excavation of the alcove. Drilling of additional boreholes, particularly during the test performance, will interfere with the test results.

The test is intended to be conducted in the welded tuff area of the ESF, where regular fracturing and joint patterns are expected.

The construction sequence for performing the test is as follows:

- Conduct fracture mapping and geophysical logging to isolate the fracture to be tested.

- Install rock instrumentation near the fracture.
- Saw four slots to a controlled depth of 3 m and measure dilation of the fracture system.
- Install flatjacks, heaters, and thermocouple instrumentation.
- Drill injection and observation holes along the discrete fracture. Install packer and pumps for injecting water into the discrete fracture system.
- Conduct hydrologic testing at ambient temperature at preselected stress levels. Elevate temperature and conduct test at same stress levels.

4.5.4 Miscellaneous Requirements

The miscellaneous requirements include laboratory testing, field control testing, special facility requirements, and constraints. Each of the requirements would be similar to those for the fracture grouting test.

Constraint on Testing--During site characterization activities, a design constraint will identify how much water can be introduced to the underground facility. The design constraint will be imposed on the test.

4.6 Seepage Control Tests

4.6.1 Objectives of the Test

The objectives of this test are to understand the hydrologic performance of drift fill and drainage through the underlying fractured rock. It is also intended to provide an understanding of the potential migration of fines and clogging of the underlying fractures. The test is intended to be performed in the fractured welded tuff of the Tiva Canyon and Topopah Spring Members.

4.6.2 Background

Sealing concepts for the YMP consider the radionuclide release by water. In developing the concepts, both small and large amounts of water inflow were considered. The primary response to water inflow is to isolate and drain the water at or near the origin, as illustrated in Figure 4-24. This may be particularly effective for smaller flows. For larger water inflows, large sealing components to control the lateral migration of the flow in the underground are proposed. Larger amounts of water will be channeled to areas removed from waste disposal areas and drained into the fractured rock. To achieve this drainage, a single embankment structure was originally proposed (Fernandez and Freshley, 1984) and subsequently modified (Fernandez et al., 1987), as shown in Figure 4-25a. More recent refinements of the original concept are illustrated in Figures 4-25b, c, and d. In Figure 4-25b, a closer analogy to the construction of surface dams was provided by placing a clay barrier on the upstream side of the dam. The concept was further modified to consider both the ease of constructing the single embankment upon retreat from the repository and the potential migration of fines within the structure. This modification resulted in only one inclined surface comprised of several layered soils that represent a graded filter designed to prevent piping and instability of the embankment. Given that a high hydraulic head and a constant supply of water would be necessary to destabilize the single embankment, the need for a filter design, as shown in Figure 4-25c, was considered highly unlikely. This resulted in the design of a single inclined surface defined as the point of contact between a coarse backfill and a backfill having a lower permeability than the coarse backfill. Because this concept is simple to construct and may control the lateral migration of anticipated and unanticipated waters, it is proposed for field testing.

4.6.3 Backfill Performance

It is important to understand the response of backfill to water. There are two particular areas of interest: the lateral migration of water flow in a backfilled drift and the migration of fines within the backfill. Both areas are discussed below.

Benefits

- Simple way of controlling small drift inflow
- Increase drainage and storage capacity of floor

Figure 4-24. Drainage Enhancement Concepts

Benefits

- Retain larger amounts of water
- Increase drainage and storage capacity of floor
- Can be emplaced in predefined areas

Benefits

- Same as Option 1
- Reduced extent of phreatic surface

Benefits

- Same as Option 2
- Filter design more stable
- Ease of construction
- Optional keyway and grout curtain

Benefits

- Same as Option 3
- Simple and effective

Figure 4-25. Single Embankment Concepts

Water Flow in a Backfilled Drift--To better understand the effectiveness of the single embankment structure in controlling the seepage of water in a drift, two hydrologic analyses were performed. The first analysis was performed to determine the maximum distance that a continuous supply of water can migrate through a coarse backfill before the water supply is absorbed into the drift floor. This analysis assumed that no single embankment structure was present. The second analysis was performed to determine the effectiveness of the design in controlling the lateral migration of water flow in a drift of a single embankment with a reduced permeability (as illustrated in Figure 4-25d). A discussion of these analyses is included in Appendix E. The results indicate that a single embankment structure can be very effective in controlling the lateral migration of water in a drift. With the single embankment structure, the phreatic surface extended ~20 m beyond the toe of the embankment. Without the embankment, the pulse of water extended ~220 m. These analyses indicate that a seepage control test could be performed over a reasonable distance in a drift. As discussed in Appendix E, results depend upon the bulk hydrologic properties of fractured welded tuff and the volume of water encountered in the underground facility. Because the analyses considered a low bulk rock hydraulic conductivity of $K_{\text{sat}} = 10^{-5}$ cm/s and a large continuous supply (1 gpm) of water for the unsaturated zone, a drift ~35 to ~40 m long is proposed to conduct seepage control tests.

Fines Migration--Water entering the potential repository would be drained in the floor of the drifts. Assuming locally saturated conditions, the hydraulic conductivity of the highly fractured, welded tuff portion of the Topopah Spring Member is dominated by the fractures. Therefore, under saturated conditions, most drainage would occur through the fractures. Water in sufficient quantities would move through the backfill (crushed rock), which would be comprised of particles of various sizes. Fine-sized particles could be dislodged and transported. If a sufficient quantity of particles is transported, the fractures could clog. This in turn would reduce the hydraulic conductivity and drainage of the rock. Source of fines, mechanisms important for fines migration, and standard engineered approach to controlling fines migrations are the aspects of fines migration discussed briefly below.

Fines migrating in a geologic setting may concentrate at pores or fractures and reduce the permeability of the rock formation. The source of these fines may be formation fines that were

created during the operational period on the floor of the drift or introduced during the placement of the backfill.

Once deposited, the particles undergo detachment from the deposition site and are transported to either a pore restriction or a fracture, where clogging could occur. The fluid interstitial velocity largely determines the rates of particle release from the deposition site (Gruesbeck and Collins, 1982). Once detached, the particles are transported in the fluid stream until they are redeposited as a film or in a pore restriction or a fracture. Two general mechanisms affecting the mobility of particles are mechanical bridging and physicochemical effects. Mechanical bridging, which is usually associated with the larger particles being transported, depends on the relative size of the particles, the size of the restriction, the amount of fluids moving, and the concentration of fines in the fluid. Increasing the concentration of fines increases the tendency for bridging. Considering size only, a particle that is greater than or equal to the pore space, much smaller than the pore space, or somewhere in between, will have a tendency to deposit at the surface of the pore space, pass through the pore space, or be retained in the pore space, respectively (Khilar et al., 1985). Deposition of the finer colloidal particles, particles having a diameter in the range of 10^{-3} to 10^{-6} mm, is a physicochemical effect that depends upon the surface properties, such as particle wettability and the surface, interfacial forces (Muecke, 1979) of the suspended particles and the matrix. Particle migration has been addressed in general engineering practice.

In engineering practice, fines migration is of concern in dam construction using earthen material. Three areas of concern are piping, permeability, and segregation of the material. These areas of concerns apply to filter design. Dam construction approaches attempt to:

- Avoid deleterious movement of the finer particles (piping), which can cause instability of the rockfill.
- Ensure that the fluid moving through the rockfill does not build up excessive seepage or hydrostatic pressures. Therefore, the filter must have sufficiently high permeability to dissipate the fluids in the rockfill.
- Avoid segregation of the coarse-sized fraction and the fine-sized fraction of the rockfill. Segregation would reduce the effectiveness of the filter.

Figure 4-26 presents a diagram illustrating the effect of a filter on particle migration. The concerns of piping, permeability, and segregation will be starting points in developing an initial design for the surface backfill and seepage control tests. The general filter criteria are presented below.

In a well-graded, sandy gravel filter, which may be similar to the backfill used in sealing tests, the larger particles are contained in a matrix of finer-sized particles. It is the finer-sized particles that govern the sizes of the pore channels through which the water flows and that can be used to determine the potential for particle movement and piping. Currently accepted filter criteria for prevention of piping is based on the finer-sized fraction of the backfill. The following criterion is generally accepted for material that have a D_{15} larger than 1.0 mm:

$$D_{15}/D_{85} \leq 5$$

where

D_{15} = Particle size in filter for which 15% by weight of the particles are smaller, and

D_{85} = Particle size in the base soil for which 85% by weight of the particles are smaller.

This filter criterion is considered to be conservative by a factor of two and to apply to angular particles of crushed rock and to filters that may be subjected to vibrations (Sherard et al., 1984a). Sherard also concluded that particles smaller than $0.15 D_{15}$ that are carried in water suspensions will generally pass through the voids out of the filter. Particles greater than $\sim 0.12 D_{15}$ would be retained near the filter face. Additional criteria mentioned by Sherard et al. (1984b) suggested that for clays, sandy silts, clays with some sand content, and fine-grained clays (D_{85} of 0.03 to 0.1 mm), a sand or gravelly sand filter with an average D_{15} not exceeding ~ 0.5 mm is both reasonable and conservative. For fine-grained soils of low cohesion and exceptionally fine soils, a filter with an average D_{15} ranging from 0.2 to 0.3 mm would be acceptable and conservative.

Figure 4-26. Conditions at a Boundary Between a Soil and a Protective Filter (Cedergren, 1967)

The permeability criterion for drainage to allow dissipation of fluids through the filter is:

$$D_{15} \text{ (filter)}/D_{15} \text{ (base)} \geq 5.$$

This relationship is roughly equivalent to the permeability of the filter being ~25 (or more) times the permeability of the base soil (Perry, 1987). To avoid segregation of the filter material, the U.S. Bureau of Reclamation (USBR) recommends that the maximum size of the filter aggregate should be <8 cm to minimize the potential for segregation and bridging of the large particles during placement (Cedergren, 1967). The above discussion provides preliminary guidance on defining the rockfill to be used for selected tests.

4.6.4 Description of the Test

The proposed test will include evaluations of water flow in a backfill and in the rock beneath the backfilled test drift, the influence of a seepage control feature in controlling the extent of water movement in a drift, and fines migration within the backfill following the introduction of water into the backfill. To complete these evaluations, three tests are proposed. The first test might involve a pilot hole for assessing the performance of instrumentation and interpreting infiltration into fractured rock. The second and third tests would be larger-scale tests in the fractured Tiva Canyon and Topopah Spring Members. Prior to flooding the drift, a detailed characterization of the rock beneath the test area will be performed. A second drift beneath the test area is proposed to characterize the fractured and saturated state of the rock mass. Cross-borehole geophysics and hydrologic techniques discussed in Chapter 3 will be used for this characterization.

In the first test, a pilot 1-m-diameter borehole would be drilled and backfilled with rockfill. Following emplacement of the backfill, the borehole will be sealed and water injection ports constructed. During the pilot testing, emphasis will be placed on evaluating the performance of the pressure suction instrumentation and perfecting the use of geophysical methods used to detect saturation front movement.

The second test evaluates water flow through backfill and rock beneath the backfill. Following the placement of backfill and associated instrumentation, the drift will be flooded and the response monitored. Depending upon the hydraulic conductivity of the rock, the proposed length of 35 to 40 m may be insufficient, necessitating two phases of testing: evaluating the flow response of the general backfill and the rock underlying the backfill and testing the seepage control structure.

A third test is proposed in the Topopah Spring Member to validate the predicted performance in this formation. The preferred test area is outside the potential repository boundary off the ramp that provides access to the Calico Hills unit. Because of the prior testing in the fractured welded tuff of the Tiva Canyon Member, the primary reason for conducting the test is to show similar hydrologic performance in the Topopah Spring Member. If the fractured and saturated state of the Topopah Spring is judged to be similar to the Tiva Canyon Member, this test may not be necessary.

4.6.5 Layout and Important Parameters

The layout for the major seepage control test is illustrated in Figure 4-27. The test area includes a monitoring drift and a test drift. Both drifts would be ~3.7 m in diameter and be separated by ~15 m. In the lower monitoring drift, geophysical boreholes will be positioned to characterize the fractured nature of the rock mass prior to the testing in the upper drift. The boreholes will be used to monitor the progression of the saturation front in the fractures as saturation in the upper drifts occurs. The current design for the borehole monitoring system is:

- Seven stations, ~5 m apart,
- Two boreholes per station,
- Borehole length of ~15 m, and
- Each borehole oriented to intercept the fractures at an oblique angle.

Figure 4-27. Layout for Seepage Control Test

The design, i.e., the number, location, and orientation of the borehole monitoring system, depends upon the actual site conditions. Therefore, the design will be adapted to the conditions encountered.

In the upper drift, backfill will be placed in the entire length. The exact quality, gradation, and emplacement technique used for the backfilled area will be determined from the surface backfill test results. Instruments will be placed concurrently with the backfill. Parameters of interest are the movement of water in the rock and the backfill and the impact of the water movement in redistributing the fines. Therefore, the parameters of interest are saturation state in the rock and the backfill, the development of a phreatic surface in the backfill, and the gradation of the backfill before and after the test. The parameters of interest, the techniques to measure the parameters, and the number of measurements are given in Table 4-9.

4.6.6 Fielding of the Experiment

Prior to fielding the experiment, a comprehensive modeling and site characterization effort will be undertaken. The rock mass beneath the drift as well as the drift floor will be characterized. Selected samples will be taken from the monitoring boreholes to determine saturated versus unsaturated properties of the rock. Because this test depends upon both bulk and matrix hydrologic properties of the rock, the current concept for testing the seepage control structure may change.

Given the current test concept, fielding of the experiment will include the following:

- Map the floor of the test drift.
- Drill the monitoring holes beneath the test drift and extract samples for laboratory testing.
- Log the holes including the geophysical logs.
- Perform hydrologic testing in all holes.
- Develop a discrete flow model for the rock mass and perform predictive numerical analyses that are required for final test design changes.

Table 4-9. Parameters, Techniques, and Number of Measurements to be Made for Each Major Seepage Control Test

Parameter	Technique	Number of Measurements
Pore Pressure	Direct monitoring using a pore pressure cell before, during, and after flooding of the drift.	12
Water Level	Direct monitoring of the water level in the backfill before, during, and after flooding of the drift using piezometers.	12
Density and Moisture Content	Direct measurements of the as-placed density and moisture content of the backfill will be made during placement operations of the backfill. A portable nuclear density device (ASTM D 2922-81) will be used. Similar field measurements will be made during sample extraction.	48
Gradations, Moisture Content, and Density	Laboratory measurements will be made of samples extracted from the drift using ASTM D 422-63 (gradation analysis) and ASTM D 2216-80.	48
Suction Potential	Direct measurements using a soil moisture tensiometer will be made on the suction potential of the backfill before, during, and after introducing water to the drift.	48
Earth Pressure	Direct measurements will be made of pressure at selected locations to assess effect of saturation on pressure. Earth pressure cells will be used.	12
Settlement	Direct measurements using a settlement gage will be made of settlement that occurs in the backfill following saturation.	4

- Begin backfill emplacement operations and concurrent placement of instruments. Place moisture detection probes in the monitoring holes.
- Introduce water into the backfill.
- After running the test for a time period determined through numerical analyses, extract selected samples from the backfill.

4.6.7 Miscellaneous Requirements

To support the field tests, several numerical analyses will be required. These analyses require the characterization of the site using geophysical and hydrologic techniques, and laboratory analyses of the rock properties. Specific laboratory analyses and field quality-control measures are presented below.

Laboratory Testing--Specific laboratory analyses required include:

- Rock.
 - Porosity (fracture and matrix).
 - Saturated and unsaturated hydrologic properties.
- Backfill.
 - Moisture content and density (contrasted with the field measurements).

Field Quality-Control Measures--Field activities that must be undertaken to interpret the results include measurements of the backfill and the rock mass under the test drift. The density and the moisture content of the backfill during and following the backfill operation will be measured. These values will be compared with the laboratory values at similar locations. Soil moisture values determined from soil moisture tensiometers will be contrasted with similar values obtained from other laboratory and field measurements.

In addition to these measurements, a suite of logs will be run to characterize the rock beneath the test area. The logs currently proposed are:

- Borehole video camera, radially and axially oriented, to assess the condition of the borehole, the location and orientation of fractures, and any fluid entry.
- Compensated density.
- Compensated neutron to measure moisture in the rock.

More than one technique will be used to measure site properties. This will provide an assurance that consistent data are being recorded. It will also enhance the likelihood that test results can be successfully predicted.

Additional field quality controls will be defined from the surface backfill tests.

Special Facility Requirements--To assure that the backfilling operation before the test simulates actual backfilling operations in the proposed repository, an underground area to store the backfill is required. Placement of large amounts of backfill for this test will also require the ability to sieve and mix large samples.

A laboratory test area identical to that proposed for the surface backfill tests is required to test a large number of samples.

Constraints on Testing--By experimental design and planning, modification of the rock mass should be minimized. This suggests the following precautions:

- No introduction of water into the rock mass prior to the seepage control test, i.e., dry drilling of monitoring boreholes and dry development of monitoring and test drifts.
- No water introduction from tests in close proximity to the seepage control test.
- No heating of rock.
- No major invasive techniques.

4.7 Backfill Tests

4.7.1 Objectives of the Test

The backfill tests are in situ tests and include demonstrating backfill emplacement, monitoring the backfill hydrological performance, and determining the as-placed, geotechnical properties of the backfill. The test is an extension of the surface backfill tests and will use the engineered backfill specifications determined from those tests.

The objective of the backfill test in the Calico Hills unit is to characterize the structural and hydrologic performance of various combinations (mechanical emplacement, throwing methods, or some combination) of rockfill material as emplaced in the underground and subjected to heating. The material properties that will be evaluated are the density, permeability, moisture content, and degree of particle degradation due to backfill emplacement procedures. Material density and settlement will also impact the degree of potential voids development. One function of backfill is to limit potential surrounding rock deformation from drift excavations. A second function of backfill is to prevent migration of groundwater.

4.7.2 Description of the Test

Material gradations may change between tunnel boring, surface emplacement, screening, and actual emplacement. This issue will be addressed in the surface backfill test. The surface backfill tests allow definition of the initial gradation at an appropriately coarser level to compensate for particle breakdown.

The selection of material and the emplacement techniques will also be addressed by backfill-rock interaction analysis for the roof and sidewall (described briefly below). The stages in the analysis are presented in Figure 4-28b. The analysis considers elastic and inelastic cases.

Backfill-Rock Interaction--The interaction of the backfill with the surrounding rock mass can be represented by the ground support interaction shown in Figure 4-28a. The ground reaction

(a) Typical Ground Support Reaction Curve

Initial Condition

After Removal of Artificial Support

After Placement of Backfill

(b) Ground Reaction After Support Removal and Emplacement of Backfill

Figure 4-28. Ground Support Reaction Curves for Backfill

curve represents the characteristic pressure deformation curve for the media. Neglecting gravitational effects, the higher the support pressure is, the lower the expected deformation will be. If support is not provided for the roof, the potential support pressures rise due to failure of the immediate roof. The backfill support stiffness line represents the support pressure provided for inward deformation. The intersection of the ground reaction curve and the backfill support stiffness line determines the equilibrium conditions under which support is provided. The support pressure provided to the roof is generally higher than that provided to the wall because of localized gravitational effects.

Initially the support pressure for the artificial support is in equilibrium with the surrounding rock. Following removal of the support and placement of the backfill if necessary, a new equilibrium level at zero support is established. The backfill may deform elastically or inelastically with increasing support pressure. In the inelastic case, the sidewall exhibits a different load deformation curve than the roof. After heating, thermal stresses increase and a new higher equilibrium pressure is achieved that depends on the deformational characteristics of the backfill and the surrounding rock. The backfill should sustain the higher stress levels without adverse rock deformations in either the backfill or the surrounding rock.

4.7.3 Layout and Important Parameters

The present layout for the backfill test consists of excavating a dedicated drift (adit) in close proximity and parallel to an existing submain. The total length of the test drift is anticipated to be ~45 m (four 10-m sections for testing different material and placement techniques plus some additional length for installation of a temporary bulkhead). The drift length is selected to simulate a representative drift length for material emplacement and to minimize the end effects of a temporary containment bulkhead. The existing drift will provide an accessible location for instrument installation and sample collection. It is assumed the test drift can be excavated near one of several drifts planned to satisfy other test requirements. Several planned drifts excavated perpendicular to the main drift at the Calico Hills level for the hydrologic properties of major faults test could be considered for the access drift of the backfill test.

The specific location selected for the test drift will be based on detailed geologic mapping of the potential test areas. The site selection may use additional geophysical characterization techniques previously discussed (Section 3.2). The intent is to select an area with a typical fracture pattern and density, while avoiding features that could significantly affect roof and side-wall performance. It will be performed in a representative area of the ESF.

The material selected for testing will be determined from the surface backfill test results. As discussed in Section 4.4.7, the laboratory testing of candidate backfill material determines fundamental index properties such as density, dry density versus moisture content, void ratio versus confining pressure, and gradation.

Earthen backfill for the test will be screened to the gradation determined from the surface backfill test analyses. The material will be screened at the surface stockpile, transported to the underground test area, and stored in a nearby alcove until emplaced in the test drift. This multiple material-handling sequence simulates an actual backfilling operation. The extent of fines creation should be representative in this material-handling sequence.

Parameters to be Measured--The following is a list of parameters to be measured in the backfill test:

- Material gradation after initial screening, transportation underground, and emplacement in the test drift,
- Moisture content of the material graded above,
- Earth pressures created by the loading of the backfill,
- Settlement of the backfill following placement operations,
- Rock mass deformation due to support removal, emplacement of backfill, and room heating,
- Rise in temperature due to room heating, and
- Permeability of the emplaced backfill.

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Measurement Methods--The primary parameters to be measured during the Calico Hills backfill test are the material properties and the rock mass deformation due to simulated repository heating. The following discussion presents the anticipated approach to collecting the relevant information.

Sieve-size analyses will be performed on the backfill at various points during the placement and postplacement sequence. Grab samples will be collected at the surface screening facility and the underground storage area prior to emplacement, and immediately after the placement and compaction (if performed). Samples will be analyzed according to ASTM D 422-63 or an equivalent method appropriate for a coarse material. Moisture contents will be determined concurrently, in accordance with this standard.

The in situ measurements to be made on the backfill include the density, earth pressures, temperature, and settlement. The in situ measurements to be made in the rock mass include deformation and temperature. Instrument gage locations for earth pressures and settlement are anticipated to be located as shown in Figure 4-29. These gages will be installed during fill placement. When a particular area is backfilled, it will be excavated by hand and the gage installed and surrounded by dense sand to provide the necessary coupling to the surrounding backfill material. The succeeding lifts will be placed as described above. A summary of the instrumentation requirements is presented in Table 4-10.

Table 4-10. Summary of Gage Requirements--Backfill Test

Instrument	Number of Tests	Gages per Test	Total Gages
Thermocouples	4	104	416
MPBXs	4	8 ^a	32
Earth Pressure Cells	4	12	48
Settlement Gages	4	4	16
^a Two MPBXs are proposed, each having four anchors.

Figure 4-29. Backfill Test at Depth

A data acquisition/recording system will be needed for the in situ measurements performed for the Calico Hills backfill test. The data acquisition system will be capable of automatic timed readings of all instruments. It will perform any necessary preprocessing or conversion of signals prior to recording the digital information. It will be capable of storing and transferring data as required for analysis and reporting. The polling devices may be a combination of specific devices that monitor one set of instruments or a centralized system that concurrently polls and records a number of instruments.

The components of a comparable system have been described previously (Roberds et al., 1982). The data acquisition system will need to support the following minimum channel requirements for four test sections in the backfill test drift:

- 104 channels for temperature output.
- 84 channels for heater power output (assumed 7 heaters per test section and 3 elements per heater).
- 32 channels for MPBX output.
- 12 channels for earth pressure cell output.
- 4 channels for settlement gage output.

Monitoring frequency will be refined during the surface backfill testing for specific fill material. In general, the frequency will be greatest in early time and decrease as the testing proceeds.

4.7.4 Fielding of the Experiment

As mentioned previously, the fielding of the experiment requires a dedicated drift ~ 45 m in length. These tests may continue for some time during which access to the test area and ventilation through the drift would be restricted. The location of the backfill tests should be selected so that other unrelated activities do not interfere with the structural stability or the stress regime in the immediate area. The probable sequence of construction activities is:

- Excavate drift and run a GPR survey.
- Emplace room heaters and rock deformation instrumentation.
- Fabricate a backfill plant.
- Transport the prepared backfill below ground.
- Place fill in the drift in layers suitable for hand compaction using a small rubber-wheeled LHD. As the fill height approaches the back of the drift, the LHD will be used to pack the fill material at as high a slope as the material allows using the "powder puff" hand-held compactor. A pneumatic placer with a "slick pipe" arrangement blows muck into the crown.
- Emplace settlement and earth pressure instrumentation concurrently with the placement of backfill.
- Monitor ground displacement during heating.
- After the test, characterize the backfill for physical, mechanical, and hydrological properties.

4.7.5 Miscellaneous Requirements

The miscellaneous requirements include laboratory testing, field control testing, special facility requirements, and test constraints. The surface backfill test discussed previously selects material and requirements through combined studies. To assure quality control of backfill material emplacement, quality-control testing may be performed prior to and during emplacement of the backfill to assure meeting the seal design specifications mentioned previously in the surface backfill tests.

The quality-control testing required is specific to the seal material employed. During emplacement of the engineered fill, the use of laboratory and field quality-control measures assures that the material meets emplacement specifications with regard to grain-size distribution, maximum dry density (emplacement density), moisture content, aggregate gradation, strength, and bulk density. Recommended quality-control laboratory and field tests for earthen material can include those defined previously for the surface backfill tests.

Special Facility Requirements--Special facility requirements have been discussed previously in Section 3.1. In addition, a processing plant will be required for blending material, as in the surface backfill test.

Constraints on Testing--Before testing can begin, the primary constraint is the completion of the surface backfill tests. There are no special constraints during testing.

4.8 Bulkhead Test in the Calico Hills Unit

4.8.1 Description of the Test

The proposed bulkhead test will be performed in the Calico Hills unit and will evaluate a series of underground construction activities for large-scale bulkheads. The bulkhead in the ESF provides a more complete isolation than that provided by smaller embankments and is intended for fault zone flowing water. The bulkhead test might be performed as part of performance confirmation testing of sealing components to resolve design issues not previously resolved by laboratory or small-scale in situ testing and to prove hydrologic performance. The bulkhead test evaluates removal of artificial rock support, grouting or other methods to restore the MPZ, and seal emplacement, as illustrated in Figure 4-30. The test will be performed at the end of a dedicated drift near an area where there is a source of inflow.⁵ After completing construction of the bulkhead, water pressure would be allowed to develop behind the bulkhead. After establishment of a pressure head behind the bulkhead, the temperatures of the bulkhead and surrounding rock would be elevated to simulate the placement of radioactive waste.

Objective of the Test--The bulkhead test in the Calico Hills unit would be performed as part of the performance confirmation testing of sealing components to resolve construction issues not previously resolved by laboratory or small-scale in situ testing and to prove hydrologic performance. The objectives of the bulkhead test are to show that:

⁵Note that with the current understanding of the unsaturated zone, it is not anticipated that the bulkhead test is necessary. Nevertheless, the bulkhead test is proposed for completeness should it be found that the discrete fault zones represent significant sources of water.

Figure 4-30. Bulkhead Test in the Calico Hills Unit

- Cementitious material can be emplaced to achieve low conductivity and a high level of performance when subjected to a pressure head in a higher temperature environment.
- Seal construction activities control the thermal effects in the placement of cementitious material involving massive pours of concrete.
- Grout emplacement reduces the conductivity in the MPZ.
- Contact grouting at the top of the bulkhead reduces the conductivity of the interface zone that might be subject to separation.

The general objective to be met relates to the tentative hydrologic design goals for the repository (Fernandez et al., 1987). As additional information becomes available through site characterization, design goals will be updated and the test should demonstrate satisfaction of the design goals.

Large-Scale Testing--Field testing includes testing of the bulkhead that may be keyed into the surrounding rock mass to resist hydraulic loading. The dead-end drift is backfilled in the normal manner near the source of inflow. A sanded grout or bentonite material might be placed over the backfill to prevent bleed off of concrete during seal emplacement. Heaters are placed in the surrounding rock. The surrounding rock mass is instrumented with MPBX to measure displacement in the MPZ. The plug is emplaced and instrumented with strain gages, stress cells, and pore pressure cells. Pore pressures are allowed to develop and the structural response of the plug and surrounding rock mass is monitored.⁶ As in the large-scale shaft seal and shaft fill tests discussed below, statistical quality-control methods ensure that cementitious material meet specifications through sampling and standardized testing.

Performance Objective and Supporting Analyses--The bulkhead tests in the Calico Hills unit entail similar flow performance considerations as the previously mentioned tests. Either transient or steady-state flow develops during testing. The objectives of the test may be met if the flow satisfies a hydrologic performance goal established for the plugs. The test performance objective for the borehole seals to be satisfied under steady-state conditions might be

⁶Hydraulic pressure loading is separated from thermal loading. The sequencing of operations depends upon site-specific conditions of the flow zone.

$$\frac{Q_t}{\Delta H} \leq C_r$$

where

Q_t = Total flow rate through a seal system, i.e., the seal, MPZ, and interface zone (in particular, the top interface zone and seal), during the tests,

ΔH = Change in air potential or head loss through the seal system during the test, and

C_r = Required conductance as determined from repository or seal design studies. The required conductance is proportional to the product of the hydraulic conductivity and seal area and inversely proportional to plug length.

The pretest analyses required for the bulkhead test in the Calico Hills unit include the following activities:

- Design a primary grouting pattern to cut off flow in the region of the bulkhead through the previous fracture grouting and heated grouted rock mass tests.
- Find the final mix design for the cementitious material for the plug that would include the w/c ratio, type of cement to be used; type, size and gradation of aggregate; and any admixtures determined through previous small-scale seal performance studies.
- Investigate the extent of the MPZ and estimate the flow paths through the MPZ that would be required to select instrumentation. Determine the final design layout.
- Design the water inflow chamber.
- Design the ventilation chamber.
- Design the heater array to simulate repository heating, if determined to be necessary.
- Perform a structural analysis of the plug, subject to heating and hydraulic pressure.
- Prepare design specifications and drawings for seal construction.

The posttest analyses include a comparison of the measured pore pressures, moisture content, and flow rates to the results of detailed modeling analysis. Similar comparisons would

examine thermally induced stresses and strains due to hydration. The analyses will show an understanding of hydrologic and structural phenomena and the partitioning of flow through the seal material, interface zone, and surrounding MPZ.

4.8.2 Layout and Important Parameters

In the bulkhead test, the construction sequence entails excavating the keyway and removing the artificial support. It also involves backfilling to the upper side of the seal, placing concrete, and contact grouting. Due to the potential separation at the upper surface of the plug, contact grouting may be especially important. In the lower-strength nonwelded tuff of the Calico Hills unit, it is anticipated that mechanical excavation can be performed. However, if needed, excavation could be achieved by using radial and lateral holes drilled on a precise pattern and loaded with an expansive agent to fragment the rock.

Parameters to be Measured--Data measurements made during these tests will be used to validate the seal, interface, and MPZ performance. The parameters to be measured are:

- Hydrological measurements.
 - Pore pressures (i.e., in the far-field rock mass, seal components, or modified disturbed rock zone).
 - Pressures in the water inflow chamber.
 - Inflow rates.
 - Effective porosity and permeability for seal material.
 - Travel times for breakthrough of tracers.
- Geomechanical measurements.
 - Stress and strain measurements in the seal material and the surrounding disturbed zone from test-induced loads.

- Thermal measurements.
 - Concrete hydration temperatures.
 - Seal component and rock mass temperatures.
 - Thermal output of the heaters.

Instrumentation Requirements--The instrumentation in these tests include pressure transducers and tensiometers for monitoring pore pressures and fluid rates into and out of the test area similar to that used in previous testing. Earth pressure cells and/or strain gages measure any changes in stress or strain within the seal material during the test. Preliminary sampling points include near the confined core of the plug and near the ends of the plug. A summary of the instrumentation for the bulkhead test in the Calico Hills unit is presented in Table 4-11.

A data acquisition/recording system will be needed for the in situ measurements performed for the Calico Hills bulkhead test. The data acquisition system will be capable of automatic timed readings of all instruments. It will perform any necessary preprocessing or conversion of signals prior to recording the digital information. It will be capable of storing and transferring data as required for analysis and reporting. The polling devices may be a combination of specific devices that monitor one set of instruments or a centralized system that concurrently polls and records a number of instruments.

The components of a comparable system have been described previously (Roberds et al., 1982). The data acquisition system will need to support the following minimum channel requirements:

- 20 channels for temperature output.
- 42 channels for heater power output (assume 2 rings of heaters; 7 heaters per ring; 3 elements per heater).
- 23 channels for pressure cell or tensiometer output.

Table 4-11. Summary of Gage Requirements--Bulkhead Test in the Calico Hills Unit

Instrument	Number of Tests	Gages per Test	Total Gages
Thermocouples	1	20	20
Pore Pressure Cell ^a or Tensiometer	1	23	23
Radial, Vertical, and Circumferential Strain Gages	1	6	6
Stress Cells	1	6	6
MPBX	1	2	2
^a Pore pressure cells and tensiometers may be used at the same location.			

- 8 channels for MPBX output.
- 18 channels for strain gage output.
- 6 channels for stress cell output.

In the previously described test, water is injected in the test zone and measured directly. The rate into the drift may be low and the surface area large, resulting in the loss of inflow to evaporation. The remainder of the water may collect on or below the floor of the downgradient drift.

Water inflow could be measured using a technique described by Wilson et al. (1983) and described below. A ventilation system evaporates all groundwater entering the isolated portion of the drift downgradient from the plug. The system measures changes in moisture content between the incoming and exhaust air streams. To operate successfully, the ventilation system provides an airflow rate and temperature that can vaporize the influx water. The difference in moisture content between the air entering and leaving the isolated area must be great enough to permit accurate measurement.

The proposed system (Figure 4-31) employs the circulation of outside air similar to that used at STRIPA. It measures the air velocity in an exhaust duct, the wet and dry bulb temperatures of the incoming and exhaust air streams, and the barometric pressures at each temperature measurement station. These measurements allow the calculation of the incoming and exhaust mass flow of the air and the transported water vapor. The difference between the incoming and exhaust water-vapor flow rates equals the rate at which water evaporates from the surfaces of the drift. Observation of the floor and walls of the drift assures the evaporation of all water entering the drift.

4.8.3 Fielding of the Experiment

The nature of the Calico Hills bulkhead test requires conducting this test in a dedicated drift. This test may continue for some time during which access to the test area and ventilation through the drift would be restricted. The location of the Calico Hills bulkhead test should be selected so that other unrelated activities do not interfere with structural stability or the stress regime in the immediate area. For example, drilling or blasting that may influence the test by creating secondary flow paths in the system should be avoided. The test location selected should minimize interference with other ongoing tests or activities in the ESF.

The following is the probable sequence of bulkhead construction:

- Drill radial pattern of heater holes and install heaters recessed into the surrounding rock mass (if temperature effects are considered important to include in the test).
- Place fill near the water pressure zone in layers suitable for hand compaction using a small rubber-wheeled LHD. As the fill height approaches the back of the drift, the LHD would be used to pack the fill material at as high a slope as the material allows using the "powder puff" hand-held compactor. A pneumatic placer with a "slick pipe" arrangement blows muck into the crown.
- A sanded grout is cast on the upstream side of the bulkhead to prevent bleed off during bulkhead construction.
- Install grout pipes. Grout the fractures in the surrounding rock mass to a depth of ~3 m.

Figure 4-31. Schematic Drawing of the Ventilation Drift Showing Installed Equipment and Boreholes

- Remove the artificial support (rockbolts, shotcrete, or liner).
- Line drill radial holes around the periphery of the seal to create an area in the keyway for horizontal drilling.
- Load with Bri-star (expansive agent) and excavate the area for horizontal or lateral drilling.
- Drill horizontal holes, load with Bri-star, and excavate the remaining portion of the keyway, removing the grout pipes used for primary rock grouting.
- Install a temporary bulkhead for the mass pour of the bulkhead.
- Position grout-pipe and cooling-pipe frames and attach pipes.
- Emplace the concrete pumped from the drift level in a single emplacement to eliminate any cold joints.
- Maintain moist and cool maturing conditions for 28 days.
- Contact grout the bulkhead and remove grout pipes.
- Remove temporary bulkhead and install the ventilation chamber fans, distribution duct for circulating air, auxiliary heaters, and bulkhead.

4.8.4 Miscellaneous Requirements

The miscellaneous requirements include laboratory testing, field control testing, special facility requirements, and test constraints. The results of the small-scale in situ and intermediate-scale borehole seal tests discussed previously are used to select the seal material and other requirements. To assure proper quality control of seal material emplacement, quality-control testing may be performed prior to and during emplacement of the bulkhead to assure meeting the seal design specifications, as mentioned previously in the small-scale in situ tests.

The quality-control testing required is specific to the seal material employed. During emplacement of the bulkhead, the use of laboratory and field quality-control measures assures that the material meets emplacement specifications for mix proportions, aggregate gradation, workability, strength, and bulk density.

The recommended laboratory and field testing for cementitious material can include those tests defined for the small-scale in situ tests.

Special Facility Requirements--The special facility requirements were discussed previously in Section 3.1. Special facilities will be necessary for batching the concrete and material handling to the location of the bulkhead test.

Constraint on Testing--The bulkhead test, if required, may be constrained by repository design and the potential impact that such a test might have on potential repository performance. It will be necessary to identify these constraints during site characterization to assure that performing such a test does not adversely affect potential repository performance.

4.9 Large-Scale Shaft Seal and Shaft Fill Tests

4.9.1 Description of the Test

The large-scale shaft seal and shaft fill tests evaluate a series of underground construction activities for large-scale seals. The tests simulate liner emplacement, liner removal, grouting or the use of other methods to restore the MPZ, and seal emplacement, as illustrated in Figures 4-32 through 4-35. The shaft seal and shaft fill reduce the flow of water down the shaft or the shaft and rock interface zone. The keyway provides direct treatment or localized restoration of the MPZ because of the removal of the more intensely fractured portion of the MPZ.

The following discussion presents two concepts for testing in nonwelded and welded tuff. The current major difference between testing in welded and nonwelded tuff is that the welded tuff may require primary and secondary grouting in the surrounding fractured tuff, while it is not anticipated that such grouting would be necessary in nonwelded tuff.

Objective of the Tests--The large-scale shaft seal and shaft fill tests might be performed as part of performance confirmation testing of sealing components to resolve design issues not

Figure 4-32. Large-Scale Shaft Seal Test in Welded Tuff

Figure 4-33. Large-Scale Shaft Seal Test in Nonwelded Tuff

Figure 4-34. Shaft Fill Test--Instrument Locations for Water Flow Monitoring

Upper Access Drift

Figure 4-35. Shaft Fill Test--Instrument Locations for Structural Monitoring

previously resolved by laboratory or small-scale in situ testing and to prove hydrologic performance. The objectives of the large-scale test program are to show that:

- Earthen or cementitious material can be emplaced to achieve low conductivity and a high level of hydrologic performance.
- Earthen or cementitious material can be emplaced to restrict maximum settlement in the shaft.
- Seal construction activities control the thermal effects in the placement of cementitious material involving massive pours of concrete.
- Grout emplacement reduces the conductivity in the MPZ.

The general objective to be met relates to the tentative hydrologic design goals for the potential repository (Fernandez et al., 1987). As additional design goals become available through site characterization, design goals will be updated and the test should demonstrate satisfaction of the design goals.

Large-Scale Testing--Field testing includes testing of shaft seal and shaft fill components, as illustrated in Figures 4-32 through 4-35. In the large-scale shaft fill test, a shaft is constructed by raise-bore methods from the lower access drift to the upper access drift and lined. Following this operation, the shaft is backfilled with an engineered material. The use of statistical quality methods ensures that the earthen material meet design specifications with regard to grain-size distribution, specified relative density, and moisture content. This may involve sampling the material and conducting standard laboratory or field tests.

Performance Objective and Supporting Analyses--The large-scale sealing tests entail similar flow performance considerations as the intermediate-scale borehole seal tests. Testing could be performed using either transient or steady-state methods. The objectives of the test may be met if the flow satisfies a hydrologic performance goal established for the plugs. In performing an air or water flow test, the test performance objective for borehole seals to be satisfied under steady-state conditions might be

$$\frac{Q_t}{\Delta H} \leq C_r$$

where

Q_t = Total flow rate through a seal system parallel to the seal axis (MPZ, interface zone, and seal during the tests),

ΔH = Change in air potential or head loss through the seal system during the test, and

C_r = Required conductance as determined from repository or seal design studies. The required conductance is proportional to the product of the hydraulic conductivity and seal area and inversely proportional to plug length.

The analyses conducted to support the tests may be separated into analyses required to define the test parameters (pretest analyses) and those required to interpret and evaluate the test results (posttest analyses).

The pretest analyses should find the optimum test configuration and test conditions. Some pretest analyses for the shaft fill test include:

- Determine the range, sensitivity, and accuracy of instrumentation.
- Select material and find the density requirements for these material. The density relations define the relative density for the material, and laboratory permeability testing allows selection of the optimum density while maintaining an effective hydraulic conductivity design goal of 10^{-5} to 10^{-4} cm/s.
- Design the particle size-gradation of the coarse shaft fill that will meet the requirements for settlement (<2%) and effective hydraulic conductivity design goal of 10^{-5} to 10^{-4} cm/s.
- Investigate the extent of the MPZ and estimate the flow paths through the shaft fill and MPZ to decide the final design of the instrumentation and layout.
- Decide on the fluids to be used as a permeant during the test (air or water).
- Design the water injection chamber used to introduce fluids in test areas.

The pretest analyses required for the shaft seal test include the following activities:

- Design the plug for structural stability and update the preliminary structural design requirements presented in Appendix F.
- Design a primary grouting pattern to cut off flow paths through the MPZ around the shaft in the region of the anchor-to-bedrock plug. The grouting plan will specify the grout hole pattern, mix types, injection pressures, and grouting methods (i.e., upstage, downstage). Prior to grouting design, fracture mapping and pressure testing may be specified (or included as part of the MPZ investigation).
- Find the final mix design for the cementitious material for the plug that would include the w/c ratio; type of cement to be used; type, size, and gradation of aggregate; and inclusion of any admixtures.
- Investigate the extent of the MPZ and estimate the flow paths through the shaft fill and the MPZ that would be required to select instrumentation. Decide the final design layout.
- Design the injection chamber used to seal the shaft test area.
- Select the method of liner removal and excavation and removal of the MPZ in the area of the plug so that the anchor-to-bedrock plug may be keyed into the shaft walls.
- Define secondary grouting requirements.

The posttest analyses include a comparison of the pore pressures, moisture content, and flow rates to the results of detailed modeling and analysis. Similar comparisons would examine thermally induced stresses and strains due to hydration. These analyses will show an understanding of hydrologic and structural phenomena and the partitioning of flow through the seal material, interface zone, and surrounding MPZ.

4.9.2 Layout and Important Parameters

Separate tests would be performed on shaft fill for structural and hydrological performance (Figures 4-34 and 4-35). During emplacement of engineered backfill and subsequent testing, a series of radial extensometers and settlement gages measure settlement in one shaft. In a separate shaft, emplacement occurs using the same methods, but pore pressure cells/tensiometers measure hydrologic flow performance. Induction logging would be used to monitor the wetting front

progressing down the shaft in four conduits shown in Figure 4-34. The electrical logging measures changes in backfill electrical conductivity with the advance of introduced fluids to the extent that the saturated interval changes over time.

In the large-scale shaft seal and shaft fill test, the construction sequence entails making saw cuts at the top and bottom of the plug, removing the liner, excavating the keyway, backfilling to the underside of the plug, placing the concrete, and optional contact grouting. Additional remote-sensing site characterization may be necessary at the location of these larger tests. A series of holes is drilled horizontally at the top of the seal to the full depth of the keyway and perhaps loaded with an expansive agent. Because of the high strength of welded tuff, mechanical excavation of a keyway may not be feasible and other methods may be necessary (Fernandez et al., 1989). However, this may not be a problem in the lower-strength welded tuff. The rock keyway is fragmented and excavated over a length of several feet to provide a large working area. Excavation of the keyway then proceeds from the top to the bottom of the plug. From this working area, vertical holes are drilled in a precise pattern and loaded with an expansive agent to remove the rest of the keyway. The rest of the pilot shaft is backfilled, and a water-injection chamber is constructed from the upper access drift.

A water injection chamber would be constructed and lined to direct flow toward the test interval. Either steady-state or transient testing using air or water might be conducted. For water testing, flowmeters measure the fluid volumes injected into the seal or backfill, and an array of pressure transducers/tensiometers assess pressure changes in the saturated and unsaturated flow regimes to find the extent of the phreatic surface. The data provides confirmation of the seal system performance. Elevated repository temperatures would not be simulated because of the near-surface placement of the plug. However, measurement of temperatures and thermal strains due to heating and cooling of the concrete would show performance of the seal system interface. The heating and cooling cycle may result in tensile strains and separation at the interface zone that could affect flow performance.

Parameters to be Measured--Data measurements made during these tests will be used to validate seal, interface, and MPZ performance. The parameters to be measured are:

- Hydrological measurements.
 - Pore pressures (i.e., in the far-field rock mass, seal components, or modified disturbed rock zone).
 - Injection pressures.
 - Inflow rates.
 - Effective porosity (if appropriate for seal material).
 - Travel times for breakthrough of tracers.
- Geomechanical measurements.
 - Test-induced stress measurements in the seal material and the surrounding disturbed zone.
 - Thermomechanical behavior (stress and strain) of the seal and surrounding rock mass.
- Thermal measurements.
 - Concrete hydration temperatures.
 - Seal component temperatures.

Instrumentation Requirements--The types of instrumentation in these tests includes pore pressure cells and tensiometers for monitoring pore pressures and fluid rates into and out of the test areas. Earth pressure cells and/or strain gages measure any changes in stress or strain within the seal material during the test. Preliminary sampling points are near the confined core of the plug and near the ends of the plug to evaluate end edge effects.

Transient or steady-state flow testing could again be used to conduct the tests. The test method depends upon the results of pretest analyses and field testing at smaller scales. A transient test would require the upper chamber to be pressurized and the pressure decay over time to be measured. Fluid volumes injected into the pressure chamber would be measured, and an array of pressure transducers and tensiometers might assess hydraulic potential changes in the saturated rock and unsaturated flow regimen. A steady-state test would be conducted with a constant injection flow rate, or a falling-head test would be conducted with the pressure monitored over time. The data allow an assessment of the seal system performance with respect to flow performance criteria. A summary of the instrumentation for the shaft fill test is presented in Table 4-12 and for the shaft seal test in Table 4-13.

A data acquisition/recording system will be needed for the in situ measurements performed for the shaft fill and shaft seal tests. The data acquisition system will be capable of automatic timed readings of all instruments. It will perform any necessary preprocessing or conversion of signals prior to recording digital information. It will be capable of storing and transferring data as required for analysis and reporting. The polling devices may be a combination of specific devices that monitor one set of instruments or a centralized system that concurrently polls and records a number of instruments.

Table 4-12. Summary of Gage Requirements--Shaft Fill Test

Instrument	Number of Tests	Gages per Test	Total Gages
Pore Pressure Cells/ Tensiometers	4	25	100
MPBXs	4	2	8
Earth Pressure Cells	4	7	28
Settlement Gages	4	15	60

Table 4-13. Summary of Gage Requirements--Shaft Seal Test (welded or nonwelded tuff)

Instrument	Number of Tests	Gages per Test	Total Gages
Pore Pressure Cells/ Tensiometers	2	27	54
Radial and Circumferential Strain Gages	2	3	6
Stress Cells	2	6	12
MPBXs	2	2	4
Earth Pressure Cells	2	4	8
Settlement Gages	2	20	40

The components of a comparable system have been described previously (Roberds et al., 1982). The data acquisition system will support the following minimum channel requirements for each test performed in the welded and nonwelded test facilities:

- 27 channels for pore pressure cell/tensiometer output.
- 7 channels for MPBX output.
- 4 channels for earth pressure cell output.
- 20 channels for settlement gage output.
- 6 channels for radial and circumferential strain gage output.
- 6 channels for stress cell output.

4.9.3 Fielding of the Experiment

The nature of the large-scale shaft seal and shaft fill tests requires conducting these tests in a dedicated shaft. These tests may continue for some time during which access to the test area and ventilation through the shaft would be restricted. The location of the shaft tests should be selected so that other unrelated activities do not interfere with structural stability or the stress regime in the immediate area. For example, drilling or blasting activities that may influence the test by creating secondary flow paths in the system should be avoided. The test location selected also should minimize interference with other ongoing tests or activities in the potential repository.

Much of the equipment required to construct the seals is available in underground mining or construction operations. Small-capacity drills capable of drilling grout and instrumentation holes will be used. Above-ground haulage equipment may be required to transport crushed fill material and premixed concrete. Compaction of fill and instrumentation installation will likely be more labor-intensive and require hand-compaction. Removal of sections of the concrete and steel liner may require special nonexplosive excavation equipment (e.g., acetylene cutting torches, saws, and hydraulic splitters). Special mucking equipment (e.g., Cryderman mucker or remote-controlled orange peel grab units) will remove the broken concrete liner from the shaft.

The following is a probable sequence of test shaft construction, assuming the basic underground test facility and the fracture grouting experiments are completed:

- Construct the collar and slab to suit the raise bore machine and raise bore method of excavating the test facility shafts. See Figures 3-4 and 4-36.
- Assemble the rig over the shaft location and center.
- Drill ~ 15 m of a 30-cm-diameter vertical pilot hole to intersect the lower drift. Move the drill cuttings into the LHD bucket and remove the cuttings to the muck bay or directly into an underground truck.
- Replace the pilot hole drill with the raise bore drill at the collar.
- Transport the reamer bit to the lower drift. Assemble and connect to drill string.
- Ream upwards to known bit diameters. Remove cuttings to muck bay or underground truck.
- Remove reamer equipment from collar area and install fence around shaft top.
- Hang shaft stage either at section of shaft requiring support or at the bottom of the raise bored shaft. See Figures 3-4 and 4-37. Install supports and lower stage as required. (Preferred support is shotcrete).
- Install instruments prior to lining as required for subsequent tests, raising and lowering stage as required.
- Install shaft form ~ 5 m long. Hang on second set of chains from the deck beams. Place the first lift of concrete.
- Repeat for second and third concrete lifts.
- Lower stage and commence grouting concrete liner and rock contact.
- Grout the fractured rock surrounding the shaft up to an approximate depth of 2.7 m from concrete face. See Figure 4-12.
- During these concrete and grout operations, collect all spillage at lower drift. Place a vinyl sheet on the invert and remove separately to surface.
- Make all preparations for shaft fill and shaft seal tests as required.

Figure 4-36. Test Facility Shaft Construction by Raise Bore Method

Figure 4-37. Schematic of Shaft Liner Construction in the Test Facility

The following is a probable sequence of shaft seal construction, assuming the lining is contact grouted and the surrounding rock pregrouted.

- Emplace the shaft fill to underside of the bulkhead.
- Saw cut 22.5 cm deep around shaft concrete at top and bottom levels of the bulkhead to be constructed. Line drill the top and bottom of the bulkhead with head jackhammers.
- Drill cut holes near the top. Load with Bri-star (expansive agent).
- Excavate the top half of the keyway with horizontal holes and the bottom with vertical holes.
- Push excavated muck into the shaft and muck out with an orange peel grab unit into a sinking bucket, hoist by the overhead crane, and dump where it can be removed by LHD equipment.
- Reconsolidate the shaft fill.
- Position grout pipe and cooling pipe frames and attach pipes.
- In the scaled-down tests situation, the concrete would be placed, probably pumped, from the collar level. A single placement is preferred, avoiding any cold joints.
- Maintain moist and cool maturing conditions for 28 days.
- Remove all concrete gear.
- Contact grout bulkhead and remove grout pipes (Figure 4-38).

The following is a probable sequence for the shaft fill experiment, assuming the facility construction is complete.

- Construct concrete block bulkheads in lower drift, one central to the two shafts and one at either end. See Figure 3-4. The dimensions are based on methods presented by Auld (1983). The bulkhead is keyed into the rock, a requirement of the design.
- Position all instruments.
- Clean lower drift inside bulkheads.

Note: Line drill at locations designated **a**.

Figure 4-38. Shaft Seal Contact Grouting

- Place and compact fill using hand compactors. Remove all supports as the fill height rises. Plain shotcrete support should be left in place. Continue this process until fill level is at the top of shaft. Hoist stage out of the shaft and keep ready for the next phase.
- Bring fill into the repository and place in the unloading station (junction UL 7, Figure 3-3) and take from there to the collar of the shaft.
- Lower fill material on a flat tray bucket that is tipped by unhooking two of the four slings and raising the other side. In the lower drift, move the fill to either side in layers suitable for compaction by hand methods or by a small rubber-wheeled LHD. As the fill height approaches the back of the drift, the LHD will be used to pack the fill material at as high a slope as the material allows. There are available small hand-held compactors known as "powder puffs" for compacting such slopes. To compact the last foot or so, a pneumatic placer with a "slick pipe" arrangement will be used to blow muck into the crown.
- When placing in the shaft, the workers will be out of the shaft while the fill is lowered. The flat tray bucket is operated remotely. When sufficient fill has been lowered, the workers return to the bottom to spread and compact each layer.
- Complete the shaft fill experiments.
 - Water flow monitoring.
 - Structural monitoring.

4.9.4 Miscellaneous Requirements

The miscellaneous requirements include laboratory testing, field control testing, special facility requirements, and test constraints. The small-scale in situ test results determine material and other requirements through combined studies. To assure proper quality control of seal material emplacement, quality-control testing may be performed prior to and during emplacement of the shaft fill and the shaft seal tests to assure meeting the seal design specifications, as mentioned previously in the small-scale in situ tests.

The quality-control testing required is specific to the seal material employed. During emplacement of the engineered shaft fill and the cementitious plug, the use of laboratory and field quality-control measure assures that the material meets emplacement specifications with regard to grain-size distribution, maximum dry density (emplacement density), moisture content, mix

proportions, aggregate gradation, workability, strength, and bulk density. Recommended quality-control laboratory and field tests for earthen material can include those tests defined for the surface backfill tests.

Special Facility Requirements--The special facility requirements were discussed previously in Section 3.1. Special facilities will be necessary for batching the concrete and earthen material and material handling to the location of the tests.

Constraints on testing--The special facilities selected in welded and nonwelded tuff outside the potential repository block should reduce constraints on testing (specifically, the constraint on introducing large amounts of water into the underground environment).

4.10 Remote Borehole Sealing Tests

The remote borehole sealing tests would demonstrate the remote emplacement of a seal in welded and nonwelded tuff. The tests confirm performance of methods used for identifying sealing zones, removing casing, emplacing seals under controlled pressure and temperature conditions, and testing the seal. All activities are performed remotely from the surface.

4.10.1 Description of the Test

Borehole sealing strategies evolve as information becomes available from a combination of office, field, and laboratory studies. The intermediate-scale borehole seal tests to be performed in welded and nonwelded tuff will provide information on the hydrologic performance of seals for selected material in welded and nonwelded tuff. As discussed previously, the current design tailors the seal to the conditions encountered in welded and nonwelded tuff. In nonwelded tuff, the degree of fracturing is not as significant, and simple plugs can be emplaced and evaluated in progressive lifts. In welded tuff, current design may require the placement of a bentonitic seal, a filter pack, and a main sealing material because of more intense fracturing at depth.

Performance Objective--The objective of the remote borehole sealing tests will be to implement current designs in an exploratory borehole with borehole geologic and hydrologic characterization, borehole preparation, seal emplacement, and seal testing performed from the surface.

Seal Hydrologic Analysis--Testing would be performed using transient or steady-state methods selected from previous intermediate-scale borehole seal testing in welded and nonwelded tuff. The test performance objective to be satisfied under the steady-state condition would be

$$\frac{Q_t}{\Delta H} \leq C_r$$

where

Q_t = Total flow rate through a seal system parallel to the seal axis (MPZ, interface zone, and seal) during the test,

ΔH = Change in potential or head loss through the seal system during the tests, and

C_r = Required conductance as determined from repository or seal design studies. The required conductance is proportional to the product of the hydraulic conductivity and seal area and inversely proportional to seal length.

The methods of analyses have been discussed previously in Section 4.2.1. Selected methods would be implemented for satisfying the performance objective.

4.10.2 Layout and Important Parameters

The present layout consists of three seals emplaced at depth, as illustrated in Figure 4-39.⁷ The important test parameters were discussed previously as part of the

⁷The figure shows two seal configurations for welded and nonwelded tuff in the upper section of the borehole. A third test might be performed in the Calico Hills formation.

Figure 4-39. Remote Borehole Sealing Test

intermediate-scale borehole seal tests. These tests could be performed in deep holes located outside the current potential repository perimeter. After identifying the seal zone, the casing is removed, the hole is backfilled, and a sacrificial packer is placed below the plug area. The plug is poured in successive lifts. If the lower lift is allowed to cure, it could serve the function of a packer.

4.10.3 Fielding of the Experiment

Fielding of the proposed remote borehole sealing test is illustrated in Figure 4-40. Fielding of the proposed remote borehole tests includes (a) cutting casing, (b) backfilling a deep borehole, (c) applying an injection pressure, (d) emplacing a seal, and (e) seal testing.

Portions of the casing would be initially removed and the hole backfilled. After identifying selected sealing locations in the welded or nonwelded tuff through video logging, a seal would be emplaced using low-pressure injection methods. The use of downhole cameras in the unsaturated zone appears to be the preferred method to identify seal locations.

The casing could be removed using several methods. At the base of the casing, where the casing is cemented to the formation, rotary shoes could cut the cement or earthen material in the casing-wellbore (Figure 4-40). Subsequently, the casing could be removed using outside mechanical cutters, inside hydraulic or mechanical casing cutters, chemical cutters, or jet casing cutters. Once the casing has been freed, it is removed in one or more retrievals (Figure 4-40).

Remote video logging would be used because acoustic caliper logs will not function in air environments. Remote video logging might be implemented with mechanical caliper logging; however, there may be limitations in nonwelded tuff where overbreak zones are significant.

A test in the seal zone would be performed to find a safe injection pressure (Figure 4-40). The safe injection pressure develops radial interface stress along the periphery of the seal, but does not degrade the surrounding formation. A combination of methods that includes in situ stress measurements would be used to make this determination.

Cutting Casing

Backfilling a Deep Borehole

Applying Injection Pressure

Emplacing a Seal

Testing Seal

Figure 4-40. Remote Borehole Sealing Test: (a) Cutting Casing, (b) Backfilling a Deep Borehole, (c) Applying Injection Pressure, (d) Emplacing a Seal, and (e) Testing Seal

After pressure injection testing, the seal would be emplaced either by direct circulation or by the dump bailer. A dump bailer, shown in Figure 4-40, is a mechanical tool that combines a measured quantity of cementitious material. The bailer tags the mechanical packer and raises above it. The bailer opens by detonating a charge that shatters a glass disk, and then it would release material into the seal zone.

During cement hydration, the seal is subjected to a low squeeze pressure using a low fluid-loss slurry. The low squeeze pressure forces filtrate from the slurry into the surrounding tuff, creating a filter cake. The low fluid loss reduces dehydration and discourages bridging as the slurry is forced along the interface zone and into the surrounding formation.

After completion of the seal in several lifts, the seal is flow-tested using transient or steady-state methods, as determined in the intermediate-scale tests. The important parameters to be measured in the remote borehole sealing tests are flow rate and hydraulic potential, using measurement methods discussed previously.

Instrumentation Requirements--The types of instrumentation in these tests include pressure transducers, deformation gages, and flowmeters. These instruments have been discussed in previous sections. A summary of the instrumentation for the remote borehole sealing tests is presented in Table 4-14.

Table 4-14. Summary of Gage Requirements--Remote Borehole Sealing Test

Instrument	Number of Tests	Gages per Test	Total Gages
Borehole Deformation Gages	3	1	3
Pressure Transducers	3	2 ^a	6
Flowmeters	3	2	6
^a Redundant gages explicitly identified here due to remote nature of test.			

4.10.4 Miscellaneous Requirements

The miscellaneous requirements include laboratory testing, quality-control testing, special facilities, and constraints on testing. The laboratory testing is identical to the intermediate-scale borehole seal tests.

Field Quality-Control Measures--Statistical quality-control testing assures conformance of seal material to design specifications. The onsite testing includes calibration of mixing equipment, redundant measuring of blended material, and testing of seal material.

Calibration of the mixing equipment is normally performed by the cement service company. For this testing, onsite calibration ensures better performance of blending operations. The equipment calibrated includes redundant measuring devices, flowmeters, volume of tanks, and densimeters. Laboratory and field quality-control measurements assure that the material meets emplacement specifications for grain-size distribution, maximum dry density, moisture content, mix proportions, aggregate gradation, workability, strength, and bulk density. Recommended quality-control methods for earthen material include:

- Sieve analysis of sampled material to verify that grain-size distribution is within allowable design specifications (ASTM D 422-63, "Standard Method for Particle-Size Analysis of Soils").
- Nuclear density and sand cone measurements in the field-compacted shaft fill material to verify emplaced density is within the allowable design specifications (ASTM D 2922-81, "Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods [Shallow Depth]" or D 1556-82, "Standard Test Method for Density of Soil in Place by the Sand-Cone Method").
- Moisture content measurements either as field measurements (ASTM D 3017-88, "Standard Test Method for Water Content of Soil and Rock in Place by Nuclear Methods [Shallow Depth]") or as laboratory measurements on field samples (ASTM D 2216-80, "Standard Method for Laboratory Determination of Water [Moisture] Content of Soil, Rock, and Soil-Aggregate Mixtures") to verify that the seal material-moisture content at emplacement is within the allowable design specifications.

Recommended laboratory and field testing for cementitious material can include those tests defined for the small-scale in situ tests and the fracture grouting tests.

Special Facilities--The facilities required are those used in the placement of seals in the oil and deep-well injection industries. These include a drilling rig able to operate wireline tools at a depth of 90 m to 300 m. Additional facilities include onsite laboratory testing facilities for seal material and additives.

Required cement mixers include jet, recirculating, and batch mixers. The jet mixer uses a funnel-shaped hopper and fluid stream to move blended cement into a tank for pump suction. Mixing speed is determined by the volume of water forced through the set and the amount of cement that may be fed through the hopper during mixing. A normal slurry mix rate is $1.4 \text{ m}^3/\text{min}$.

Constraint on Testing--The test should be located well outside the potential repository block so that water can be introduced during testing. Nevertheless, a constraint may exist on the test that would be identified during site characterization activities.

5.0 SAFETY AND QUALITY ASSURANCE CONSIDERATIONS

A summary of safety considerations (Section 5.1) and Quality Assurance considerations (Section 5.2) associated with in situ seal testing are presented below.

5.1 Safety Considerations

Safety considerations include the potential hazards (Section 5.1.1) and the mitigation of the hazards (Section 5.1.2).

5.1.1 Potential Hazards of Seal Testing

The following discussion addresses the potential hazards of seal testing and associated underground construction.

- Ground Support--Ground support hazards include rock falls and often involve artificial rock support.
- Ventilation--The very act of plugging/sealing the shaft or drift will cause a considerable problem with existing ventilation not only in the immediate area of the seal, but also in the surrounding area of the underground excavation that relies on that shaft or drift for ventilation. Small-scale sealing of boreholes, either above ground or within the test facilities, should not require additional ventilation.
- Electrical Shock--Electrical shock hazards can occur around any electrical machinery, electrical lighting, or electrically operated test equipment. Sources of electrical shock are usually frayed wiring, open circuits and connections, or use of electrical equipment incorrectly.
- Chemical--Possible chemical hazards include:
 - Tracer gases or liquids that could be hazardous to inhale, contact with the skin, or ingest.
 - Concrete, grout, or earthen material (bentonite clay) that can be hazardous upon contact with the skin (by drying the skin excessively or provoking a caustic reaction).

- Gas Pressure--Hazards of high gas pressures are associated with:
 - Permeability testing (steady-state or transient pressure decay), which can include possibly dangerous high gas pressures from high-pressure gas bottles, regulators, valves, and fittings; a pressurized rock/earthen cavity; and pressurized packers (either gas- or liquid-filled).
 - Gas-pressurized pneumatic stowing of backfill, shotcrete, or earthen seal material.
- Fluid Pressure--Hazards of high fluid pressures are associated with:
 - Permeability testing (steady-state or transient pressure decay), which can include possibly dangerous high fluid pressures from high-pressure fluid valves, tubing, and fittings; a pressurized rock/earthen cavity; and pressurized packers (liquid-filled).
 - Pressure grouting of fractures.
- Heavy Objects--Heavy objects that are mobile and could therefore cause bodily injury by crushing may include:
 - Concrete buckets on cable ends used for pouring seal material into shafts or boreholes.
 - Borehole drills, concrete mixers, front end loaders, and other machinery.
- Noise--Operations that could cause excessive or loud noises may include:
 - Drilling of boreholes.
 - Mechanical compaction of earthen seals.
 - Pneumatic emplacement of seal material.
 - Venting of pressurized gases.
 - Operation of heavy equipment or machinery.
- Dust--Dust hazards can be generated from pneumatic stowing of backfill or earthen seal material, the emplacement and testing of earthen seals containing bentonite clay, or the mixing of concrete/grout.
- Lighting--Lack of lighting or improper lighting can be hazardous in that it may lead to other hazards/injuries.

5.1.2 Mitigation of Hazards

The following discussion addresses major issues in mitigating hazards.

- Protective Clothing and Equipment--Safety glasses, ear protection, and steel-toed shoes should be worn at all times in the test area. Safety glasses, steel-toed shoes, and hard hats should be worn at all times in the shafts or underground facility.
- Ground Support--This safety hazard/concern will likely be addressed by the existing site-specific safety procedures and regulations for mining and tunneling.
- Ventilation--The ventilation system for the underground facility should be designed to deliver an adequate airflow to test areas or closed-off drifts through the use of auxiliary fans and vent tubing. All nonessential personnel must be removed from the affected areas during testing.
- Electrical Shock--Electrical wiring should be visually inspected for fraying and insulation prior to each test as well as on a regular schedule. Electrical equipment should not be used in wet areas unless the equipment is specifically designed to function safely in such an environment.
- Fire Prevention--The emplacement and testing of seal components introduces minimal additional fire hazards. Existing site-specific safety procedures are likely to be sufficient.
- Chemical--Mitigation of possible chemical hazards associated with tracers and seal material include the following activities:
 - Selection of a nontoxic, noncorrosive, noncaustic tracer for the testing, if possible. If this is not possible, the tracer safe-handling procedures found on Material Safety Data Sheets should be used.
 - Avoiding prolonged skin contact with concrete, grout, or bentonite earthen seals. Seal emplacement personnel should wear long sleeves and gloves during seal emplacement. Emergency showers should be available nearby.
- Gas Pressure--The mitigation of hazards associated with high gas pressures during testing include the following activities:
 - Periodically pressure-test and check high-pressure gas systems for leaks (before each test as well as on a regular schedule). Systems should also be pressure-tested under safety-controlled conditions (in a pressure test chamber) to 1.5 times maximum working pressure before first use. Gas-filled packers should have certification of being pressure-tested by the manufacturer to 1.5 times the maximum working pressure before being used. Guides to the safe operation of high-pressure

gas bottles and regulators should be followed. Most laboratory or operations safety manuals include such guides, and the supplier of the gas bottle can also provide them.

- Fluid Pressure--The mitigation of hazards associated with high fluid pressures during both testing and pressure grouting of fractures include the following activities:
 - Periodically pressure-test and check high fluid-pressure systems for leaks (before each test as well as on a regular time schedule). Systems should also be pressure-tested under safety-controlled conditions (in a pressure test chamber) to 1.5 times maximum working pressure before first use. Fluid-filled packers should have certification of being pressure-tested by the manufacturer to 1.5 times the maximum working pressure before being used.
 - Employ either the safety procedures (if available) of the manufacturer of the pressure grouting equipment or site-specific safety procedures.
- Noise--Ear plugs or headphones should be worn by personnel working in noise hazard areas.
- Dust--Dust masks and safety goggles should be worn by personnel working in dust hazard areas.
- Lighting--Proper lighting should be available in all work locations. Portable lighting should be made available if permanent lighting is inadequate or unavailable.
- First Aid--First aid should be easily available to all work areas. This subject is covered by site-specific procedures.

5.2 Quality Assurance Considerations

All activities performed to support field testing of sealing components, including analysis, field test design, and seal placement, will be controlled in accordance with applicable elements of the SNL Quality Assurance Program. Technical and experimental procedures and study plans will be prepared to support the fielding of the tests. While not inclusive, some areas where Quality Assurance requirements will be applied in the field testing program for sealing include:

- Control of measuring and test equipment.
- Identification and control of samples.
- Test control.

- Numerical analysis control.
- Change/nonconformance control.
- Inspection.

6.0 SCHEDULE

The broad range of tests proposed in this report are intended to:

1. Reduce uncertainties associated with the performance and emplacement of sealing components, and
2. Maintain flexibility in resolving sealing issues.

Flexibility is required to address several site uncertainties, including presence or absence of freely flowing water in the underground facility, the significance of faults and other discontinuities, and hydrologic properties of the rock.

For the ten categories of proposed tests, numerous activities required to field the tests have been identified. The activities can be categorized into several areas: characterization of the host rock in the vicinity of the test, laboratory testing to support the field tests, pretest activities, and activities associated directly with the testing of the sealing components. In developing the schedule for each test, it is assumed that the time duration associated with the characterization of the host rock and laboratory testing was less than the time allocated to the pretest activities. The pretest activities are generally assumed to take longer than individual tests. The pretest activities include:

- Performing numerical analyses.
- Developing engineering work packages and test procedures.
- Procuring equipment, instruments, and specialty services.
- Performing reliability tests on the instruments.
- Assembling the instruments.
- Shipping the equipment and instruments to the site.
- Preparing the site.

For the ten categories of testing defined, several tests are proposed. The number of tests shown in Table 6-1 are used to define the test duration schedules in Table 6-2.

Table 6-1. Proposed Tests for the Ten Categories of Seal Component and Performance Confirmation Testing

Major Test Category	Proposed Tests in Each Category
1. Small-Scale In Situ Test	Ten tests in both welded and nonwelded tuff environments--six tests with a diameter of 0.46 m and four tests with a diameter of 0.91 m.
2. Intermediate-Scale Borehole Seal Test	Four tests in both welded and nonwelded tuff environments.
3. Fracture Grouting Test	Three phases of testing in fractured tuff. The first phase is testing conducted in individual boreholes, the second phase is testing line grout patterns, and the third phase is testing multiple grout lines specifically associated with shaft grouting. Phases II and III are conducted concurrently.
4. Surface Backfill Test	Nine tests on welded and nonwelded tuff rockfills--eight tests for partial and complete drift placement and for mechanical and pneumatic stowing and a ninth test to evaluate the seepage effects in rockfill.
5. Grouted Rock Mass Test	One test.
6. Seepage Control Test	Two major tests--one test in the Tiva Canyon Member and a validation test in the Topopah Spring Member.
7. Backfill Test	Two tests--one test in the Calico Hills unit and an optional test in the Topopah Spring Member.
8. Bulkhead Test in the Calico Hills Unit	One test.
9. Large-Scale Shaft Seal and Shaft Fill Test	Four tests--two tests in nonwelded tuff and two tests in welded tuff.
10. Remote Borehole Sealing Test	Three tests in a single remote borehole.

Table 6-2. Estimated Test Durations for Seal Component and Performance Confirmation Tests Under Assumed Conditions

Test	Estimated Test Durations ^a		
	Basic ^b	Delayed ^c	Accelerated ^d
1. Small-Scale In Situ Test			
• welded	3y7m	3y7m	3y7m
• nonwelded	3y7m	3y7m	3y7m
2. Intermediate-Scale Borehole Seal Test			
• welded	2y4m	2y4m	1y5m(PT) 11m (T)
• nonwelded	2y4m	2y4m	1y5m(PT) 11m (T)
3. Fracture Grouting Test			
• welded	3y8m	3y8m	3y8m(PT+T)
4. Surface Backfill Test			
• welded and nonwelded	5y4m ^e	5y4m	3y4m ^f
5. Grouted Rock Mass Test			
• welded	2y	2y	1y5m(PT) 7m (T)
<p>Note: ^ay = years; m = months; PT = pretesting activities; T = testing activities.</p> <p>^bBasic duration of individual tests.</p> <p>^cDelayed duration means that performance confirmation testing of selected tests will be delayed until after LA. These tests are the seepage control test in the Topopah Spring Member, the large-scale shaft seal and shaft fill tests (one in welded tuff and one in nonwelded tuff), and the bulkhead test in the Calico Hills unit.</p> <p>^dAccelerated durations means that pretesting activities of some tests will be performed concurrently with the other tests to accelerate the testing schedule.</p> <p>^eTesting of welded and nonwelded rockfill is performed in parallel. A surface-based test facility is available. The original duration was reduced from 7y2m to 5y4m by performing the tests in parallel.</p> <p>^fNo surface-based test facility is developed. Two additional drifts are required to perform the necessary tests.</p>			

Table 6-2. Estimated Test Durations for Seal Component and Performance Confirmation Tests Under Assumed Conditions (Continued)

Test	Estimated Test Durations ^a		
	Basic ^b	Delayed ^c	Accelerated ^d
6. Seepage Control Test	3y	1y9m(Pre-LA) 1y3m(Post-LA)	2y (PT) 1y (T)
7. Backfill Test			
• welded	2y9m	2y9m	1y10m(PT) 11m (T)
• nonwelded (optional)	2y9m	2y9m	1y10m(PT) 11m (T)
8. Bulkhead Test in the Calico Hills Unit	3y6m	3y6m(Post-LA)	2y5m(PT) 1y1m(T)
9. Large-Scale Shaft Seal and Shaft Fill Test			
• welded	4y4m	3y2m(Pre-LA) 2y2m(Post-LA)	3y2m(PT) 1y6m(T)
• nonwelded	4y4m	3y2m(Pre-LA) 2y2m(Post-LA)	3y2m(PT) 1y6m(T)
10. Remote Borehole Sealing Test	3y	3y	1y10m(PT) 1y2m(T)
<p>Note: ^ay = years; m = months; PT = pretesting activities; T = testing activities.</p> <p>^bBasic duration of individual tests.</p> <p>^cDelayed duration means that performance confirmation testing of selected tests will be delayed until after the LA. These tests are the seepage control test in the Topopah Spring Member, the large-scale shaft seal and shaft fill tests (one in welded tuff and one in nonwelded tuff), and the bulkhead test in the Calico Hills unit.</p> <p>^dAccelerated durations means that pretesting activities of some tests will be performed concurrently with the other tests to accelerate the testing schedule.</p>			

Figures 6-1 and 6-2 present the basic and accelerated schedules for sealing and backfill tests, respectively.

Figure 6-1 shows the overall structure of the field testing program. This figure displays the progressive component evaluation method associated with the testing program. For example, a fundamental understanding of seal component performance is required in areas such as small-scale borehole seals, fracture grouting, and rockfill, to field other sealing tests. For all tests, a test duration was derived. Using the logic of Figure 6-1 and the time needed to complete each of the ten test categories, a total time for the field test program was estimated, as given in Table 6-3. Because of the time needed to complete the testing, alternative strategies were considered in the event that shortening the duration is required. Two alternative strategies are:

- Strategy 1--Split some of the testing between the pre-LA and the post-LA periods. This delays complex testing. The tests delayed include the seepage control test in the Topopah Spring Member; the bulkhead test in the Calico Hills unit; and two of the four large-scale shaft seal and shaft fill tests.
- Strategy 2--Accelerate the testing program by performing pretest activities of some tests with test activities from other tests. Part of this strategy included the use of drifts to conduct the surface backfill tests.

Little reduction in time is achieved by implementing Strategy 1, although it does have the benefit of proceeding cautiously to increase the likelihood of successful completion of the testing program. Strategy 2 reduces an estimated 12-yr program to a 6-yr program. However, the shorter program requires that results from portions of the testing program are incorporated into follow-on tests with little or no difficulty. Because tests of this nature have not been performed previously, a realistic duration of the testing program is somewhere between the accelerated durations and the basic durations. The overall testing program schedule will be refined when access to the underground testing facility is obtained.

Figure 6-1. Basic Schedule for Seal Component and Performance Confirmation Tests

Table 6-3. Estimated Total Durations for Testing Seal Components

Test	Estimated Total Field Testing Durations ^{a,b}		
	Basic	Delayed	Accelerated
Large-Scale Shaft Seal and Shaft Fill Test	10y5m	9y2m(Pre-LA) 2y2m(Post-LA)	6y1m
Remote Borehole Sealing Test	9y	9y	5y9m
Seepage Control Test	8y8m	7y6m(Pre-LA) 1y3m(Post-LA)	5y3m
Bulkhead Test in the Calico Hills Unit	11y7m	8y1m(Pre-LA) 3y6m(Post-LA)	5y4m
<p>^aThe total times needed to complete the tests are associated with specific tests. See Figure 6-1 for the basic and delayed test logic and Figure 6-2 for the accelerated test logic.</p> <p>^bThe estimated times do not include time to prepare and approve study plans, time to integrate results with posttest evaluation activities, or time to document results.</p>			

APPENDIX A
INFORMATION NEEDS FOR THE SEALING PROGRAM

APPENDIX A

INFORMATION NEEDS FOR THE SEALING PROGRAM

Table A-1. Hydrologic-Related Site Parameters

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (DOE, 1988) (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Saturated hydraulic conductivity of alluvium	Within 75 m of shaft and borehole locations	1×10^{-5} to 10^2 cm/s	Medium	Low	1×10^{-2} to 10^{-3} cm/s	8.3.1.2.2.1.1 8.3.1.2.2.3.1 (Ali Phases) 8.3.1.2.2.3.2	Characterization of hydrologic properties of surficial material	Medium	(a)
							Matrix hydrologic properties testing Site vertical borehole studies		
Saturated bulk rock hydraulic conductivity of Tiva Canyon Member		1×10^{-5} to 1×10^{-2} cm/s	Medium	Low	1.2×10^{-3} cm/s	8.3.1.2.2.4.4 8.3.1.2.2.3.2	Radial borehole tests in the exploratory shaft facility Site vertical borehole studies	High	(b)
Morphology of bedrock surface	In order of priority: downgradient and about 160 m from shaft or ramp locations	Determine contours to within ± 3 m	Medium	Low	Contours accurate to 3 m	8.3.1.14.2.1	Alluvium and rock stratigraphy and structure	High	(c)
Thickness of alluvium	Within 75 m of shaft or ramp locations	Determine thickness to within $\pm 10\%$	Medium	Low	0 to 10 m	8.3.1.2.2.1.1 8.3.1.14.2.1 8.3.1.14.2.1.2 8.3.1.14.2.1.3	Characterization of hydrologic properties of surficial material Alluvium and rock stratigraphy and structure Preliminary exploration Detailed exploration	High	(d)
Quantity of water due to surface flooding events (100- and 500-yr floods and probable maximum flood [PMF]), including area of inundation and debris load of flows	Determine parameter at shaft, ramp, and borehole locations	Inundation maps with elevation of inundated area to within ± 2 m	Medium	Low	(See Figure 6-8 of SCP)	8.3.1.2.1.2 8.3.1.16.1.1.1	Characterization of runoff and stream flow Site flood and debris hazards studies	High	(e)
							Site flood and debris hazards studies Transport of debris by severe runoff		
		Estimates of debris quantity and category	Low	Low		8.3.1.16.1.1.1 8.3.1.2.1.2.2		High	(f)

Table A-1. Hydrologic-Related Site Parameters (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Continuous saturation profile of alluvium to bedrock-alluvium interface	At shaft and borehole locations	Determine topography of drainage area using 2-m contours	Medium	Medium		8.3.1.1.14.1	Studies to provide the topographic characteristics of potential locations of surface facilities	High	(g)
		$\pm 10\%$ of natural saturation every m	Medium	Low	TBD	8.3.1.2.3.1 8.3.1.16.1.1.1	Characterization of the site saturated-zone groundwater flow system Site flood and debris hazards studies	Medium	(h)
			Medium	Low		8.3.1.14.2.1.2 8.3.1.2.2.1	Preliminary exploration Characterization of unsaturated-zone infiltration	Medium	(i)
Gradation of alluvium	At shaft locations predominantly within 15 m from shaft and borehole locations	Determination through standard sieving analyses	Not applicable	Low	Soils classified as GP to GM	8.3.1.2.2.1.1	Characterization of hydrologic properties of surficial material	Medium	(i)
Extent and hydraulic conductivity of the MPZ	MPZ in Tiva Canyon and Topopah Spring unit	Less than or equal to 60 times the undisturbed, rock mass hydraulic conductivity (saturated), averaged over one radius from the wall of the shaft	Medium	Low	1 to 20 times undisturbed, rock mass hydraulic conductivity	8.3.1.14.2.1.2 8.3.1.14.2.1.3 8.3.1.2.2.4.4 (All Phases)	Preliminary exploration Detailed exploration Radial borehole tests in the ESF	High	(i)
			Medium	Low		8.3.1.2.2.3.1 8.3.1.2.2.3.2	Matrix hydrologic properties testing Site vertical borehole studies	High	(k)

Table A-1. Hydrologic-Related Site Parameters (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Drainage capacity	Topopah Spring unit at selected drift floor locations at potential repository horizon	Saturated bulk rock hydraulic conductivity $k_{SAT} > 1 \times 10^{-5}$ cm/s	High	Low	1.2×10^{-3} cm/s	8.3.1.2.2.3.1	Matrix hydrologic properties testing	Medium	(l)
Drainage capacity	Topopah Spring unit at base of shafts	Saturated bulk rock hydraulic conductivity $k_{SAT} > 1 \times 10^{-5}$ cm/s	High	Low	1.2×10^{-3} cm/s	8.3.1.2.2.3.1 (All Phases)	Matrix hydrologic properties testing	Medium	(m)
Saturated bulk rock hydraulic conductivity	Calico Hills unit at base of ES-1 and in boreholes	Saturated bulk rock hydraulic conductivity $k_{SAT} > 1 \times 10^{-5}$ cm/s	Low	Low	2.4×10^{-4} cm/s	8.3.1.2.2.4.6 (Phases 2 & 3) 8.3.1.2.2.4.4 (All Phases)	Calico Hills test in the ESF Radial borehole tests in the ESF	High	(n)
Magnitude of water entering shafts		< 150 m ³ /yr per shaft considering anticipated processes	Low	Low	0 to 100 m ³ /yr	8.3.1.16.1.1.1 8.3.1.2.2	Site flood debris hazards studies Studies to provide a description of the unsaturated-zone hydrologic system at the site	Low	(o)
Erosion potential		< 1 m per 1,000 yr preferential erosion at shaft entry points	Low	Low	< 40 cm per 1,000 yr preferential erosion at shaft entry points	8.3.1.16.1 8.3.1.6.1 8.3.1.6.2	Flood recurrence intervals and levels at potential locations of surface facilities Studies to determine present locations and rates of surface erosion Potential effects of future climatic conditions on locations and rates of erosion	High	(p)
TPP 91-5 - Test Planning Package 91-5 Phase 1 - North Ramp Access Phase 2 - Calico Hills									
Phase 3 - Planned Exploratory Studies Facility Tests TSu2 - Topopah Springs welded tuft unit 2 ES-1 - Exploratory Shaft-1									

Table A-2. Miscellaneous Information

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Compressive strength of rock mass	Tiva Canyon at shaft and ramp locations	No more restrictive than for issue 1.11	Medium	Low	See Table 6-12 of SCP	8.3.1.15.1.3.1	Compressive mechanical properties of intact rock at baseline experiment conditions	High	(g)
	Topopah Spring unit	No more restrictive than for issue 1.11	Medium	Low	See Table 6-12 of SCP				
	Calico Hills unit	No more restrictive than for issue 1.11	Medium	Low	See Table 6-12 of SCP				
In situ stresses	In Tiva Canyon, Topopah Spring unit, and Calico Hills unit	Vertical stress accurate to ± 1 MPa Horizontal stress accurate to ± 2 MPa	Low	Low	4 to 10 MPa Horizontal to vertical ratio 0.3 to 1.0	8.3.1.15.2.1.1 8.3.1.15.2.1.2 (Phases 2&3)	Inelastic strain recovery experiments in core holes Overcore stress experiments in the ESF	High	(r)
	At shaft and ramp location (surface and repository horizon) At selected location in underground facility In Calico Hills unit for boreholes within boundary of the underground facility	TBD through design studies	High	Low	Acceleration $< 0.65g$	8.3.1.8.2.1.5 8.3.1.17.3.5	Assessment of postclosure ground motion in the subsurface Ground motion at the site from controlling seismic events	Medium	(s)

Table A-2. Miscellaneous Information (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Meteorological Environment									
Temperature variations at ground surface	At ground surface at shaft and ramp entry points	TBD by laboratory testing and activities	TBD by laboratory and design activities	Low	-14 to 114°F	8.3.1.2.1.1	Characterization of meteorology for regional hydrology	High	(t)
pH of rainfall	At shaft and ramp location	>4.5	Medium	Low	pH > 6	8.3.1.12.1.1	Characterization of the regional meteorological conditions	Low	(u)
Chemistry of Alluvium									
Dissolved sulphates SO_4^{2-}	At end upgradient from shaft location	<0.10% soluble SO_4^{2-} in soils or surface water <150 ppm dissolved SO_4^{2-}	Low	Low	<15 mg/L (see Table 3-3 of SCP)	8.3.1.3.1.1	Groundwater chemistry model	Medium	(v)
pH of alluvium	At end upgradient from shaft location	>4.5	Medium	Low	>7 (see Table 3-3 of SCP)	8.3.1.3.1.1	Groundwater chemistry model	Medium	(v)
Geochemistry	Tiva Canyon, Topopah Spring unit, Calico Hills vitric, and Calico Hills zeolitic	No more restrictive than for issue 1.1	Medium	Low	See Section 4.1.1 of SCP	8.3.1.3.2.1.2 (Phases 2 & 3)	Mineral distributions between the host rock and the accessible environment	High	(w)
Maximum temperature at seal locations	Upper portion of shaft	<90° C	High	Low	<90° C	8.3.2.2.6	Repository thermal loading and predicted thermal and thermo-mechanical response of the host rock	High	(x)
	At repository horizon around shaft	TBD through design trade-off studies			<115° C				

Table A-2. Miscellaneous Information (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Thermal expansion, heat capacity, and thermal conductivity of seal emplacement environment	At selected drift locations in repository horizon	TBD through design trade-off studies		Low	<150° C				
	Calico Hills unit in boreholes below the repository	<90° C	High	Low	<90° C				
	Topopah Spring unit Calico Hills unit	TBD through design trade-off studies	Medium	Low	See Table 6-16 of SCP	8.3.1.4.2.2 (All Phases) 8.3.1.15.1.1 (All Phases) 8.3.1.14.2	Characterization of the structural features within the site area Laboratory thermal properties Studies to provide soil and rock properties of potential locations of surface facilities	High	(y)
Shaft and Ramp Fill Properties									
Saturated hydraulic conductivity		1x10 ⁻² cm/s	High	Low	1x10 ⁻² to 1x10 ⁻⁶ cm/s	8.3.3.2.2.1.2	Hydraulic conductivity and consolidation testing of crushed tuff		
Gradational analyses, angle of internal friction, compressibility		TBD through design trade-off studies	High	Low	Not applicable	8.3.3.2.2.1.1	Detailed property determination of cementitious-based and earthen material	High	(z)
						8.3.3.2.4.1	Development of the advanced conceptual design (ACD) for sealing		

Table A-2. Miscellaneous Information (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Fracture characteristics	Tiva Canyon Topopah Spring unit Calico Hills unit PTn	<20 fractures/m <40 fractures/m <5 fractures/m <10 fractures/m	High	Low	<20 fractures/m <40 fractures/m <5 fractures/m <10 fractures/m	8.3.1.4.2.2 (All Phases)	Characterization of structural features within the area	High	(aa)
Chemistry of waters (if any) in fault including sediment content		Elemental concentrations similar to those contained in Table 4-6	Medium	Low	See Table 4-6 of SCP	8.3.1.3.1.1	Groundwater chemistry model	High	(bb)
Grade of emplacement drifts and dimensions	In repository	1 to 10%	Not applicable	High	1 to 10%	8.3.2.5.8	Identification of technologies for surface facility construction, operation, closure, and decommissioning Reference postclosure repository design	High	(cc)
Casing location and condition for exploratory boreholes	All boreholes	Location of casing to ± 5 m Conditions determined by logging and drilling records	High	Medium	See Fernandez and Freshley (1984)	8.3.2.2.7 8.3.3.2.4	Reference design of seals for shafts, drifts, and boreholes	Low	(dd)
Unit contacts in exploratory boreholes	All boreholes	Contact location ± 5 m	High	Low	Not applicable	8.3.1.4.2.1	Characterization of the vertical and lateral distribution of stratigraphic units within the site area	Medium	(ee)
TPP 91-5 - Test Planning Package 91-5 Phase 1 - North Ramp Access Phase 2 - Calico Hills Phase 3 - Planned Exploratory Studies Facility Tests									

Table A-3. Additional Requirements

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Characteristics of hydrologic flow in fault zones Moisture flux beneath the repository horizon	Flux occurs through matrix and not fractures	Isotropic, homogeneous flow	High	Low	Isotropic homogeneous flow	8.3.1.2.2.4.1 (All Phases)	Drift to imbricate normal fault zone	Medium	(ff)
		<0.5 mm/yr	High	Low	<0.5 mm/yr	8.3.1.2.2.4.1 (All Phases) 8.3.1.2.2.1.2	Drift to the Ghost Dance Fault Intact fracture test in the ESF Evaluation of natural infiltration percolation tests in the ESF	High	(gg)
						8.3.1.2.2.4.2 (Phases 2 & 3)			
Infiltration rate for unanticipated conditions	As for both faults and rock matrix	1.0 mm/yr	High	Medium	1.0 mm/yr	8.3.1.2.1.1	Characterization of meteorology for the regional hydrology Characterization of unsaturated-zone infiltration	Medium	(hh)
Future climatic conditions	Conditions assumed to remain the same as present day	PMF and 500-yr flood volumes	Medium	Medium	PMF and 500-yr flood volumes	8.3.1.2.2.1 8.3.1.5.1.1 8.3.1.5.1.6	Characterization of modern regional climate Characterization of the future regional climate and environments Characterization of the future regional hydrology due to climate changes	High	(ii)
						8.3.1.5.2.2			
Thermal effects of water saturation and desaturation		Will balance out over the life of the repository	Medium	Low		8.3.4.2.4.3.1 (Phase 3) 8.3.4.2.4.4.2 (All Phases)	Waste package environment temperature field analysis Repository horizon rock-water interaction	High	(jj)
		No intersections	High	Low	No intersections	8.3.1.4.2.3 8.3.1.17.4.7	Three-dimensional geologic model Subsurface geometry and concealed extensions of Quaternary faults at Yucca Mountain	High	(kk)

Table A-3. Additional Requirements (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section (TPP 91-5)	SCP Activity/Study	Confidence-Information Delivery	Remarks
Fault recharge rate		0.5 mm/yr	High	Low	0.5 mm/yr	8.3.1.2.2.4.1 (All Phases)	Hydraulic properties of major faults encountered in main test level of the ESF	High	(II)
Fault zone hydraulic characteristics	Lateral hydraulic conductivity	Impermeable	Moderate	Low	Some transverse conductivity 10^{-5} to 10^{-2} cm/s	8.3.1.4.2.2.4 (All Phases)	Geologic mapping exploratory shaft and drifts		
	Hydraulic conductivity		High	Low		8.3.1.2.2.4.1	Hydraulic properties of major faults encountered in main test level of the ESF		
	Porosity		High	Low	0.004 to 0.4				
Saturation of alluvium	Rate of saturation by runoff	Alluvium becomes saturated instantaneously during a flooding event	High	Low	0 to fully saturated	8.3.1.2.2.1.3	Evaluation of artificial infiltration	High	(mm)
Surface water supply	Defined by thunderstorms	4 thunderstorms/yr > 1.3 cm lasting for 1 hr	High	Low	4 thunderstorms/yr > 1.3 cm lasting for 1 hr	8.3.1.2.1.2.1	Surface-water runoff monitoring	Medium	(mm)
TPP 91-5 - Test Planning Package 91-5 Phase 1 - North Ramp Access Phase 2 - Calico Hills Phase 3 - Planned Exploratory Studies Facility Tests									

Table A-4. Waste Package Performance

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section	SCP Activity/Study	Confidence-Information Delivery	Remarks
Waste packages and fuel rods	Failure rate of waste packages	Constant failure rate with all packages failed by 2,300 yr after closure	Low*	Low	Failure begins at 300 years after closure at 0.1%/yr		Table 8.3.5.9-1 from p. 8.3.5.9-9 indicates that the tentative SCP goal is that the fraction of waste containers failed at the end of 300 yr is 0.05. Performance goal after 300 yr is <20% failure up to 1,000 yr after closure.	Low	(oo)
	Fuel rod failure at time of closure	0.02%	Low*	Low	0.02%	8.3.5.10.3.5.1 8.3.5.10.4.1	Development of system model Deterministic calculation of releases from the waste package Probabilistic calculation of releases from the waste package	High	
	Fuel rod failure rate after closure	0.01%/yr	Low*	Low	0.01%/yr	8.3.5.10.3.5.1 8.3.5.10.4.1	Development of system model Deterministic calculation of releases from the waste package Probabilistic calculation of releases from the waste package	High	
	Failed waste packages and fuel rods distribution	Uniformly distributed in repository	Low* to Medium	Low	Statistical variation	8.3.5.10.4.2	Assumption not subject to test but may be subject to distribution/operative solution	Medium	(pp)

Table A-4. Waste Package Performance (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section	SCP Activity/Study	Confidence-Information Delivery	Remarks
Corrosion rate of fuel rod cladding	Oxidation rate of Zircaloy cladding	<0.0469 $\mu\text{m}/100,000 \text{ yr}$	Medium	Medium	0.0469 μm to 17 $\mu\text{m}/10,000 \text{ yrs}$	8.3.5.10.2.1.3	Corrosion of Zircaloy	High	(qq)
	Release of C-14, Ni-59, and Zr-93 is controlled by Zircaloy cladding corrosion		Medium	High		8.3.5.10.2.1.5	Evaluation of the inventory and release of carbon-14 from Zircaloy cladding	High	(rr)
Dissolution of fuel rod material	Matrix dissolution	Congruent	High	Medium	Congruent to preferential depending upon degree of migration to grain boundaries	8.3.5.10.2.1.1	Dissolution and leaching of spent fuel	High	(ss)
	0.5% of the Cs-135, Cs-137, I-129, and Sr-90 are contained in gap/boundaries	Released in the year that the fuel rods fail	High	Low	0.05% of the inventory of Cs-135, Cs-137, I-129, and Sr-90 released	8.3.5.10.2.1.1 8.3.1.3.1.1	Dissolution and leaching of spent fuel Groundwater chemistry model	High	(tt)
	Preferential dissolution factor for Tc-99	Assumed to be a factor of 10 relative to bulk of the matrix	Low	Low to Medium	Probably 3.8 to 8.7	8.3.5.10.2.1.1 8.3.1.3.1.1	Dissolution and leaching of spent fuel Groundwater chemistry model	High	
	Solubility limit of the uranium matrix	<5 mg/l	High	Medium	<5 mg/l	8.3.5.10.2.1.1 8.3.1.3.1.1	Dissolution and leaching of spent fuel Groundwater chemistry model	High	(uu)

Table A-4. Waste Package Performance (Continued)

Performance or Design Parameters	Modifiers	Tentative Parameter Goal	Needed Confidence	Current Confidence	Expected Parameter Values	SCP Section	SCP Activity/Study	Confidence-Information Delivery	Remarks
Development of gaseous radionuclides	Glass waste forms	Will not contribute disproportionately	Low	High	Same to less contribution	8.3.5.10.2.2.1 8.3.5.10.2.2.2 8.3.5.10.3.4	Leach testing of glass Material interactions affecting glass leaching Generate models for release from glass waste forms	High	(vv)
	When waste packages fail	0.3 % of C-14 released as a gas	Low to Medium	Low to Medium	0.3 % of the C-14 is released as gas	8.3.5.10.3.5.1 8.3.5.10.4.1 8.3.5.10.4.2	Development of system model Deterministic calculation of releases from the waste package Probabilistic calculation of releases from the waste package	Medium	(ww)
*These information needs are rated Low because the assumptions were made for anticipated conditions, whereas performance allocations for waste package and fuel rod release were calculated using unanticipated conditions.									

APPENDIX A

REMARKS TO PARAMETER TABLES

- (a) It is recommended that SCP Activity 8.3.1.2.2.1.1, Characterization of hydrologic properties of surficial material, should include a specific requirement that the hydraulic conductivity of the alluvium should be determined within 75 m of all shaft/ramp locations. Standard infiltration tests in conjunction with studies of the alluvium distribution with respect to the shaft/ramp locations should be used for the determinations.
- (b) The saturated bulk rock hydraulic conductivity of the Tiva Canyon Member will be investigated over the area of the site during the site vertical borehole studies. The unit will be investigated in greater detail at one location during the radial borehole tests in the EGF where holes are planned to characterize the top and bottom of the Tiva Canyon Member.
- (c) The morphology of the bedrock surface is required for a number of different purposes, and the proposed studies will require maps of that surface to be constructed. Control within the immediate areas of the potential repository facilities should be satisfactory.
- (d) As in (c) above, the morphology and thickness of alluvium will be determined during the course of several activities. The immediate vicinity of the shaft locations will be investigated during the geotechnical design of the shaft/ramp facilities.
- (e) The data requirements will be met through the planned studies from the available data base. Ideally, the inundation maps should be updated as more monitoring data becomes available and as the historical data base is improved.

It is recommended that the inundation maps be continuously updated as more runoff and precipitation data becomes available. Provision should be made within the SCP Activities 8.3.1.2.1.2, Characterization of runoff and stream flow, and 8.3.1.16.1.1.1, Site flood and debris hazards studies, for continued revision of the inundation maps.

- (f) The planned studies will supply the necessary information requirements through the activity studying the transport of debris by severe runoff. Additional information will be generated by studies of the site flood and debris hazards studies.
- (g) The study addressing this information need is completed.
- (h) The framework for satisfying the information needs is already incorporated within existing test activities. Mention of investigations of continuous-saturation profiles in the vicinity of shaft/ramp locations is not made, however, and the geophysical logging program should specifically include techniques that yield the required information.

It is recommended that the geophysical logging program include the development of continuous-saturation profiles at shaft locations within existing study activities, primarily preliminary exploration for shaft/ramp development, such as SCP Activity 8.3.1.14.2.1.2, Preliminary exploration, which provides data for design of the shaft locations. The general geophysical borehole investigation program should investigate the alluvium where possible, but may be limited by the presence of casing in the existing boreholes.

SCP Activity 8.3.1.14.2.1.2 should include measurement of continuous-saturation profiles of alluvium from bedrock to the bedrock-alluvium interface. The quality of the data should be sufficient to determine the water saturation within $\pm 10\%$ every m throughout the alluvium. The suggested method of determination is neutron logging calibrated for the alluvium found at the shaft locations if the holes remain stable. Gamma ray logging should also be conducted in conjunction with the neutron logging program to assist in determining sediment types. If possible, the variation in saturation as a function of precipitation should be determined, which would necessitate repetitive runs depending upon precipitation availability.

- (i) The existing study activities will examine the nature of the alluvium over the entire repository area. Special attention should be made to determine the gradation of alluvium within 15 m of the shaft locations.

It is recommended that existing studies be modified, including SCP Activities 8.3.1.14.2.1.2 and 8.3.1.14.2.1.3, to provide data on the gradation of alluvium within 15 m of the shaft locations. The gradation of alluvium and its variation should be established within 15 m of all shaft locations using standard sieve analysis methods.

- (j) One of the stated objectives of the radial borehole tests in the ESF is to examine the zone modified by the blasting and excavation activities.
- (k) The specified activities should adequately define the hydraulic properties of the TSw2 through matrix hydrologic properties testing.
- (l) The planned activity in the ESF involving the matrix hydrologic properties testing provides direct information for this information need. However, it only addresses one location within the repository area. Consequently, more tests of the drainage capacity of the TSw2 may be required.

It is recommended that additional tests of the drainage capacity in selected locations within the potential repository horizon be performed to obtain estimates of areal distribution of the drainage capacity of TSw2. This should be part of the SCP Studies including Section 8.4.2.4.2, Drift to Drillhole Wash structure; Section 8.4.2.4.3, Drift to imbricate normal fault zone; and Section 8.4.2.4.1, Drift to Ghost Dance Fault.

The infiltration properties of the repository are extremely important to the sealing program because water inflow will often be controlled by drainage from sumps behind earthen dams or bulkheads. The long-term behavior of the infiltration properties of the repository horizon may be altered by the addition of fines contained in the backfill. Consequently, an additional test should be designed that details the effect of the addition of fines to the potential repository horizon and the effect upon infiltration rate. It is anticipated to be attached as an extension of the infiltration test of SCP Activity 8.3.1.2.2.3.1, Matrix hydrologic properties testing.

- (m) Information on the drainage capacity at the base of the shafts/ramps is required. It is recommended that infiltration tests be performed at the base of shafts/ramps as they become available for study.
- (n) The radial borehole tests will examine the saturated hydraulic conductivity in the Calico Hills in conjunction with the Calico Hills test in the ESF.
- (o) Planned study activities will provide a basis for calculating the magnitude of water entering the shafts/ramps, but no study activity seems to be charged with the responsibility for actually performing those calculations. The calculations should provide estimates of the magnitude of the water entering the shafts and ramps. The calculations should use information from the SCP Activity 8.3.1.16.1, Flood recurrence intervals and levels at potential locations of surface facilities, for runoff potential and the effects of flooding. Flow system information from SCP Activity 8.3.1.2.2 should provide information detailing the effects of the alluvial hydrologic flow in the vicinity of the shafts. Based upon this information, the yearly inflow per shaft and the yearly inflow into all potential repository facilities should be estimated.
- (p) Studies of the erosion potential in the proposed repository area should adequately cover the particular areas surrounding the facility shafts.
- (q) Efforts to determine the compressive strength of the rocks in the potential repository area will be made in several key studies because of the importance of this parameter to many different SCP activities.
- (r) Tests in the ESF studies involving overcoring to determine the general in situ stress regime and strain recovery experiments are specifically designed to determine the horizontal stresses at the site.
- (s) The seismic hazards analysis program is detailed and is capable of providing guidelines for construction and sealing issues.

- (t) Determination of surface temperatures is an integral part of the general characterization of the regional and local meteorological activities.
- (u) The meteorological monitoring program is a comprehensive program that will probably monitor many parameters including the pH of the rainfall. Currently, the program makes no mention of monitoring the pH of the rainfall.

It is recommended that monitoring rainfall pH be performed along with the other parameters measured during the meteorological monitoring program. Technical procedures for sample collection should be established that ensure the pH of the collected precipitation is not altered during storage and/or transportation. Ideally, procedures should be established that allow measurement of pH at the collection site according to standard methods.

- (v) The groundwater geochemical modeling activities will probably entail measurement of groundwater chemistries from all hydrologic units. Special care should be taken, however, to measure the variation of groundwater chemistries within the alluvium, especially with respect to sulphates and the pH of the groundwater in the alluvium.

It is recommended that specific reference to pH and sulphate measurements be incorporated in the test activities dealing with the groundwater geochemical modeling, specifically SCP Activity 8.3.1.3.1.1, Groundwater chemistry model. The pH measurements and the sulphate measurements should be made according to standard methods.

- (w) Because a number of issues depend upon geochemical parameters, the geochemistry of all hydrologic and lithologic units will be examined in detail by activities of the SCP.
- (x) A thermal model of the potential repository in time and space is planned under the SCP Activity 8.3.2.2.6, Repository thermal loading and predicted thermal and thermomechanical response of the host rock.

- (y) Specific investigations are scheduled that will address this information need, including the SCP Activities 8.3.1.15.1.1, Thermal conductivity characterization, and 8.3.14.2, Studies to provide soil and rock properties of potential locations of surface facilities.
- (z) Shaft and ramp fill material properties and trade-off studies will be conducted to determine the optimum composition to fulfill the required goals for the material.
- (aa) Structural studies of the geologic environment as it is exposed by excavation are a fundamental part of the SCP activities. This continuing effort will contribute to a geological data base.
- (bb) The groundwater geochemistry activities will be especially concerned with the composition of the groundwater at the potential repository site. It is anticipated the groundwater will be sampled in different environments. Nevertheless, a stated objective of the groundwater geochemistry program should be to sample waters contained in fault systems. It is therefore recommended that groundwater geochemistry activities include sampling of waters and any associated sediment in fault zones.
- (cc) Present potential repository planning includes this design specification.
- (dd) Boreholes that represent potential pathways to the accessible environment are currently planned to be sealed. The borehole locations are known, but the condition of the casing is not well determined. Prior to plugging, the condition of the casing should be determined as well as the possibility of pulling the casing to expose the formation for optimum sealing.

It is recommended that activities for determining the casing condition be included in the compilation of borehole construction and geologic logs in SCP Activity 8.3.3.2.4, Reference design of seals for shafts, drifts, and boreholes. The activity should include compilation of existing data on subsurface borehole condition, direct evaluation of casing condition through borehole logging, and evaluation of the potential of pulling the casing.

- (ee) The proposed SCP activities will focus upon the location of contacts within exploratory boreholes. According to the SCP, appropriate contacts will be located, although no specific mention is made of assurance of the contact location to within ± 5 m.
- (ff) Characterization of the hydrology within the fault zones associated with the potential repository is important for evaluation of rates and flow paths. Several activities will directly investigate fault zones, but none appear to specifically include investigations detailing the mechanisms for water movement within those zones, especially with respect to potential flow anisotropy. It is recommended that current planning include the investigation of flow anisotropy in fault zones in and near the repository. This activity could be associated most easily with SCP Activity 8.3.1.2.2.4.10.
- (gg) The mechanism of flow in rocks of the repository is the subject of many of the SCP activities. Activities such as SCP Activity 8.3.1.2.2.1.2 should provide insight into those mechanisms.
- (hh) The activities listed should provide a good data base for resolution of this information need, although the absolute infiltration rates will probably continue to be a concern, due to the uncertainties in measurement techniques. No further activities can be recommended at this time.
- (ii) Given the present state of the historical data base, the planned activities are appropriate to resolution of this information need. As new information becomes available, the PMF and 500-yr-flood volume estimates should be updated.
- (jj) Thermal modeling is an integral part of the SCP and is likely to provide answers to this information need.
- (kk) The three-dimensional geologic model will integrate the available borehole, geophysical, and shaft facility data into a comprehensive geometric model.

- (ll) The hydrologic characteristics of the fault zones will be investigated directly through several subsurface investigations. These investigations should provide the needed information regarding flow in the fault zones, with the possible exception of characterization of anisotropic flow.
- (mm) The rate of saturation of alluvium by runoff should be estimated using the information generated during the evaluation of artificial infiltration, which will examine the infiltration rates of various material occurring at the repository site.
- (nn) The information dealing with precipitation events is limited by the historical data base available, but planned studies will provide additional flood and debris levels and flow rates. As the project progresses, additional runoff data will be collected and will be included in the runoff estimations.

Many SCP activities address fluid flow in the fault zones of the potential repository, but no direct activity appears to address the question of recharge of the Ghost Dance Fault as a function of distribution of alluvium in contact with the fault. This relates directly to the expected recharge rate and should be examined further.

It is recommended that activities be included in the surficial investigations that detail the importance of the overlying alluvium to recharge of the Ghost Dance Fault. This might involve the drilling of short boreholes into the fault zones and areas underlain by alluvium compared to areas barren of overlying alluvial sediments. It would involve determining the shallow flow characteristics of water in the fault zone throughout the limits of the potential repository.

- (oo) The SCP Table 8.3.5.9-1 indicates that the tentative SCP goal is that the fraction of waste containers failed at the end of 300 yr is 0.05 or less.

It is recommended that a brief reevaluation of the assumptions dealing with the performance allotment should be performed using the revised performance goals of the

waste packages. This activity could be incorporated into SCP Activity 8.3.5.10.3.5.1, Development of the system model, or SCP Activity 8.3.5.10.4.2, Probabilistic calculation of releases from the waste package.

- (pp) The studies outlined in the SCP should directly address this information need and validate the assumptions used for performance allocation.
- (qq) The SCP Activity 8.3.5.10.2.1.3, Corrosion of Zircaloy, directly addresses this information need.
- (rr) The SCP Activity 8.3.5.10.2.1.5, Evaluation of the inventory and release of C-14 from Zircaloy cladding, directly addresses this information need.
- (ss) The assumed value of $<5 \mu\text{g}/\ell$ represents the maximum solubility of uranium from fuel rods in J-13 groundwater measured to date. Planned studies on the dissolution and leaching of spent fuel should consider this sufficiently for the purposes of the sealing program.
- (tt) The assumed relative dissolution factor is higher than the probable range of actual preferential dissolution factors measured to date. Planned studies will further reduce this uncertainty.
- (uu) The SCP Activity 8.3.1.3.1.1, Groundwater chemistry model, should include characterization of the solubility limits of uranium. Careful coordination of these efforts with those of SCP Activity 8.3.5.10.2.1.1, Dissolution and leaching of spent fuel, may be important.
- (vv) The activities associated with the leach testing of glass directly addresses this information need.

(ww) The deterministic and probabilistic model development will probably include the evolution of gaseous species including C-14.

APPENDIX B

MODELING OF HYDROLOGICAL PERFORMANCE OF SHAFT AND BOREHOLE FIELD TESTS

APPENDIX B

MODELING OF HYDROLOGICAL PERFORMANCE OF SHAFT AND BOREHOLE FIELD TESTS

B.1 Introduction

An objective in seal design is to develop, either through volumetric or thermal expansion of the seal, a tight interface that will be maintained over a long period. Previous sealing studies (Case et al., 1984, and Kelsall et al., 1982) showed the relative importance of the interface zone under certain environmental conditions.

The proposed series of borehole and shaft sealing experiments assesses the behavior and performance of various seal components under realistic hydrogeological conditions. The testing includes the demonstration of current engineering technology for the emplacement of shaft and borehole seals.

The supporting calculations presented in this appendix evaluate the flow conditions in and around shaft and borehole plugs. The objective of the hydrologic calculations is to perform a preliminary assessment of the expected magnitude of total flow rates in a plug system in the proposed field experiments. Several flow regimes through the plug, plug and rock interface, and the damage zone around the excavation might contribute to flow performance, and it is of interest to assess the dominant flow regimes for each component in the borehole or shaft plugs over a range of permeabilities. This analysis defines the duration of the test and suggests the feasibility of performing the tests for the various assumed permeabilities. Current test plans call for placement of seals 0.5 m to 3.0 m in diameter in welded and nonwelded tuff. Plug length and diameter in the various evaluations were varied to assess the overall hydrologic performance.

B.2 Methods of Analysis

Sensitivity analyses were performed for steady-state and transient conditions, in which pressure was allowed to decay to 90% of initial pressure. There are several potential flow paths for plugs emplaced in borehole or shaft seals. These include the plug, the interface zone between the plug and the rock, the MPZ, and the surrounding undisturbed zone. The following steady-state flow model was developed assuming parallel flow through the plug, the interface zone, and surrounding MPZ, in which the specific discharge is given by Darcy's Law.

For steady-state flow, the total flow rate is given by

$$Q = \frac{\Sigma (K_i A_i) \Delta H}{L}$$

where

Q = Flow rate,

K_i = Conductivity for either the plug, interface, or MPZ,

A_i = Cross-sectional area for either the plug, interface, or MPZ,

ΔH = Hydraulic potential across the plug, and

L = Length of the plug.

A transient test with air was also modeled to evaluate an alternative to the steady-state flow test. A formula adapted by Sutherland and Cave (1980) for pressure decay tests and originally developed by Brace et al. (1968) for laboratory testing of low-permeability rock was adapted:

$$\frac{\Delta P}{\Delta P_o} = e^{-\alpha t_d}$$

$$\alpha = \Sigma (K_i A_i) \frac{1}{LV_s \mu \beta}$$

where

- t_d = Time to decay 90% of pressure pulse,
- ΔP = Difference in pressure between the reservoirs,
- ΔP_o = Initial difference in pressure between the reservoirs,
- V_s = Volume of the smaller reservoir,
- μ = Absolute viscosity of the fluid (air),
- β = Compressibility of the fluid (air), and
- α = Exponential constant, and other terms are as defined previously.

Assuming further that the volume of the downstream reservoir is 10% of the volume of the plug, the time required to decay 90% of the pressure pulse may be expressed as

$$t_d = \frac{-\ln \{(0.10)(0.10L^2A\mu\beta)\}}{\sum (K_i A_i)}$$

where

A = Cross-sectional area of the flow path, and other variables are as defined previously.

B.2.1 Model Input Properties

The material properties and model boundary conditions are presented in Tables B-1 and B-2. A range of values was analyzed for the hydraulic conductivity of the intact rock, the porosity of the MPZ and interface zones, and the degree to which the MPZ is altered. Analyses were performed for boreholes and shafts. The analysis considered water to be the permeant for steady-state conditions and air for transient conditions. Plug lengths and the pressure differential were also varied. The analyses combined such factors as low permeability with long plugs and small pressure differentials to obtain either flow rates for steady-state conditions or the time to reach 90% pressure pulse for transient conditions.

**Table B-1. Yucca Mountain Site Characterization Project Borehole
Parameters--Borehole Model Values**

Parameter Name (unit)	Best Case Values	Medium Case Values	Worst Case Values
Pressure Differential (MPa)	0.07	0.7	1.2
Length of Plug (m)	10	5	0.5
Radius of Plug (m)	0.5	0.5	0.5
Thickness of Interface (m)	1.00×10^{-6}	1.00×10^{-5}	1.00×10^{-4}
Interface Permeability (m^2)	8.33×10^{-14}	8.33×10^{-12}	8.33×10^{-10}
Thickness of MPZ (R)	1	1	1
Hydraulic Conductivity of Intact Rock (m/s)	1.00×10^{-7}	1.00×10^{-5}	1.00×10^{-4}
Density of Water (kg/m^3)	1000	1000	1000
Viscosity of Water (Ns/m^2)	0.001	0.001	0.001
Gravity (m/s^2)	9.81	9.81	9.81
Permeability of Intact Rock (m^2)	1.02×10^{-14}	1.02×10^{-12}	1.02×10^{-11}
Permeability Factor for MPZ	1	7.5	15
Permeability of MPZ (m^2)	1.02×10^{-14}	7.65×10^{-12}	1.53×10^{-10}
Porosity of Interface (%)	100	100	100
Porosity of Plug (%)	26	26	26
Porosity of MPZ (%)	0.02	0.2	2
Viscosity of Gas (Ns/m^2)	2.00×10^{-5}	2.00×10^{-5}	2.00×10^{-5}
Compressibility of Gas (1/Pa)	1.00×10^{-6}	1.00×10^{-6}	1.00×10^{-6}

**Table B-2. Yucca Mountain Site Characterization Project Shaft
Parameters--Shaft Model Values**

Parameter Name (unit)	Best Case Values	Medium Case Values	Worst Case Values
Pressure Differential (MPa)	0.01	0.1	0.1
Length of Plug (m)	5.5	5.5	5.5
Radius of Plug (m)	5.5	5.5	5.5
Thickness of Interface (m)	1.00×10^{-6}	1.00×10^{-5}	1.00×10^{-4}
Interface Permeability (m^2)	8.33×10^{-14}	8.33×10^{-12}	8.33×10^{-10}
Thickness of MPZ (R)	1	1	1
Hydraulic Conductivity of Intact Rock (m/s)	1.00×10^{-7}	1.00×10^{-5}	1.00×10^{-4}
Density of Water (kg/m^3)	1000	1000	1000
Viscosity of Water (Ns/m^2)	0.001	0.001	0.001
Gravity (m/s^2)	9.81	9.81	9.81
Permeability of Intact Rock (m^2)	1.02×10^{-14}	1.02×10^{-12}	1.02×10^{-11}
Permeability Factor for MPZ	20	20	20
Permeability of MPZ (m^2)	2.04×10^{-13}	2.04×10^{-11}	2.04×10^{-10}
Porosity of Interface (%)	100	100	100
Porosity of Plug (%)	26	26	26
Porosity of MPZ (%)	0.02	0.2	2
Viscosity of Gas (Ns/m^2)	2.00×10^{-5}	2.00×10^{-5}	2.00×10^{-5}
Compressibility of Gas ($1/Pa$)	1.00×10^{-6}	1.00×10^{-6}	1.00×10^{-6}

B.3 Analyses Results and Conclusions

The analyses showed both a wide range of potential responses and that actual measurements depend on many factors. Several categories of tests have been identified for field testing of seals. Site characterization testing will determine pertinent hydrologic properties as part of site characterization activities and will lead to refinement of the field test concepts in the selection of appropriate instrumentation. The calculations presented here show that steady-state or falling-head tests are appropriate and that test parameters can be selected to obtain meaningful results for a broad range of in situ conditions.

The results of the analyses are presented in Figures B-1 through B-4. The steady-state plots present worst, middle, and best cases for combinations of parameters that provide the highest or lowest flow conditions. The transient analysis results include plots of the time to reach 90% pressure pulse versus plug permeability, also for the worst, middle, and best case of parameter combinations. Flow rate for boreholes range over seven orders of magnitude, while flow rates for shafts range over four orders of magnitude. The analyses show that when plug permeabilities are low, flow occurs dominantly through the MPZ and, conversely, when plug permeabilities are high, flow occurs dominantly through the plug. For transient testing, the time to reach 90% pressure decay is similarly affected with flow dominantly occurring through the MPZ for low plug conductivities.

The results presented here suggest that a transient test could be performed in a matter of hours under appropriate conditions. The results also suggest the appropriateness of performing steady-state flow measurements.

Figure B-1. Steady-State Flow--Flow Rate vs. Borehole Seal Permeability (see Table B-1 for MPZ and interface properties)

Figure B-2. Steady-State Flow--Flow Rate vs. Shaft Seal Permeability (see Table B-2 for MPZ and interface properties)

Figure B-3. Borehole Test--Time to 90% Pressure Decay (see Table B-1 for MPZ and interface properties)

Figure B-4. Shaft Test--Time to 90% Pressure Decay (see Table B-2 for MPZ and interface properties)

APPENDIX C
ANALYSIS OF FRACTURE GROUTING

APPENDIX C

ANALYSIS OF FRACTURE GROUTING

C.1 Introduction

An important issue to be resolved by the grouted rock mass test is the potential stability of grouted fractures when subjected to repository heating. Grouting in welded tuff may be used for treatment of the MPZs around shafts and ramps, below seepage control structures, and around exploratory boreholes. The following calculations evaluate strength requirements for grouted fractures in support of the grouted rock mass test.

The cases to be evaluated include (1) an exploratory borehole at the Topopah Spring/Paintbrush contact and potential repository horizon (seal locations 1 and 2), (2) seepage control structures in the mains and on the repository periphery, (3) a bulkhead in a panel, (4) a shaft plug at the potential repository horizon (vertical fractures oriented parallel to the shaft axis), (5) a shaft plug at the Tiva Canyon contact, and (6) a shaft station plug at the potential repository horizon (vertical fractures oriented perpendicular to the tunnel axis). The locations of several seals at the potential repository horizon are shown in Figure C-1. Table C-1 presents a series of conditions evaluated for each calculation.

The MPZ around exploratory boreholes or shafts or ramps accessing the repository is an important factor for air or water flow through seals (Case and Kelsall, 1987). The MPZ results in the development of stress concentrations around boreholes, shafts, or ramps that in turn might result in shear failure along joints near the boreholes, shafts, or ramps. Recent work in classifying existing exploratory borehole conditions at SNL shows that sealing in welded tuff may require grouting of fractures subjected to stress relief during drilling. During excavation or drilling, joints dilate, increasing the permeability of the rock mass. Grouting may help to strengthen the rock mass, primarily by reducing the effective fracture apertures and secondarily by increasing the joint strength. The potential increase in rock mass strength may be important if repository heating occurs.

Figure C-1. Locations for Seal Structures

Table C-1. Fracture Grouting Summary

Seal Component	Seal Location	Time (yr)	Temperature (°C)	Far-Field Stress (MPa)	Thermal Stress (MPa)	Support Stress (MPa)
Exploratory Borehole	Upper Contact Zone	2000	41.6	1.38	1.21	0.79
Exploratory Borehole	Potential Repository Horizon	300	87.6	5.46	10.29	2.20
Seepage Control Structure	Potential Repository Periphery	300	56.6	5.46	5.33	0
Seepage Control Structure	Potential Repository Mains	500	57.5	5.46	5.59	0
Panel Bulkhead	Panel	300	87.6	5.46	10.29	1.05
Exploratory Shaft	Potential Repository Horizon	1000	39.4	5.46	2.43	1.05
Station Plug	Potential Repository Horizon	1000	39.4	8.43	2.43	1.05
Exploratory Shaft	Upper Contact Zone	1000	26.2	1.38	0.22	1.05

If the repository starts to accept the nuclear waste, the waste will begin to generate heat. Therefore, the temperature of the rock mass surrounding the repository would increase. Because thermal stresses increase shear and normal stress across joints, the thermal stresses combined with the existing in situ stresses may result in an increase in the stress around the boreholes that in turn results in potential joint failure around boreholes or excavations. Experimental evidence shows that joints dilate if they undergo shear failure. Joint dilation depends on joint stiffness properties and joint normal stress.

For the joints near a borehole or excavation, two cases were studied (Figure C-2). In the first scenario, a joint intersects the borehole at an angle ranging from zero to 90 degrees as illustrated in the figure. In the second case, a joint is near the borehole but not intersecting it (Figure C-2b). The value β represents the ratio of the distance of the joint from the center to the radius of the borehole. In the cases presented subsequently, the value for β is 1.1.

Seal emplacement reduces the stress in the rock mass around boreholes because the interface pressure induced by the plug provides support to the surrounding rock mass. Measures to increase the compressive interface stress in the seal include using expansive cement, injecting the seal under pressure, or developing a contrast in temperature between the seal and surrounding rock mass. A reduction in joint shear stress and an increase in confinement in the surrounding rock mass results.

Parametric studies evaluated different types of cements, different placement temperatures, and different rock temperatures for the surrounding rock mass. The selected design for analysis included a Type II cementitious material, with a placement temperature of 4°C at ambient rock temperature and an injection pressure of 50% of the far-field stress for each seal location. The current strategy assumes that the seals would be emplaced prior to waste emplacement, resulting in an internal pressure for the borehole that reduces the difference between the far-field and internal stresses.

(a)

Note: Orientations of θ from 0° to 90° were considered

(b)

$$\beta = d/a$$

Note: Orientations for α of 0° to 90° were considered

Figure C-2. Proximity of a Single Joint Near a Borehole: (a) Intersecting Case and (b) Nonintersecting Case

C.2 Analysis Methods

The analysis methods are based upon simple closed-form solutions that consider stresses along rock joints near boreholes subject to far-field stress and internal interface stress (Brady and Brown, 1985). Joint normal and shear stresses are calculated and compared for the various cases evaluated (Table C-1).

The following assumptions are used in this study:

- The direction of the principal stresses for the in situ and thermal stresses is the same and the principal stresses can be superimposed.
- For the near-field calculations, the far-field state of stress is hydrostatic and equal to the average value of minimum and maximum principal stresses.
- The behavior of the rock mass is linearly elastic.
- All fractures are parallel to the borehole or excavation axis.
- Changes in temperature and induced thermal stresses at the contact between the Topopah Spring and the Paintbrush formations and the potential repository horizon were calculated from STRESS3D analyses. These analyses assume linearly elastic behavior of a homogeneous rock mass.
- For assessing the grouted rock mass, grouting increases the cohesion of the joints without affecting the coefficient of friction.
- The analysis obtains the frictional properties of the joints from the Reference Information Base (DOE, 1991), which presents minimum, maximum, and recommended values for the shear strength parameters. This study uses the recommended values.

The analysis uses a conservative assumption for the superposition of in situ and thermal stresses to assess the grouting requirements.

Brady and Brown (1985) present the following relations for calculating normal and shear stress on a joint near a borehole:

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$$\sigma_n = P_o + (P_o - P_i)\cos(2\theta)$$

$$\tau_n = (P_o - P_i)\sin(2\theta)$$

where

- θ = Angle of the intersecting joint,
- P_o = External far-field stress,
- P_i = Internal support stress,
- σ_n = Normal stress, and
- τ_n = Shear stress.

Brady and Brown (1985) present the following relations for calculating normal and shear stress on a joint intersecting a borehole:

$$\sigma_n = P_o - (P_o - P_i)\frac{\cos^2(\theta)\cos(2\theta)}{\beta^2}$$

$$\tau_n = (P_o - P_i)\frac{\cos^2(\theta)\sin(2\theta)}{\beta^2}$$

where β = Ratio of the distance from the center of the excavation to the joint divided by the radius, and other terms are as defined previously.

C.3 Assessment of Far-Field Stress

The analysis first considers joints in the far field, where there is no stress redistribution due to excavation effects. The analysis uses the far-field stresses. For the upper contact zones (seal location 1) and the potential repository horizon (seal location 2), two separate far-field stress conditions consisting of in situ stress and in situ and thermal stresses have been studied (Figure C-3).

Figure C-3. State of Stress in the Far Field

A comparison between the far-field or in situ state of stress and the internal friction of the joint shows that joints are stable at the upper seal location and at the potential repository horizon for ambient temperature conditions. After repository heating, the joints would be at incipient failure.¹ A more detailed treatment of joint deformational and strength properties might show an increase in confinement and reduction in shear stress and joint stability.

C.4 Assessment of a Single Joint Near an Exploratory Borehole

Figures C-4 and C-5 present results for single joints intersecting a borehole or next to a borehole, respectively. Various cases of in situ stress and in situ stress plus thermal stresses (as outlined previously) for orientations ranging from 0 to 90 degrees are shown. Shear and normal stresses across joints are lower for seal location 1 than seal location 2. Also, smaller thermal effects at seal location 1 at the upper contact zone are shown than for seal location 2 at the potential repository horizon, as expected. The intersecting fracture case is more severe than the nonintersecting fracture case in exceeding shear strength. For the nonintersecting fracture, the shear stress is higher and confinement is lower near the borehole ($\beta = 1.1a$). In both intersecting and nonintersecting fracture cases, the development of interface stress increases confinement and reduces shear stress, resulting in a more stable seal configuration, as shown by the comparison of shear and normal stress to the joint-failure criteria. For the confined case, a Type II cementitious material is considered with a placement temperature of 4°C at ambient rock temperature with a 50% injection pressure for a 0.2-m-diameter plug.

In conclusion, the results suggest a number of factors influence the performance of fractures. These include the in situ stress, the elevated temperature field, the proximity of the fracture to the borehole, and the effect of the seal in reducing shear stress and increasing confinement along the fracture plane. The greater the effectiveness of the seal in increasing confinement, the less the requirement for joint strength for grouted fractures. The preliminary results presented in this appendix suggest that, for the given levels of confinement analyzed, there is a requirement for increased joint strength through either increased cohesion or increased friction

¹Note that failure is in the context of larger displacements under stress with the rock mass sustaining load.

Figure C-4. Normal and Shear Stresses on a Nonintersecting Joint: (a) Open Borehole and (b) Sealed Borehole

Figure C-5. Normal and Shear Stresses on an Intersecting Joint: (a) Open Borehole and (b) Sealed Borehole

angle. If joint strength through increased cohesion is considered, the results suggest a joint cohesion of 10 MPa.

C.5 Analysis of a Seepage Control Structure

Agapito and Associates performed analyses for underground locations, including the mains and the potential repository periphery (Lin and Hardy, 1990). Figures C-6 through C-9 present the worst-case thermal analyses for large-scale embankments at the potential repository horizon locations. The seepage control structure does not confine the fractures, so it is unnecessary to perform an analysis under confined conditions. Temperatures at these two locations are nearly equivalent, reflecting the effects of the panel-barrier pillar isolating the peripheral drift or mains from the disposal areas.

C.6 Analyses of a Bulkhead in a Panel

A bulkhead within the panel is considered in Figures C-10 and C-11. No specific seal analyses were conducted for a panel bulkhead. These analyses use the thermal analysis for exploratory borehole calculations based upon an exploratory borehole penetrating the panel after 300 years.

For the confined bulkhead case, the analyses consider a Type II cementitious material, a placement temperature of 4°C at ambient repository temperature at the potential repository horizon, and a 50% injection pressure for a 3.05-m-diameter seal. The results are comparable to the results for an exploratory borehole intersecting the potential repository horizon.

C.7 Analyses of Shaft Plugs

Agapito and Associates performed analyses for the ES-1 shaft seal locations at the potential repository horizon (Lin and Hardy, 1990). Figures C-12 through C-15 consider the worst-case thermal analyses for large-scale seals at the potential repository horizon and at the upper contact zone. For the station plug at the potential repository horizon, the vertical fractures are parallel to the axis of the shaft. For the shaft station plug, the vertical fractures are perpendicular to the

Figure C-6. Normal and Shear Stresses on a Nonintersecting Fracture for a Seepage Control Structure on the Potential Repository Boundary

Figure C-7. Normal and Shear Stresses on an Intersecting Joint for a Seepage Control Structure on the Potential Repository Boundary

Figure C-8. Normal and Shear Stresses on a Nonintersecting Fracture for a Seepage Control Structure in the Potential Repository Mains

Figure C-9. Normal and Shear Stresses on an Intersecting Fracture for a Seepage Control Structure in the Potential Repository Mains

Figure C-10. Normal and Shear Stresses on a Nonintersecting Fracture at a Panel Seal Location:
(a) Open Excavation and (b) Sealed Excavation

Figure C-11. Normal and Shear Stresses on an Intersecting Fracture at a Panel Seal Location:
(a) Open Excavation and (b) Sealed Excavation

Figure C-12. Normal and Shear Stresses on a Nonintersecting Fracture for Shaft Plugs:
(a) Open Excavation and (b) Sealed Excavation

Figure C-13. Normal and Shear Stresses on an Intersecting Fracture for Shaft Plugs:
 (a) Open Excavation and (b) Sealed Excavation

Figure C-14. Normal and Shear Stresses on an Intersecting Joint for a Shaft Station Plug:
 (a) Open Excavation and (b) Sealed Excavation

Figure C-15. Normal and Shear Stresses on a Nonintersecting Joint for a Shaft Station Plug:
 (a) Open Excavation and (b) Sealed Excavation

tunnel axis. For the case of a sealed shaft plug, the analyses consider a Type II cementitious material, a placement temperature of 4°C at ambient rock temperature at the potential repository horizon, and a 50% injection pressure for a 3.05-m-diameter seal.

C.8 Conclusions

The higher thermal environment at the potential repository horizon of the exploratory borehole or the panel bulkhead results in the worst-case thermal stresses. A minimum joint cohesion should be 10 MPa. This figure should be multiplied by a safety factor to obtain the grout strength requirement.

APPENDIX D

STRUCTURAL HYDRATION CALCULATIONS

APPENDIX D

STRUCTURAL HYDRATION CALCULATIONS

An objective in seal design is to develop, either through volumetric or thermal expansion of the seal, a tight interface that will be maintained over a long period. Previous sealing studies showed the relative importance of the interface zone under certain environmental conditions (Case et al., 1984, and Kelsall et al., 1982). As the YMP at SNL obtains hydrologic information, the design requirements will be refined. If it becomes necessary to emplace cementitious seals of low conductivity, the interface stress will be evaluated to control shrinkage, hydration, and temperature effects.

D.1 Thermal and Thermomechanical Behavior of Seals

Normal interface stress develops through various stages of construction and potential repository operation, as illustrated in Figure D-1. These stages include the initial mixing and placement, thermal and volumetric expansion, thermal contraction during cooling, backfill emplacement and subsequent deformational response of the plug, and the postclosure period. The following discussion identifies several important design considerations in the two initial stages of mixing and placement and thermal and volumetric expansion. This section does not consider backfilling operations and repository heating as affecting seal performance, although these factors affect interface zone performance. Thermal effects on backfill and repository heating are more complex interactions and are outside the scope of this report. They will be addressed in later phases of the YMP.

The design mix includes such factors as the water-to-cement ratio, aggregate size, and temperature and moisture environments at depth. The design mix workability is attained through control of these factors. The geothermal conditions at depth control temperature of the surrounding rock. The placement temperature controls the initial temperature of the mix, and exothermic chemical reactions during cement hydration elevate temperature. The water-to-cement ratio, humidity, and containment (or addition) of moisture during the curing process control moisture content.

1. Initial mixing and placement
2. Volumetric and thermal expansion (cement hydration)
3. Thermal contraction during cooling
4. Backfill emplacement and subsequent deformational response of plug
5. Postclosure period

Figure D-1. Stages in Seal Development

Residual compressive or tensile stresses can develop within the seal that could result in separation at the interface zone. The permanent effect, shown by modeling, depends upon factors such as heat evolution due to hydration, thermal diffusion to the surrounding welded and nonwelded tuff, thermal expansion coefficient of the seal, evolution of the thermomechanical properties of the seal during the curing process, and volumetric expansion due to hydrating phases.

The heat of hydration for the selected seal material produces an increase in temperature and thermal gradients that results in thermal stress. With completion of hydration, heat diffuses into the surrounding rock mass, reducing temperatures. The rate of heat dissipation depends on the thermal diffusivity of the plug and the surrounding host rock. The seal expands thermally during curing and contracts during cooling, depending on the temperature and the thermal expansion coefficient for the plug.

The formation of expansive constituents leads to the volumetric expansion of cement and the development of interface stress (Hoff, 1974). The effectiveness of the expansive constituents depends upon temperature, moisture environments, evolution of thermomechanical properties, and degree of external restraint. Moisture and temperature environments affect the rate of exothermic reaction and therefore the rates of expansive constituent formation and hardening (Beirute and Tragesser, 1973). A reduction in water available for cement hydration reduces the rate and amount of expansive constituent formation. Expansivity may be optimized only if moisture is available during curing.

Volumetric expansion and stiffness properties change during curing. The relationship between the seal material stiffness and the volumetric expansivity can be characterized as early stiffening (i.e., the material stiffens before the main expansive phase) or late stiffening (i.e., the material stiffens after the main expansive phase). An early-stiffening material develops a higher interface stress. A late-stiffening material develops a lower interface stress, because the volumetric expansion precedes hardening or stiffening of the material.

The degree of restraint offered by the surrounding rock mass and formwork also affects the development of interface stress. Expansive constituents grow preferentially toward regions

where lower restraining forces are acting. In tests on cement by Ish-Shalom and Bentur (1975), uniaxial restraint developed a stress value of 0.4 MPa, whereas isotropic triaxial restraint developed values exceeding 9.8 MPa. Analysis of the development of interface stress during curing requires expansivity and stiffness data as functions of time and restraint in representative environmental conditions.

The supporting calculations presented in this appendix evaluate conditions favorable to developing a tight interface. The supporting calculations also provide predictions of temperature and interface stress to be used in finding the required range, sensitivity, and accuracy for instrumentation for the seal tests.

D.2 Methods of Analysis

The one-dimensional radial numerical and analytical model SHAFT.SEAL solves for temperatures and stresses as a function of radial position, either in the seal or the surrounding host rock (Figure D-2). The model uses the implicit finite-difference method (Carnahan et al., 1990) to analyze temperature rise and fall following completion of the hydration. The model assumes conductive heat transfer and analyzes thermal and volumetric stress using closed-form solutions that account for thermal or initial volumetric strains.

D.3 Input Properties

The water-to-cement ratio, humidity, and containment (or addition) of moisture during the curing process control the design-mix moisture content. The design mixes and associated hydration properties for the cementitious seals were selected based on the water-to-cement ratio, aggregate size (grout versus concrete), and temperature and moisture environments at depth. The geothermal conditions at depth and repository heating determine the temperature of the surrounding rock at selected sealing locations in welded and nonwelded tuff. The analysis varies the placement temperature with respect to the surrounding host rock temperature during the tests to evaluate initial temperature effects on interface stress development.

Figure D-2. One-Dimensional Modeling and Analysis Using SHAFT.SEAL

The analyses used input parameters determined from laboratory studies and values in the literature for welded tuff thermal and thermomechanical rock properties at specific sealing locations (DOE, 1991) and cementitious seals. Table D-1 summarizes the thermal and thermomechanical input properties. Figure D-3 summarizes the heat of hydration and Young's modulus as a function of time. The injection pressures ranged from 0% to 50% of the minimum horizontal stress at different sealing locations in welded and nonwelded tuff. The placement temperatures ranged from 4°C to the ambient geothermal temperature at depth. (These temperatures were achieved by cooling the aggregate or circulating water through the seal.) The analyses used rock mass temperatures ranging from ambient geothermal temperature (22°C) to elevated temperatures (42°C) due to potential repository heating. The analyses used these factors to evaluate seals of 0.3 m to 3.0 m in diameter.

D.4 Analysis Results

The results from SHAFT.SEAL include temperature and stress time histories in the seal and surrounding rock. Figure D-4 presents typical results for radial temperature and stress distributions from 1 day to 60 days. The thermal analysis shows that temperatures initially rise uniformly due to cement hydration, maintaining a high radial thermal gradient within the interface zone. Thermal gradients decrease with increasing time as hydration nears completion and heat thermally diffuses to the surrounding rock mass. After 7 days, the temperature rises to 35°C, inducing a sharp thermal gradient at the interface zone. After this time, the temperatures fall and, though elevated, approach the in situ rock temperature after 60 days. The temperatures in the seal and surrounding rock mass suggest that the transient heat pulse due to concrete hydration is short compared to the time required for heat to diffuse from the seal. Figure D-4 also illustrates the radial and tangential stress distributions after 7 days and 60 days for the seal and the surrounding rock. Seal stresses are due both to thermal and volumetric effects after 7 days. After 60 days, the seal radial and tangential stresses fall as hydration nears completion. Temperatures are still somewhat elevated for the massive plug, as heat has not fully dissipated into the surrounding rock mass. In the surrounding rock, the tangential stress is higher, reflecting the in situ state of the stress. Comparisons of stress distributions within the rock after 60 days to initial stress distributions (0 days) show that the rock radial stresses rise, while tangential stresses fall due to the developing interface stress. In time, as temperatures approach the in situ placement

Table D-1. Summary of Thermal and Thermomechanical Input Properties for the Seal and Host Rock

Parameter	Units	Boreholes	Shafts
Seal Size	m	0.3 and 0.61	1.52 and 3.05
Seal Ultimate Young's Modulus	MPa	6,890	6,890-27,800
Seal Poisson's Ratio	-	0.2	0.2
Seal Thermal Expansion	$^{\circ}\text{C}^{-1}$	12.6×10^{-6}	12.6×10^{-6}
Seal Thermal Conductivity	J/(m Day $^{\circ}\text{C}$)	255,200	259,200
Specific Heat	J/kg $^{\circ}\text{C}$	962.8	962.8
Ultimate Volumetric Expansion	%	0.03	0.03
Rock Young's Modulus	MPa	7,800	7,800
Rock Poisson's Ratio	-	0.16	0.16
Rock Thermal Expansion	$^{\circ}\text{C}^{-1}$	6.4×10^{-6}	6.4×10^{-6}
Rock Thermal Conductivity	J/(m Day $^{\circ}\text{C}$)	113,443	113,443
Thermal Capacitance	J/(m ³ - $^{\circ}\text{C}$)	1.97×10^6	1.97×10^6
Rock Temperature	$^{\circ}\text{C}$	22 - 42	22 - 42
In Situ Stress	MPa	2.4	2.4

Figure D-3. Evolution of Cementitious Material Properties: (a) Heat of Hydration and (b) Young's Modulus

Figure D-4. Radial Distribution in Seal and Adjacent Rock: (a) Temperature and (b) Interface Stress (size = 3.0 m, placement temperature = 4°C, and injection pressure = 0)

temperature, the radial and tangential stress distributions reflect the in situ far-field and residual radial interface stresses. Thermal stresses develop only when thermal gradients are present, and thermal gradients are absent over the long term. The parametric studies examined the effects of seal size, rock temperature, seal placement temperature, and injection pressure on temperature and stress. Figures D-5 through D-9 present temperature and stress histories as affected by seal size, rock temperature, placement temperature, and injection pressure.

The preliminary analyses showed that the peak temperatures for seals placed at ambient temperature ranged from 22°C to 38°C, with interface stresses ranging from -0.4 MPa to 1.0 MPa. For larger seals, hydration effects are more significant due to the mass of seal material; the temperatures rise to higher levels in comparison to smaller seals (Figure D-5). Because the volume-to-surface-area ratio is larger, heat diffuses more slowly into the surrounding rock mass. Because the stiffness of the seal is increasing with time, the larger seal is stiffer when temperatures fall, and the model predicts the development of residual tensile stresses at the interface zone after some period. The results suggest difficulty in achieving a tight interface in a larger seal; contact grouting of the interface zone may be necessary.

Analyses for borehole and shaft plugs showed the development of a tight interface (high interface stress) also depends upon the contrast in temperature between the seal and the rock, as illustrated in Figures D-6 and D-7. A cooler seal temperature leads to the development of residual compressive stress. Tensile radial interface stress can be controlled in the planned tests by decreasing the placement temperature in relation to the surrounding rock temperature and selecting seal locations in tuff with high stiffness (welded tuff).

It is preferable to develop radial compressive interface stress directly by selecting the cementitious seal with a low heat of hydration and emplacing under a slight injection pressure (Figures D-8 and D-9).

D.5 Conclusions

The SHAFT.SEAL analytical results suggest the importance of selecting an early stiffening expansive cement, an injection pressure tailored to in situ conditions, and a low heat of hydration

Figure D-5. Influence of Seal Size on (a) Temperature and (b) Interface Stress (rock temperature = placement temperature = 22.7°C, and injection pressure = 0)

Figure D-6. Influence of Placement Temperature on (a) Borehole Temperature and (b) Interface Stress (size = 0.3 m, rock temperature = 22.7°C, and injection pressure = 0)

Figure D-7. Influence of Placement Temperature on (a) Shaft Temperature and (b) Shaft Interface Stress (size = 3.0 m, rock temperature = 22.7°C, and injection pressure = 0)

Figure D-8. Influence of Injection Pressure on (a) Borehole Temperature and (b) Borehole Interface Stress (size = 0.3 m, and rock temperature = placement temperature = 22.7°C)

Figure D-9. Influence of Injection Pressure on (a) Shaft Temperature and (b) Shaft Interface Stress (size = 3.0 m, and rock temperature = placement temperature = 22.7°C)

to increase the potential for developing residual compressive interface stress. Results also suggest that it is favorable to place the seal with a temperature contrast between the seal and the surrounding host rock formation. The temperature contrast could be developed by reducing the emplacement temperature of the cement seal relative to the surrounding rock temperature.

Results show the required instrumentation should measure temperatures from 20°C to 50°C and stresses of ~0.1 MPa to ~1.0 MPa from initial placement to 60 days. Results also show the temperature transient lasts from 10 days for small boreholes to more than 500 days for large shaft seals and is dependent upon seal size.

Greater temperature effects are predicted in large-scale seals with a greater potential for developing residual tensile interface stress. Progressive component evaluation is indicated to interpret the results of small-scale tests and to use these results to develop strategies for performing the large-scale tests. After performing the small-scale tests, interpreting the results, and refining the thermal/structural model, the large-scale tests can serve as performance confirmation tests. These tests will demonstrate a complete understanding of structural hydration effects. The selection of correct material, construction methods, and quality assurance and quality-control procedures for validated seal design will be based upon this understanding.

APPENDIX E
WATER FLOW IN A DRIFT

APPENDIX E

WATER FLOW IN A DRIFT

Two analyses were performed to improve understanding of the behavior of water flow in a backfilled drift. Two basic questions associated with water flow in a drift are "How far does water flow in a drift before it is absorbed into the floor of the drift?" and "How effective can a backfill having a slightly reduced permeability from the general backfill be in controlling the lateral migration in a drift?" These questions form the basis for the objective of the analyses. The objectives of these analyses are:

- For the first analysis, to determine the maximum distance that a continuous supply of water entering a drift can migrate before the water supply is absorbed into the drift floor.
- For the second analysis, to determine the effectiveness of a reduced permeability backfill on the lateral migration of water flow in a drift.

The first analysis is described in Section E.2 and the second analysis is described in Section E.3. Section E.1 describes the inputs that were used in the analyses.

E.1 Input for the Analysis

Inputs discussed below are selection of a suitable gradation curve for crushed tuff, the hydrologic properties of the crushed tuff, and the boundary conditions.

E.1.1 Selection of a Suitable Gradation Curve

Development of a repository requires the extraction of the rock to develop the waste-emplacement and nonwaste-emplacement rooms. This extracted, crushed rock will be used to seal (backfill) the repository. The current baseline design plans to extract the rock using a TBM. With this extraction method in mind, an attempt was made to obtain a typical gradation curve for tuff extracted by a TBM. This information was not readily available; gradation curves

from other sources were used. This gradation curve was used to determine the hydrologic properties of the backfill.

The gradation curves are displayed in Figure E-1. Curves 1 and 2 were generated by crushing partially welded tuff from G-Tunnel on the NTS in a laboratory crusher. The rock was first reduced to slabs 7.5 cm to 15 cm thick and further reduced with a sledge hammer. The rock was then passed through a jaw crusher with a 7.5-cm opening and a jaw crusher with a 3.75-cm opening. Curves 3 and 4 were generated by simulating the crushing of welded tuff in a laboratory using disks and picks similar to those on a TBM. At the Colorado School of Mines (CSM), variations were made in the penetration and the spacing of the disks and cutters and results evaluated. The most representative penetration depth and spacing were assumed to be 0.75 cm and 10 cm, respectively. Curves 5 and 6 were developed from crushed rock samples obtained from a TBM operation in nonwelded tuff. The samples were obtained from the Wahmonie Formation at Little Skull Mountain at the NTS.

E.1.2 Development of Hydrologic Properties for Backfill

The curves in Figure E-1 were used as guidance to select gradation curves for backfill. Curves 3, 4, 5, and 6 represent gradation variations between welded and nonwelded tuff. Other factors to consider when defining the gradation include the following:

- Handling crushed rock will modify the gradation. Figure E-2 illustrates the material flow of crushed rock from a mining and backfilling operation.
- Extracted material can be modified, if necessary, through a large-scale sieving operation.

Hydrologic properties of selected backfill used in this modeling process were developed by:

- Classifying the grain-size distribution, displayed on Curves 1 through 6 of Figure E-1, according to the Unified Soil Classification (Lambe and Whitman, 1969).
- Knowing the soil classification to select soil types from both Mualem (1976) and Case et al. (1983).

Figure E-1. Gradation of Crushed and Mined Tuff - A Comparison

Figure E-2. Material Flow Diagram for Material Preparation and Handling of Backfill

- Processing selected data from these catalogs using the SOIL program (El-Kadi, 1987).

In classifying the gradation, the D_{60} , D_{30} , and D_{10} values were computed to determine the values of C_u (uniformity coefficient) and C_c (compression index). This analysis resulted in all gradations falling into the "GW" category, which is described as a well-graded gravel and gravel/sand mixture with few or no fines.

The selection criteria used to select an appropriate soil/backfill were:

- The soil in the catalogs should match or be similar to the soil classification, and
- The data in the catalogs should be sufficient to develop unsaturated-saturated hydrologic properties.

Two different material with different saturated, hydraulic conductivity (K_s) values were intended to be selected. The K_s values assumed were 10^{-1} cm/s. Using these general selection criteria, a gravelly sand (#4135) was selected from Mualem (1976) and glacial outwash material (data file #p8al.cv3), described as a coarse sand and gravel, was chosen from Case et al. (1983).

Data from the catalogs were input to the SOIL program (El-Kadi, 1987) to develop the Van Genuchten curve fit parameters. The Van Genuchten relationship used was:

$$S = (S_s - S_r) \left[\frac{1}{1 + |\alpha h|^\beta} \right]^\lambda + S_r$$

where

- h = Suction head,
- S = Saturation as a function of pressure,
- S_s = Maximum saturation (≈ 1),
- S_r = Residual saturation,

α and β = Curve-fit parameters, and
 λ = $1 - 1/\beta$.

When this relationship is used, the following analytical expression of hydraulic conductivity as a function of pressure head results when the method of Mualem (1976) is used.

$$K(h) = K_s \frac{[1 - |\alpha h|^{\beta-1} \cdot (1 + |\alpha h|^\beta)^{-\lambda}]^2}{(1 + |\alpha h|^\beta)^{\lambda/2}}$$

where

K = Hydraulic conductivity as a function of pressure head,
 K_s = Saturated hydraulic conductivity,
 h = Pressure head, and other terms are as defined previously.

The resulting K_s , α , β , and S_r are included in Table E-1, as are other properties required for the hydrologic flow analyses, such as properties of the Topopah Spring Member densely welded and fractured tuff (TSw2).

E.1.3 Boundary Conditions

Boundary conditions for the modeled region are specified on Figure E-3. The left and right sides were treated as "no flux" (zero pressure-head gradient) boundaries. A water-table boundary (zero pressure head) was specified at the bottom of the modeled region. A Darcy flux of 3.17×10^{-12} m/s (0.1 mm/y) was imposed as recharge along the top boundary.

Introduction of additional water into the modeled area is determined by the objective of the analysis. For the first analysis, additional water was assumed to flow into the right-side modeled area at a rate of 1 gpm over a drift width of ~ 3 m.

Table E-1. Hydrologic Properties of Selected Backfills

Simulated Backfill or Rock	Saturated Hydraulic Conductivity (cm/s)	Porosity (α, m^{-1})	Van Genuchten		
			α, m^{-1}	β	S_r
Glacial Outwash	7.83×10^{-2}	0.378	5.973	4.037	0.05378
Gravelly Sand	2.78×10^{-4}	0.326	1.491	2.833	0.2417
TSw2, Matrix ^a	1.9×10^{-9}	0.11	0.00567	1.798	0.08
TSw2, Fracture	^b	18×10^{-5}	1.2851	4.23	0.395
^a All rock properties are from Klavetter and Peters (1986).					
^b Depends upon the assumed bulk rock hydraulic conductivity.					

The reasons for selecting this value follow:

- In Fernandez et al. (1987), an unanticipated amount of water inflow entering into the underground facility was estimated at 5600 m³/year or 2.81 gpm. Assuming four discrete sources for this amount, a value of 0.7 gpm per source is computed.
- A discrete water inflow at G-tunnel was estimated at 0.01 gpm (Fernandez and Freshley, 1984).

A value of 1 gpm for a 3-m-wide drift was selected by considering these assumptions and estimated values for inflow.

For the second analysis, the drift was filled with two backfills with different hydrologic properties (Figure E-3). To maximize the internal extent of the phreatic surface in the gravelly sand, it was assumed that the glacial outwash was fully saturated.

Figure E-3. Schematic of Problem Analyzed (For the first analysis, the only material in the drift is modeled as glacial outwash.)

E.2 Flow in a Drift Assuming Constant Inflow

The objective of this analysis is to determine the maximum distance that a continuous supply of water entering a drift can migrate before the water supply is absorbed into the drift floor.

E.2.1 Approach

The finite-element code NORIA-SP (Hopkins et al., 1991) was used to model water flow in a drift. The drift was inclined at 6% with a continuous inflow of 0.35 gpm per unit width of drift. Both fracture and matrix properties were considered to develop an area-weighted composite model. The composite model was used in the NORIA calculations to account for fracture flow (Figure E-4). Figure E-5 gives the conductivity for the glacial outwash. Conductivity as a function of pore pressure was obtained using the van Genuchten curve fit; the α and β values are given in Table E-1. These figures show that conductivity is highly nonlinear with pore pressure. Thus accurate results can be computed only if an extremely refined numerical mesh is used. To obtain a check on the effect of the mesh size, two geometric widths, 240 and 480 m, were used (see Figure E-6). Two drift water inlet geometries were used with the 240-m mesh.

The numerical mesh for both lengths contained 525 eight-node biquadratic elements. The resolution of these meshes is a compromise between desired resolution and the computer expense associated with computing solutions using large arrays of elements. The initial results were calculated using a total drift length of 240 m. This size was chosen because preliminary calculations, based on unit pressure gradient assumptions, predict that the 0.35 gpm of water being injected into the drift should be absorbed ~200 m from the injection point.

E.2.2 Results

Two solutions were obtained with the 240-m geometry. In the first, the 0.35-gpm source was injected into the drift through a slot 0.5 m (one element) high. In the second calculation,

Figure E-4. Hydraulic Conductivity for the TSw2 Material

Figure E-5. Hydraulic Conductivity for the Glacial Outwash Material

Figure E-6. Dimensions and Geometry of Modeled Area

the injection slot height was specified to be equal to the drift height (3.68 m or five elements). Excluding the region near the injection point, the trends given by both calculations agree. The flow into the TSw2 decreases at $x = 20$ m (220 m from the injection point). This calculation was repeated using the 240-m mesh stretched to a width of 480 m. Results show the flow of water into the tuff material below the drift decreases for $x < 260$ m. As with the first mesh, this is ~ 220 m from the injection point. Thus the flow trends computed using both meshes are consistent. To determine the effect of the relatively low flow resistance offered by saturated fractures in the TSw2 material, results were calculated using saturated fracture conductivities decreased by a factor of 10^3 . As a result, the water injected into the drift flooded the entire drift volume. Drainage rate from the drift was constant along the entire length of the numerical mesh.

E.3 Effect of Material Contrast on Drift Flow

The objective of this analysis is to determine the effect of selected drift backfill in controlling the lateral migration of water flow in a drift. The modeled regions are shown on Figure E-7. Two cases were evaluated:

- Case 1--The glacial outwash material to the right of embankment is fully saturated. The saturated hydraulic conductivity of the TSw2 unit is 10^{-5} m/s.
- Case 2--This case is the same as Case 1 except the saturated hydraulic conductivity of the TSw2 unit is 10^{-7} m/s. For this case, two runs were made as described below.

The dashed lines of Figure E-7 indicate the extended region used for Case 2, Run 2. The region modeled for Case 1 and Case 2, Run 1 was 87.78 m wide in the horizontal (x) direction and 140.1 m high in the vertical (y) direction. A 3.68-m-high horizontal drift was 124.2 m above the water table. A 2:1 embankment was halfway between the vertical solid-line boundaries of the modeled region. Drift material to the left of the embankment was a gravelly sand and drift material to the right was a glacial outwash. The remainder of the modeled region

Figure E-7. Modeled Regions With Boundary Conditions

consisted of the TSw2 unit. The Case 2, Run 2 region contained an additional 28.36 m of material added to the left of the Case 2, Run 1 modeled region.

E.3.1 Approach

The computer code DCM3D (Updegraff et al., 1991) was used to model the effect of selected drift backfill in controlling water flow in drifts. DCM3D models groundwater flow in a variably saturated, fractured, porous media.

E.3.2 Initial Conditions

Because the purpose of this modeling effort was to generate a steady-state solution, the choice of initial conditions was unimportant. Modeling steady-state problems in unsaturated flow regimes is usually accomplished by setting saturated initial conditions in the region and allowing the region to drain to a steady-state solution. Solutions are thus generated because the region is entirely saturated during the initial stages of the simulation; the problem is linear and, therefore, easier to solve. The solution does not generate any steep infiltration fronts, which can add to the difficulty of solving the problem. The initial conditions specified for this problem were saturated pressure heads that produced an initial zero-velocity distribution. DCM3D runs better if the initial velocity distribution is consistent throughout the modeled region. Therefore, the initial pressure-head distribution was set as though the initial pressure head were hydrostatic everywhere except in the glacial outwash. The initial pressure-head distribution was described by:

$$p_{ir}(y_w) = h_r - y_w$$

where

- $p_{ir}(y_w)$ = Initial pressure head in the modeled region,
- h_r = Height of the modeled region (140.1 m), and
- y_w = Elevation above the water table.

This pressure-head distribution results in a constant total head throughout the modeled region, producing a zero-flux initial condition everywhere.

The initial pressure-head distribution in the glacial outwash is described in terms of elevation above the drift bottom. This initial pressure head is

$$p_{ig}(y_d) = h_d - y_d$$

where

- $p_{ig}(y_d)$ = Initial pressure head in the glacial outwash,
- h_d = Height of the drift (3.68 m), and
- y_d = Elevation above the drift bottom.

The material properties are given in Table E-1. The main difference between the Case 1 and Case 2 runs is the value for the saturated hydraulic conductivity for the TSw2 unit as described previously. The Case 2 runs used a value one-hundredth of the value used for the Case 1 run.

E.3.3 Gridding

The grid consisted of variably sized, rectangular grid blocks. For Case 1 and Case 2, Run 1, there were 48 grid blocks in the horizontal direction and 148 grid blocks in the vertical direction. For Case 2, Run 2, there were 52 grid blocks in the horizontal direction. The smallest grid blocks, in the embankment area, were 0.46 m by 0.46 m. Moving away from each side of the embankment in the horizontal direction, the grid blocks increased in size by a factor of 1.2 (i.e., 20% larger than the size of the preceding grid block). In the vertical direction, the grid blocks were 0.46 m in drift area. Above and below the drift, the sizes ranged from a minimum of 0.46 m (immediately above and below the drift) to a maximum of 1.0 m (far from the drift).

E.3.4 Case 1 Results

The Case 1 pressure-head distribution for the entire modeled region is shown in Figure E-8. Pressure heads greater than zero represent saturated conditions and pressure heads less than zero represent unsaturated conditions. The zero pressure-head line indicates the phreatic surface. Figure E-9 shows the development of the phreatic surface. The saturated zone includes the glacial outwash unit and extends 19 m into the embankment and 93 m below the drift bottom. At the top of the drift, the zero pressure line extends left from the right boundary, representing the top of the saturated zone in the glacial outwash (as prescribed in the initial conditions mentioned above) and the influx of water from the glacial outwash into the gravelly sands of the embankment. At the bottom of the drift, the zero-pressure line makes a sharp break to the right (not shown in Figure E-9), caused by the reduced vertical flux in the embankment due to a lower saturated hydraulic conductivity of the embankment material. At the embankment toe, the zero pressure-head line drops sharply downward. This is caused by the relatively high Darcy fluxes (Darcy velocities) where the glacial outwash material in the drift contacts the fractures in the TSw2 unit.

An interesting phenomenon occurs at the drift bottom along the left boundary. At this location, a depression in the pressure-head field develops. Some one-dimensional, vertical simulations indicate that this is a result of the material-property contrast between the gravelly sand and the TSw2 unit. In the one-dimensional simulations with a flux of 1.0 mm/yr at the top, an extremely steep pressure-head gradient of about 158 m/m forms just below the drift bottom. The two-dimensional simulation with constant pressure head in the glacial outwash part of the drift does not inject enough water into the embankment to remove this depression. The depression appears to be real and is a response of the recharge at the top of the grid. In this area of the grid, the pressure-head response is due more to the recharge than to the glacial outwash.

Downward Darcy fluxes along the drift bottom are presented in Figure E-10. Distances to the right of the embankment are represented by positive distances in the figure. The total fluxes range from 3.37×10^{-5} m/s just to the right of the embankment toe to 1.09×10^{-5} m/s

Figure E-8. Case 1--Pressure Head Distribution in Modeled Region

Figure E-9. Effect of Material Contact on Extent of Phreatic Surface: (a) Case 1 and (b) Case 2

Figure E-10. Case 1--Darcy Fluxes From Base of Drift

at the right boundary. To the left of the embankment toe, the downward Darcy fluxes decrease until the recharge rate is reached at the left end. Throughout the drift bottom, almost all the flux occurs in the fractures.

The average downward flux to the right of the embankment (glacial outwash) is 1.20×10^{-5} m/s. This value is 20% higher than the saturated hydraulic conductivity of the fractures in the TSw2 unit.

E.3.5 Case 2, Run 1 Results

The Case 2, Run 1 pressure-head distribution for the entire modeled region is shown in Figure E-11. A blowup of the phreatic surface developed in the drift area is shown in Figure E-9b. The saturated zone includes the glacial outwash unit and extends 23 m to the left of the embankment toe and 109 m below the drift bottom. At the top of the drift, the zero-pressure line extends leftward from the right boundary, representing the top of the saturated zone in the glacial outwash (as prescribed in the initial conditions mentioned above) and the influx of water from the glacial outwash into the embankment. At the bottom of the drift, the zero-pressure line makes a more gradual break to the right than the more abrupt break to the right in Case 1 (see Figure E-8), because the vertical Darcy flux within the embankment is similar to those to the right of the embankment (see Figure E-12). In general, lines of constant pressure head are pushed more to the left for the case with a low fracture hydraulic conductivity (Case 2) than for the case with a high one (Case 1).

The depression in the pressure-head field still forms along the left boundary at the drift bottom, as in Figure E-8. However, it is not nearly as pronounced as in Figure E-8 because the constant pressure-head lines are pushed more to the left for this case.

Downward Darcy fluxes from the drift bottom are presented in Figure E-12. The Darcy fluxes range from 1.18×10^{-7} m/s just to the right of the embankment toe to 1.09×10^{-7} m/s at the right boundary. To the left of the embankment, the downward Darcy fluxes decrease until about twice the recharge rate is reached at the left end, indicating that more grid blocks are

Figure E-11. Case 2, Run 1--Pressure Head in Modeled Region

Figure E-12. Case 2, Run 1--Darcy Fluxes From Base of Drift

needed on the left side to model the problem properly. Throughout most of the drift bottom, all Darcy flux occurs in the fractures. The exception occurs near the left boundary, where Darcy fluxes in the matrix and the fracture are almost evenly divided.

The average downward Darcy flux to the right of the embankment (glacial outwash) is 1.08×10^{-7} m/s. This value is 8% higher than the saturated hydraulic conductivity of the fractures in the TSw2 unit.

E.3.6 Case 2, Run 2 Results

The pressure-head distribution in the modeled region and drift area for this case are shown in Figures E-13 and E-14, respectively. For pressure heads greater than -1.5 m, the results are similar for Run 1 and Run 2. For pressure heads less than or equal to -2.0 m, the pressure heads are pushed more to the left for Run 2. This is probably a result of the left boundary being moved more to the left for Run 2. The depression in the pressure-head field that forms along the drift bottom on the left boundary is more pronounced than in Run 1, similar to that formed for Case 1. This is an indication that the additional grid blocks used for Run 2 provide better results than Run 1 near the left boundary. Vertical Darcy fluxes are presented in Figure E-15. The velocities are the same for Run 1 and Run 2 at distances less than 23 m. To the left of this point, vertical Darcy fluxes decrease until they equal the recharge rate near the left boundary. The average vertical Darcy fluxes are the same as those for Run 1.

E.3.7 Summary

Case 1--The phreatic surface extends ~ 19 m into the embankment as measured from the embankment toe and ~ 93 m below the drift bottom. The combined fracture and porous media Darcy flux from the bottom of the drift within the glacial outwash averages 1.20×10^{-5} m/s with a range from 3.37×10^{-5} m/s at the embankment toe to 1.09×10^{-5} m/s at the right boundary.

Case 2--The phreatic surface extends ~ 23 m into the embankment as measured from the embankment toe and ~ 109 m below the drift bottom. The combined fracture and porous media

Figure E-13. Case 2, Run 2--Pressure Head in Modeled Region

Figure E-14. Case 2, Run 2--Pressure Head in Drift Region

Figure E-15. Case 2, Run 2--Darcy Fluxes From Base of Drift

Darcy flux from the bottom of the drift within the glacial outwash averages 1.08×10^{-7} m/s with a range from 1.18×10^{-7} m/s at the embankment toe to 1.06×10^{-7} m/s at the right boundary. This case was also run with an extended grid on the left side. The results for the extended grid run produced the same numbers for the extent of the phreatic surface and Darcy fluxes.

APPENDIX F

STRUCTURAL DESIGN REQUIREMENTS FOR SHAFT PLUGS

APPENDIX F

STRUCTURAL DESIGN REQUIREMENTS FOR SHAFT PLUGS

This appendix presents the results of performing preliminary structural analysis on potential repository shaft and ramp plugs to aid in the selection of preliminary design requirements. Section F.1 presents the introduction and background and identifies the scope of the present calculations. Section F.2 presents the technical approach and documents solution methods used for preliminary analysis. Section F.3 presents the results of analysis for the variation of several important parameters. Section F.4 presents the conclusions of the preliminary shaft and ramp plug analysis.

F.1 Introduction

This appendix presents the results of stress analysis at several points in a concrete plug and the surrounding rock (Figure F-1). The analysis evaluates four plug locations: a deep anchor-to-bedrock plug located in the Tiva Canyon formation near the ground surface in a shaft, a deep shaft plug located near the potential repository horizon, a station plug located in a horizontal drift next to a vertical shaft or ramp, and a bulkhead located in the underground facility. The analysis assumes the plugs to be cylindrical in shape and composed of a rigid material. The plug is keyed ~ 1 m into the wall of the shaft or tunnel by removing the liner and surrounding rock in the area of the plug (see Figure F-1a). This shape allows the most direct treatment for restoration of the MPZ. It is also advantageous in that backfill loads will be transferred as bearing compression to the surrounding rock.

These calculations evaluate two different loadings: (1) dead loads due to the weight of an overlying saturated backfill and the plug itself or lateral earth pressures from the backfill and water in the case of a station plug and (2) thermal loads due to the rise in potential repository temperature. In evaluating dead loads, the analysis uses simple statics with certain assumptions regarding load resistance. In evaluating thermal loads, the simple linear analysis assumes that the plug and temperatures at the seal location rise uniformly.

Figure F-1. Plug Geometries: (a) Anchor-to-Bedrock or Deep Shaft and (b) Station or Drift Plug

The analysis initially evaluates dead loads for approximate conditions at the time of potential repository closure or in areas isolated from the high-temperature environment. Subsequently, the dead-plus-thermal load approximates the long-term conditions in the potential repository. The analysis evaluates stresses at the interface in both the plug and rock, accounting for normal and shear stress discontinuities across this boundary. In calculating stresses within the rock, the analysis considers the preexisting state of stress in the rock before plug placement. The analyses assumed an anisotropic stress field to maximize the range of the boundary or tangential stress.

The analyses evaluate various length plugs; these include plug L/D of 1, 2, and 4. The analyses assess the influence of constructing larger plugs on structural performance.

F.2 Technical Approach

This section presents the formulas and detailed discussion of the assumptions used in performing the analysis. Section F.2.1 presents the Kirsch solution used to evaluate rock stresses before plug emplacement. Section F.2.2 presents several simple statics formulas for evaluating dead loads. Section F.2.3 presents a closed-form solution for evaluating linearly elastic thermal stresses. Section F.2.4 presents the formulas for calculating principal and maximum shear stresses. In the following discussion, primed stresses refer to stresses in the rock.

F.2.1 State of Stress in the Rock Before Plug Emplacement

The Kirsch solution assumes the rock is homogeneous, isotropic, and continuous. The analysis further assumes the rock exhibits linearly elastic behavior. The radial and tangential components are given by (Goodman, 1980):

$$\sigma'_{\theta p} = \left[\frac{P_1 + P_2}{2} * \left(1 + \frac{a^2}{r^2} \right) \right] - \left[\frac{P_1 - P_2}{2} * \left(1 + \frac{3a^4}{r^4} \right) \cos(2\theta) \right]$$

$$\sigma'_{rp} = \left[\frac{P_1 + P_2}{2} * \left(1 - \frac{a^2}{r^2} \right) \right] + \left[\frac{P_1 - P_2}{2} * \left(1 - \frac{4a^2}{r^2} + \frac{3a^4}{r^4} \right) \cos(2\theta) \right]$$

where

$\sigma'_{\theta p}$ = Tangential stress in the rock before plug emplacement,

σ'_{rp} = Radial stress in the rock before plug emplacement,

r = Radius to the point where stress is calculated,

θ = Angle from the x axis to the point that stress is calculated,

P_1, P_2 = Far-field stresses, and

a = Radius of the plug.

In this analysis, the weight of the overlying rock gives the far-field vertical stress. The maximum horizontal stress equals the vertical stress. The minimum horizontal stress, P_2 , is taken as 30% of this stress. Because the minimum and maximum stresses are not equal, the maximum and minimum tangential stresses are calculated for $\theta = 0^\circ$ and $\theta = 90^\circ$. The vertical stress due to the weight of the overlying rock is

$$\sigma'_{zp} = g \sum \rho_i h_i$$

where

σ'_{zp} = Vertical stress in the rock before plug placement,

g = Gravitational constant,

ρ_i = Mass density of the i^{th} layer, and

h_i = Thickness of the i^{th} layer.

F.2.2 Stresses Resulting From Dead Loads

The stresses within the plug transmitted to the rock are calculated from the weight of the plug or backfill for vertical plugs. For horizontal plugs, the analysis considers the lateral earth pressure for horizontal plugs. The total load for a vertical plug is

$$P_d = \gamma_b \pi r_s^2 h_f + \pi a^2 L \gamma_c$$

where

- P_d = Total dead load,
- r_s = Radius of the shaft,
- h_f = Height of backfill in the shaft,¹
- γ_b = Saturated unit weight of backfill,
- γ_c = Unit weight of concrete,
- a = Plug radius, and
- L = Plug length.

For the anchor-to-bedrock plug (depth of ~10 m), the dead load on the plug is much lower than that on the deep shaft plug (depth of 310 m).

For the station plug or drift plug, the active lateral earth pressure generated by the column of backfill next to the plug is

$$P_d = \sigma_n * A_t = [(\gamma_b h_f - \gamma_w h_f) * \tan^2(45^\circ - \frac{\phi}{2}) + u] * A_t$$

¹Note that this is a conservative assumption because peripheral "skin" friction reduces this loading.

where

- ϕ = Internal angle of friction of the backfill,
- h_f = Height of backfill next to the plug,
- γ_w = Unit weight of water,
- u = Pore pressure ($\gamma_w * h_f$),
- σ_n = Lateral earth pressure,
- A_t = Cross-sectional area of the tunnel, and other terms are as defined previously.

For the station plug, the column of backfill would have an approximate height of 310 m. For the drift plug, the column of backfill was assumed to be only as high as the plug diameter (9.6 m). In both cases, the analysis assumes fully saturated backfill.

The plug resists the dead load in shear along the perimeter of the plug and in bearing compression at the point where the plug keys into the rock. To provide a preliminary analysis, assume the plug resists the load in uniform shear at points A and B or in bearing compression at point C. The shear stress (τ_d) at points A and B is calculated by

$$\tau_d = \frac{P_d}{2\pi r_s L}$$

The bearing stress (σ_{zd}) at the bottom of the plug (Point C) is calculated by

$$\sigma_{zd} = \frac{P_d}{\pi(a^2 - r_s^2)}$$

In the above formula, the analysis takes no account of the compressive stresses that would develop because of elastic interaction due to Poisson's effects between the plug and the surrounding rock. These lateral compressive stresses increase the concrete and rock mass strength. Thus the calculations are conservative in not accounting for the effects of confinement. The analysis also assumes the backfill to settle below the plug and not to provide support.

F.2.3 Stresses Resulting From Thermal Loads

The thermal stress analysis uses elastic solutions under the assumption of a uniform temperature rise between the plug and the rock at the plug location. The analysis assumes that the plug interface stress at point B equals the rock interface stress at the plug boundary and the plug radial displacement equals the rock radial displacement at this boundary. The solution presented below is for the condition of plane strain that might develop at midheight of a long plug. The plug stresses that develop at point B for a uniform temperature rise may be derived from Case et al. (1992):

$$\sigma_{rt} = \frac{\alpha_p E_p E_r (1 + \nu_p) \Delta T}{E_p (1 + \nu_r) + E_r (1 - \nu_p) (1 - 2\nu_p)}$$

$$\sigma_{zt} = E_p \alpha_p \Delta T + 2\nu_p \sigma_{rt}$$

$$\sigma_{\theta t} = \sigma_{rt}$$

where

- σ_{rt} = Radial thermal stress in the plug at the plug/rock interface at midheight,
- $\sigma_{\theta t}$ = Tangential thermal stress in the plug at the plug/rock interface at midheight,
- σ_{zt} = Vertical thermal stress in the plug at the plug/rock interface at midheight,
- E_p = Young's modulus of the plug,
- ν_p = Poisson's ratio of the plug,
- E_r = Young's modulus of the rock,
- ν_r = Poisson's ratio of the rock,
- ΔT = Temperature increase in the plug, and
- α_p = Coefficient of thermal expansion of the plug.

The analysis assumes the rock to be confined in the axial direction and under a condition of plane strain. The rock stresses that develop at point B due to a uniform temperature rise may be derived:

$$\sigma_{rt} = \sigma'_{rt}$$

$$\sigma'_{\theta t} = \frac{\alpha_r E_r \Delta T}{1 - \nu_r} - \sigma'_{rt}$$

$$\sigma'_{zt} = \nu_r(\sigma'_{rt} + \sigma'_{\theta t}) + \alpha_r E_r \Delta T$$

where

α_r = Coefficient of thermal expansion of the rock,

σ'_{rt} = Radial thermal stress in the rock at the rock/plug interface,

$\sigma'_{\theta t}$ = Tangential thermal stress in the rock at rock/plug interface, and

σ'_{zt} = Vertical thermal stress in the rock at rock/plug interface.

Near the ends of the plug, the plug core is not confined in the axial direction. Therefore it can move in the axial direction due to thermal expansion and the shear stress will develop at the rock/concrete interface. Experience shows that shear stresses are maximum at the ends of the plugs and decrease gradually toward the middle of the plug. As a result and in order to simplify the problem, it was assumed that the shear stress has a triangular distribution over the end quarter-length of the plug. This assumption is crude and should be closely examined or reevaluated when more detailed analysis is conducted. As a result of this assumption, the lengthening of the plug would decrease the maximum shear stress because the shear stresses are resisted over a greater surface area. A more detailed evaluation of the effects of plug length using the finite element method would give a better approximation for the shear stress distribution.

$$\tau_{rz} = \tau'_{rz} = \frac{4a}{L} \sigma_{zt}$$

where

τ_{rz} = Shear stress in plug along plug/rock interface, and

τ'_{rz} = Shear stress in rock along plug/rock interface.

F.2.4 Principal Stresses

To provide bounding calculations for assessing the required plug strength, the second-order tensor transformation determines the principal and maximum shear stresses in the rz plane.² The principal stresses in the rz plane may be calculated from Scott (1963):

$$\sigma_{p1} = \frac{\sigma_{rr} + \sigma_{zz}}{2} + \sqrt{\left(\frac{\sigma_{zz} - \sigma_{rr}}{2}\right)^2 + \tau_{rz}^2}$$

$$\sigma_{p2} = \frac{\sigma_{rr} + \sigma_{zz}}{2} - \sqrt{\left(\frac{\sigma_{zz} - \sigma_{rr}}{2}\right)^2 + \tau_{rz}^2}$$

where

σ_{p1} , σ_{p2} = Major and minor principal stresses in the rz plane,

σ_{rr} = Combined radial stress,

σ_{zz} = Combined vertical stress, and

τ_{rz} = Combined shear stress.

²The tangential stress is a principal stress because the shear stress in the r θ plane is equal to zero for the axisymmetric case.

The maximum shear stress relates to the difference between the major and minor principal stress.

$$\tau_{\max} = \frac{\sigma_{p1} - \sigma_{p2}}{2}$$

In the above calculation, the analysis assumes that the hoop or tangential stress is intermediate to the principal stresses in the rz plane. If hoop stress is either the major or minor principal stress for the axisymmetric case, then the maximum shear stress calculation uses the tangential stress.

The axisymmetric states of stress (σ_{rr} , σ_{zz} , τ_{rz} , and $\sigma_{\theta\theta}$) were determined for the superposition of dead and dead-plus-thermal loading for points A, B, and C in both the plug and rock.

F.3 Parametric Analysis

The analysis uses a spreadsheet calculation for varying parameters. Table F-1 presents the input material properties and geometry. Table F-2 presents a summary of the input boundary conditions.

The parameters investigated include the plug lengths of one plug diameter, two plug diameters, and four plug diameters. The investigation also included the stress field for various plug depths, the orientation of the points (angle $\theta = 0^\circ$ or 90°) with respect to the anisotropic stress field, and the uniform temperature rise (0°C or 94.7°C , maximum expected temperature rise above ambient at the potential repository depth horizon).

Table F-1. Summary of Input Material Properties and Geometry

Material/Plug	Property	Value	Source
Anchor-to-Bedrock Plug	Plug diameter	8.71 m	(1)
	Bulk density of plug material	2,400 kg/m ³	(1)
	Young's modulus	34.6 GPa	(1)
	Poisson's ratio	0.26	(1)
	Coefficient of thermal expansion	1.0x10 ⁻⁵ /°C	(1)
	Depth to plug	10 m	(1)
	L/D	1, 2, and 4	(1)
Deep Shaft Plug	Plug diameter	8.71 m	(1)
	Bulk density of plug material	2,400 kg/m ³	(1)
	Young's modulus	34.6 GPa	(1)
	Poisson's ratio	0.26	(1)
	Coefficient of thermal expansion	1.0x10 ⁻⁵ /°C	(1)
	Depth to plug	310 m	(1)
	L/D	1, 2, and 4	(1)
Station Plug and Drift Plug	Plug diameter	9.62 m	(1)
	Bulk density of plug material	2,400 kg/m ³	(1)
	Young's modulus	34.6 GPa	(1)
	Poisson's ratio	0.26	(1)
	Coefficient of thermal expansion	1.0x10 ⁻⁵ /°C	(1)
	Depth to plug	310 m	(1)
	L/D	1, 2, and 4	(1)
Backfill Material	Bulk density	1,920 kg/m ³	(1)
	Internal angle of friction	35°	(1)
Rock Mass/Overburden (Anchor-to-Bedrock Plug)	Bulk density	1,930 kg/m ³	(3)
	Young's modulus	20.0 GPa	(4)
	Poisson's ratio	0.10	(4)
	Coefficient of thermal expansion	8.8x10 ⁻⁶ /°C	(4)
Rock Mass/Overburden (Deep Shaft, Station, and Drift Plug)	Bulk density	2,113 kg/m ³	(3)
	Young's modulus	15.2 GPa	(1)
	Poisson's ratio	0.22	(1)
	Coefficient of thermal expansion	8.8x10 ⁻⁶ /°C	(1)
(1) Assumed values for parametric studies. (2) After Terzaghi and Peck, 1967, p. 107. (3) Calculated from information in YMP Reference Information Base. (4) From the YMP Reference Information Base, Section 1.3.1.			

Table F-2. Summary of Input Boundary Conditions

Material/Plug	Condition	Value	Source
Anchor-to-Bedrock Plug	In situ far-field stresses		
	Horizontal-P1	0.189 MPa	(1)
	Horizontal-P2	0.057 MPa	(1)
	Ratio of P2/P1	0.3	(2)
	Maximum temperature increase	0°C	(2)
Deep Shaft Plug	In situ far-field stresses		
	Horizontal-P1	7.000 MPa	(3)
	Horizontal-P2	2.100 MPa	(1)
	Ratio of P2/P1	0.3	(2)
	Maximum temperature increase	94.7°C	(2)
Station Plug or Drift Plug	In situ far-field stresses		
	Vertical-P _{vertical}	7.000 MPa	(3)
	Horizontal-P _{horizontal}	2.100 MPa	(1)
	Ratio of P _{vertical} /P _{horizontal}	0.3	(2)
	Maximum temperature increase	94.7°C	(2)
(1) Calculated from information in YMP Reference Information Base. (2) Assumed values for parametric studies. (3) From the YMP Reference Information Base, Section 1.3.1.			

F.3.1 Anchor-To-Bedrock Plug

The anchor-to-bedrock plug is at a shallow depth of 10 m outside the high-temperature environment. The analysis initially evaluates dead loads. Figure F-2 presents the Mohr circles at points A and C in the plug ($\theta = 0^\circ$ orientation). Stress levels are low (-0.1 MPa to 1.8 MPa) due to the shallow depth of backfill supported by the plug. The analyses show that lengthening the plug will reduce the potential for diagonal tension to occur at the base of the plug due to the weight of the plug itself.

F.3.2 Deep Shaft Plug

The deep shaft plug is at the approximate depth of the potential repository horizon (310 m). Figures F-3 and F-4 present the Mohr circles for stresses in the plug for dead loads and dead-plus-thermal loads respectively.

The results for dead loads show that stress levels within the plug range from -1.1 MPa to ~ 7.2 MPa, reflecting the greater support required for a greater depth of backfill. The calculations are conservative in providing bounds for tension and compression. For a deep shaft plug, the arching that occurs around the plug would be significant in reducing the occurrence of tensile stresses.³ The analysis shows that an increase in bearing compression due to plug lengthening is proportionately less significant for the deep shaft plug than for a shallow anchor-to-bedrock plug.

The analysis calculates stresses in the rock for a deep shaft plug for two orientations, $\theta = 0^\circ$ and $\theta = 90^\circ$. The orientation of $\theta = 90^\circ$ maximizes compressive boundary stress prior to plug placement. The results show that stress levels within the rock range from -0.7 MPa to 7.2 MPa at $\theta = 0^\circ$ orientation and -0.2 MPa to 18.9 MPa at the $\theta = 90^\circ$ orientation. This

³An estimate of the amount of lateral confinement for a plug is obtained for one-dimensional loading over a continuous plane surface for the assumption of zero lateral strain. According to Goodman (1980, p. 101), the ratio of horizontal to vertical stress is given by $\nu_p/(1-\nu_p)$. Assuming a depth of 1020 feet and a saturated backfill density of 120 pcf, the calculated vertical stress is 850 psi (5.9 MPa), and with a Poisson's ratio of 0.26, the calculated lateral stress across the interface zone is 2.1 MPa. This calculation shows that a state of compression exists at the interface, particularly in the instance of a long plug.

(a)

(b)

Figure F-2. Mohr Circles Showing the Effect of Plug Length on Dead Load Stress in Anchor-to-Bedrock Plug: (a) Point A and (b) Point C

(a)

(b)

Figure F-3. Mohr Circles Showing the Effect of Plug Length on Dead Load Stress in Deep Shaft Plug: (a) Point A and (b) Point C

(a)

(b)

Figure F-4. Mohr Circles Showing the Effect of Plug Length on Dead Load Stress in Deep Shaft Plug at Elevated Temperature: (a) Point B and (b) Point C

analysis shows that the rock will respond differently in the tension and compression zones due to plug emplacement. If the effects of lateral confinement are considered, then plug placement might increase the tensile stress (reduction of boundary stress in compression) in the tensile zone ($\theta = 0^\circ$), while reducing the compressive boundary stress in the compressive zone ($\theta = 90^\circ$).

The Mohr circles for combined dead and thermal loading in the plug are presented in Figure F-4. The linear elastic analyses show that for a uniform temperature rise of 94.7°C , significantly large stress levels, ranging from approximately -80 MPa to 80 MPa, would develop at the interface near the plug ends. Compressive vertical stresses ranging from 12.2 to 39.2 MPa would develop at the plug midheight. As discussed previously, the analysis assumes that the shear stresses due to the longitudinal expansion of the plug have a triangular distribution over the end quarter length of the plug. Therefore the results show that lengthening the plug would reduce this maximum shear stress because the shear stresses are resisted over a greater surface area. The analysis assumes the end shear stresses occur over one-quarter of the plug length. A more detailed evaluation of the effects of plug length by the finite element method might not show the same degree of benefit in reducing end shear effects by lengthening the plug.

The preliminary thermal calculations have important implications regarding the design of large-scale seals in a high-temperature environment. It is likely that thermal stresses for a constrained plug could result in shear failure along the interface at the plug ends. Because thermal stresses are self-relieving loads, the stress relief that occurs at the ends might not propagate to the center of plug, which might remain intact under a state of compressive thermal stress.

The manner that the plug is keyed into the rock when subjected to a temperature environment may reduce the incidence of large end shear stresses. A plug keyed into the rock would result in end reactive compressive stress. Further, the effects of plug tapering in the keyway may be beneficial.

The analysis for rock stresses for combined dead and thermal loading ($\theta = 0^\circ$) shows that thermal stresses are dominant. No difference exists between maximum and minimum principal rock stresses at the plug end for the $\theta = 0^\circ$ or the $\theta = 90^\circ$ orientations.

The high end shear stresses at point A might also develop within the rock, resulting in potential failure where the plugs resist thermal loading in end shear. This shear stress may be reduced by keying the plug into the rock or tapering the plug. The rock stresses at point B range from 3.3 MPa to 22.9 MPa. A comparison with the dead load only at point B for the two orientations reveals a contrasting behavior for the $\theta = 0^\circ$ and $\theta = 90^\circ$ orientations at the plug midheight. For the $\theta = 0^\circ$, the maximum shear stress increases (3.7 MPa to 9.9 MPa) when the plug is subjected to increased temperature. For the $\theta = 90^\circ$ orientation, the maximum shear stress state is compressive. The increase in interface stress will tend to reduce the maximum shear stress that occurs in the rock. The converse is true when a tensile zone exists within the rock mass.

F.3 Deep Station and Drift Plugs

The results of the stress calculation for the deep station plug show a slight reduction in stress for dead loads when considering the active lateral earth pressure and the full pore pressure. These results are presented in Figure F-5. Similarly, the lateral pressures for a drift plug are insignificant.

In considering thermal stress, the state of stress is essentially equivalent to the previous analysis of the deep shaft plug.

F.4 Conclusions

The preliminary calculations provide guidance to the design effort by providing a basis for the selection of preliminary conservative design requirements and by suggesting areas in which the analysis and design of plugs should be further refined. The major conclusions are:

(a)

(b)

Figure F-5. Mohr Circles Showing the Effect of Plug Length on Dead Load Stress in Station Plug: (a) Point A and (b) Point C

- The preliminary calculations show the comparative ease of designing an anchor-to-bedrock plug that supports backfill at a shallow depth in a lower temperature environment in comparison to plugs at or near the potential repository horizon that support backfill at a greater depth in a higher temperature environment.
- The preliminary calculations show the comparative ease of designing the deep shaft plug or the station plug for dead loads in comparison to combined dead-plus-thermal loading. It may be advantageous to locate shaft plugs or station plugs away from the high-temperature environment of the potential repository. For the deep shaft or the station plug, this might be done by selecting seal locations within a shaft pillar, away from waste emplacement areas. For a deep shaft plug, this might be done by locating the plug above the potential repository horizon.
- The calculations show a comparative difficulty in designing a plug within a high-temperature repository environment. The manner in which the plug is keyed into the rock or tapered may be an important design consideration. An alternative approach might be to allow localized failure at the ends of the plug and to design a central core that will not fail under triaxial compression.
- The calculations show the orientation of the stress field in the rock that might result in tension and compression zones around the shaft. The structural response of the rock mass at midheight to plug emplacement will be different in these zones, with maximum shearing stresses potentially increasing in tension zones.

For either shallow or deep plugs in a nonthermal environment, the analysis suggests the adequacy of 3,000 psi concrete (21 MPa). For thermal environments, the analysis suggests the adequacy of 10,000 psi (69 MPa) high-strength concrete and the need to increase plug length and to change the plug shape to resist thermal loading.

APPENDIX G
REFERENCE INFORMATION BASES

APPENDIX G

REFERENCE INFORMATION BASES

- **Candidate Information for the Reference Information Base**

This report contains no candidate information for the Reference Information Base.

- **Candidate Information for the Site and Engineering Properties Data Base**

This report contains no candidate information for the Site and Engineering Properties Data Base.

- **Information from the Reference Information Base Used in the Report**

The values indicated in Table G-1 below were taken from the Reference Information Base and used in this report. Their use is intended to provide guidance in fielding tests.

Table G-1. Values Used in This Report

Parameter	Section	Report Value	Reference Information Base Value	Reference Information Base Section	Explanation
Hydraulic Conductivity of Intact Rock	Tables B-1 and B-2	1.0×10^{-4} 1.0×10^{-7} m/s	4×10^{-5}	1.4.4	Range of values selected for parametric analysis
Far-Field Stress	Table C-1	1.38 to 8.43 MPa	1.71 to 6.46 MPa	1.2.10	Range of values selected for parametric analysis
Rock Young's Modulus	Table D-1	7800	7800	1.2.10	
Rock Poisson's Ratio	Table D-1	0.16	0.16	1.2.10	
Rock Thermal Expansion	Table D-1	6.4×10^{-6} °C ⁻¹	6.4×10^{-6} °C ⁻¹	1.2.3	
In Situ Stress	Table D-1	2.4 MPa	2.66	1.2.10	
Plug Diameter	Table F-1	8.7-9.6 m	None	None	
Plug Density	Table F-1	2400 kg/m ³	None	None	
Plug Young's Modulus	Table F-1	34.6 GPa	None	None	
Plug Coefficient of Thermal Expansion	Table F-1	1.0×10^{-5} /°C 8.8×10^{-6} /°C	None	None	
Depth to Plug	Table F-1	10-310 m	None	None	
Backfill Density	Table F-1	1920 kg/m ³	None	None	
Internal Angle of Friction	Table F-1	35°	None	None	

Table G-1. Values Used in This Report (Continued)

Parameter	Section	Report Value	Reference Information Base Value	Reference Information Base Section	Explanation
Anchor-to-Bedrock Plug - Rock Density - Rock Young's Modulus - Rock Poisson's Ratio	Table F-1	1930 kg/m ³ 20 GPa 0.10	2200 20 GPa 0.10	1.2.1 1.2.10 1.2.10	The analysis in Appendix F was performed prior to the current version of the Reference Information Base. The differences are not considered significant.
Deep Plug - Density - Young's Modulus - Poisson's Ratio	Table F-1	2113 kg/m ³ 15.26 Pa 0.22	2297 16.4 0.26	1.2.1 1.2.10 1.2.10	
Deep Plug - Thermal Expansion	Table F-1	8.8 x 10 ⁻⁶ °C ⁻¹	8.2 x 10 ⁻⁶ °C ⁻¹	1.2.3	

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