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Nevada Nuclear Waste Storage Investigations Project

Technical Basis for Performance Goals, Design Requirements, and Material Recommendations for the NNWSI Repository Sealing Program

Joseph A. Fernandez, Peter C. Kelsall, John B. Case,
Dann Meyer

Prepared by
Sandia National Laboratories
Albuquerque, New Mexico 87185 and Livermore, California 94550
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TECHNICAL BASIS FOR PERFORMANCE GOALS, DESIGN REQUIREMENTS, AND MATERIAL RECOMMENDATIONS FOR THE NNWSI REPOSITORY SEALING PROGRAM

Joseph A. Fernandez
NNWSI Geotechnical Design Division
Sandia National Laboratories
Albuquerque, NM 87185

Peter C. Kelsall
John B. Case
Dann Meyer
IT Corporation
2340 Alamo S.E.
Albuquerque, NM 87106

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Abstract

This report presents the initial attempt to establish the technical basis for developing seal designs for the Nevada Nuclear Waste Storage Investigations Repository Sealing Program. This report supports the Advanced Conceptual Design effort, which is the next phase in the design process of evaluating a nuclear waste repository in the unsaturated zone at Yucca Mountain, Nye County, Nevada. Because the site geohydrologic data forming the basis for this report are preliminary, the technical basis for sealing designs may change as additional site information is acquired through site characterization activities. The specific objectives of this comprehensive study are to develop performance goals, to assess the need for seals, to define design requirements, and to recommend potential sealing materials for the sealing system. Performance goals are the allowable amounts of water that can enter the waste disposal areas directly from the rock mass above the repository and indirectly from shafts and ramps connecting to the underground facility. These goals are developed using a numerical model that calculates radionuclide releases. To determine the need for sealing, estimates of water flow into shafts, ramps, and the underground facility under anticipated conditions are developed and are compared with the performance goals. It is concluded that limited sealing measures (such as emplacement of shaft fill) are sufficient to properly isolate the radioactive waste in the repository. Nevertheless, a broad range of sealing design options and associated hydrologic design requirements are proposed to provide a greater degree of assurance that the hydrologic performance goals can be met even if unanticipated hydrologic flows enter the waste disposal areas. The hydrologic design requirements are specific, hydraulic conductivity values selected for specific, seal design options to achieve the performance goals. Using these hydrologic design requirements and additional design requirements, preferred materials are identified for continued design and laboratory analyses. In arriving at these preferred materials, results from previous laboratory testing are briefly discussed.

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EXECUTIVE SUMMARY

The Nevada Nuclear Waste Storage Investigations (NNWSI) Project, managed by the Nevada Operations Office of the U.S. Department of Energy, is examining the feasibility of developing a nuclear waste repository in an unsaturated tuff formation beneath Yucca Mountain. Yucca Mountain is located on and adjacent to the Nevada Test Site, Nye County, Nevada. The sealing subsystem is part of the Yucca Mountain Mined Geologic Disposal System (YMMGDS).

Seals may be required to limit water flow into or out of the underground facility and air flow out of the underground facility. Two potential sources of water include flow (down shafts) of surface flood waters resulting from major precipitation events and inflow directly into the underground facility from faults. To determine the need and extent of sealing in the YMMGDS, a sealing program was initiated. The first step in the sealing program was the development of sealing concepts (Fernandez and Freshley, 1984). The second step was the preparation of a detailed program plan (Fernandez, 1985) based on the sealing concepts. This report represents the third major step in the sealing program: the development of the performance goals, design requirements, and material recommendations. Both horizontal and vertical emplacement configurations are considered. The technical basis is provided for the design requirements and materials recommended for use in the Advanced Conceptual Design.

Performance goals are the allowable amounts of water that can enter the waste disposal areas. In developing these performance goals, specific criteria in 10 CFR 60 (NRC, 1986) are considered. The qualitative design criteria for seals for shafts and boreholes are given in 10 CFR 60.134. The quantitative criteria given in Section 60.113 are used to develop the performance goals for the engineered barrier subsystem and the shaft and borehole seal (shaft portion only) subsystem. The performance goals are derived by computing the volume of water required to release radionuclides in amounts equal to the annual release rate established by the Nuclear Regulatory Commission (NRC) in 10 CFR 60 for all radionuclides of concern. This annual release rate is computed using two radionuclide release models. The first model assumes a <u>constant</u> rate of failure of the waste packages (beginning 300 years following closure of the repository) and release of radionuclides from areas outside of the uranium dioxide matrix. The second model assumes all of the radionuclides are located in the matrix and all waste packages have failed 300 years after closure. The releases from the second model are computed using the SPARTAN code (Lin, 1985). From the calculation in Chapter 3 hydrologic performance goals for the entire sealing subsystem are selected. The performance goals increase from 1,180 m^3/year (at 300 years following closure) to 136,000 m^3/year (at 1,000 years following closure). These goals apply to the underground facility and the shafts and ramps. The performance goals increase because the radionuclide inventory of the radionuclides that are used to compute performance goals decreases after closure of the repository. During the period in which containment is required to be substantially complete,

i.e., 300 to 1,000 years following closure, the performance goals are lower than after 1,000 years following closure.

The qualitative criteria given in Section 60.134 are used in developing the hydrologic performance goal for the shaft and borehole seals subsystem (borehole portion only). As indicated in Section 60.134, "boreholes shall be designed so that following closure they do not become pathways that compromise the geologic repository's ability to meet performance objectives." For our current planning basis, we adopt the position that this requirement is satisfied if the potential for vertical flow through boreholes is one percent or less of the potential for vertical flow through the host rock. Similarly, a performance goal for airborne releases is selected for the shafts. We adopt the position that the total, gaseous radionuclide release through shafts should be less than one percent of the allowable release limits given in 40 CFR 191 (EPA, 1986) for specific radionuclides. These performance goals will be reinvestigated as additional site information becomes available from site characterization activities and as the repository design is refined.

To determine the need for sealing, anticipated inflows into waste disposal areas are compared to the performance goals. Anticipated conditions are derived from the current understanding of hydrologic flow at Yucca Mountain. Because hydrologic data are limited, engineering judgment is used in defining the conditions that are evaluated. Engineering judgment is supplemented by analytical solutions for water flow to obtain estimates of water flow into shafts and the underground facility.

After comparing the anticipated, water inflows with the performance goals, it is concluded that only limited sealing efforts (such as the emplacement of a general fill in the shafts and ramps) are necessary to meet the quantitative NRC performance requirements. However, a broad range of design options and associated design requirements are proposed to provide a greater degree of assurance that the performance goals can be met even if unanticipated conditions are encountered in the repository. In Chapter 6, design options are presented and associated design goals and design requirements are identified.

The logic used to arrive at preferred design options (or sealing components), goals, and design requirements is as follows. First, hydrologic performance goals are allocated specifically to the shafts and ramps and the underground facility. The hydrologic performance goals refer to the allowable amounts of water that could contact the waste packages and not result in releases that exceed (even for unanticipated flow scenarios considered) the annual release rates established by the NRC in 10 CFR 60.113.a.1.ii.B. Performance goals are shown schematically in Figure 1-4 by the maximum-allowable-performance-goal curve. To add additional conservatism to the sealing activities, this curve is interpreted as the amount of free water allowed to enter the underground facility (not just that allowed to contact the waste). Even more conservatism has been added by the selection of the design-basis-performance goals (particularly during the first 1,000 years after closure) to be substantially less

than the maximum-allowable values. The design-basis-performance goals for the sealing subsystem are identified in Table 6-1. The goals for the total quantity of flow are then allocated to both the underground facility and to the shafts and ramps. The subsystem goals were then further subdivided into the tentative design goals for the various sealing components; these goals for the components are provided in Table 6-2.

Using design calculations and the design goals, design requirements can be developed. The design requirements are based on simple hydrologic models that assume fully saturated conditions and Darcy flow. Because the repository is in the unsaturated zone, the assumption of saturated conditions overestimates the flow through the rock. In this way, conservative design requirements are developed. Because the performance goals for the sealing system relate mainly to reducing or diverting water flow, the bulk of the design requirements relate to hydrologic properties such as hydraulic conductivity. Additional design requirements considered in Chapter 7 include strength, density, emplacement considerations, longevity, and cost.

To arrive at a design life for sealing components in the underground facility and the shafts and ramps subsystems, portions of the performance goal for the entire sealing subsystem are allocated to each of these subsystems. Allocation of performance goals considers separately the estimated amounts of water that enter either the underground facility and shafts and ramps considering unanticipated, water flow scenarios. In arriving at the time period in which to allocate a portion of the entire performance goal, it is assumed that the most severe unanticipated scenario (of those considered in this report) occur yearly following a containment period of 300 years. The difference between the performance goal and the unanticipated flow represents the reduction of flow required to meet the performance goal. In Chapter 6, no reduction of flow is required for sealing components in the underground facility about 500 years after closure and for sealing components in the shafts and ramps about 1,000 years after closure. Therefore, sealing measures are not necessary after this time period. The seals emplaced at closure would, of course, remain in place over the long term together with the shaft and ramp fill.

Specific design requirements are developed for the sealing components within shafts, ramps, and the underground facility. Preferred design options are also selected. For shafts and ramps, the preferred option* is the anchor-to-bedrock plug. For the underground facility (vertical emplacement configuration), the preferred options to encourage vertical infiltration of water are earthen dams and sumps that may, in certain nonemplacement areas, be supplemented with drainage channels. In emplacement drifts the preferred option is to store no waste downgrade from where the inflow occurs. For the horizontal emplacement configuration, dams are also preferred to encourage vertical

*The preferred option is the principal sealing component relied on to achieve the hydrologic performance goal which involves water control.

infiltration of water into the rock mass. If water is encountered in emplacement boreholes, the preferred option is to abandon the hole. For both the vertical and horizontal emplacement configurations, it is preferred that water control, if needed, be achieved by designing the repository to accommodate the water inflow conditions encountered.

Beyond specific design requirements associated with the sealing design options, it is proposed that a water storage capacity of 10,000 m^3 be provided in the lowest point of the repository. It is intended that this storage volume can accommodate the potential water inflow calculated for extreme conditions, although the probability of such conditions is considered to be very low.

The preferred design option for sealing exploratory boreholes drilled from the surface is the emplacement of a seal within the Calico Hills unit. Two requirements imposed in the design of the underground facility are (1) the underground facility does not intercept any borehole, and (2) a minimum distance of 15 m occurs between a drift excavation and the borehole.

The materials selection process described in this report considered the following requirements or conditions:

- hydraulic conductivity,
- strength or density,
- emplacement considerations,
- potential influence of seals on ground-water chemistry, and
- emplacement conditions
  - host rock lithology/chemistry
  - temperature range at seal location.

From this screen, materials identified to be acceptable are

- concrete,
- grout,
- crushed tuff,
- crushed tuff and clay, and
- crushed tuff, fines removed.

Our current preference is to propose earthen materials for as many applications as possible to limit degradation of physical properties and adverse effects on ground-water chemistry in the repository environment. For higher temperature environments, within the emplacement drifts, earthen materials are anticipated to be more stable than concretes and grouts. The impact of earthen materials on waste package corrosion, waste form solubility and mobility of radionuclides is considered to be less severe than the impact of cementitious-based materials. This conclusion is reached by comparing potential increases in pH and cationic species in the chemistry of ground water that contacts the seals. Therefore, earthen materials are preferred for use in emplacement drifts. Earthen materials also may be used in places where low strength is acceptable. The most likely application for cementitious materials is where high strength and low deformability may be required (e.g.,

in some bulkheads) or where high strength combined with pumpability may be required (e.g., for grouting fractures).

1.0 INTRODUCTION

1.1 PURPOSE OF REPORT

The Nevada Nuclear Waste Storage Investigations (NNWSI) Project, managed by the Nevada Operations Office of the U.S. Department of Energy, is examining the feasibility of developing a nuclear waste repository in an unsaturated tuff formation beneath Yucca Mountain. Yucca Mountain is located on and adjacent to the Nevada Test Site, Nye County, Nevada. The sealing subsystem is part of the Yucca Mountain Mined Geologic Disposal System (YMMGDS). The role of the sealing subsystem in achieving the overall performance objective for the YMMGDS is currently being evaluated by Sandia National Laboratories.

To determine the need and extent of sealing in the YMMGDS, a sealing program was initiated. To manage the technical activities, this program was divided into three Work Breakdown Structures (WBS). All of these WBS categories, illustrated in Figure 1-1, contribute to documentation supporting the License Application for the NNWSI Project. The first step in this documentation process was the development of sealing concepts (Fernandez and Freshley, 1984). The second step was the preparation of a detailed program plan (Fernandez, 1985) based on the sealing concepts. This report represents the third major step in the sealing program: the development of the performance goals, design requirements, and material recommendations. Both the design requirements and the materials recommendations provide the basis for the Advanced Conceptual Design, as illustrated on Figure 1-1. The efforts presented in this report are part of the seal performance requirements task (WBS 1.2.4.2.3.1) and the materials development task (WBS 1.2.4.2.3.2).

This report presents the design requirements and recommends materials to be used in the NNWSI Repository Sealing Program. Design requirements include (1) the design options that should be considered in arriving at a suitable sealing subsystem design, (2) quantitative and qualitative requirements imposed on sealing design options that must be met to achieve a desired performance, and (3) site conditions that must be considered in the design. These requirements represent our current design basis for arriving at suitable sealing component designs.

To establish the need for sealing, minimum desired performance goals are computed. These performance goals are specific values of how well the sealing subsystem must perform its function. This function is to contribute to the containment and isolation of radioactive wastes by controlling the water flow into the waste disposal areas and by restricting the radionuclide releases, both waterborne and airborne, to the accessible environment.

In establishing the need for sealing, the water flows entering the repository, under both anticipated and unanticipated conditions, are compared to the performance goals. If the need for sealing is established, the design requirements are developed. By evaluating likely water-flow conditions (anticipated

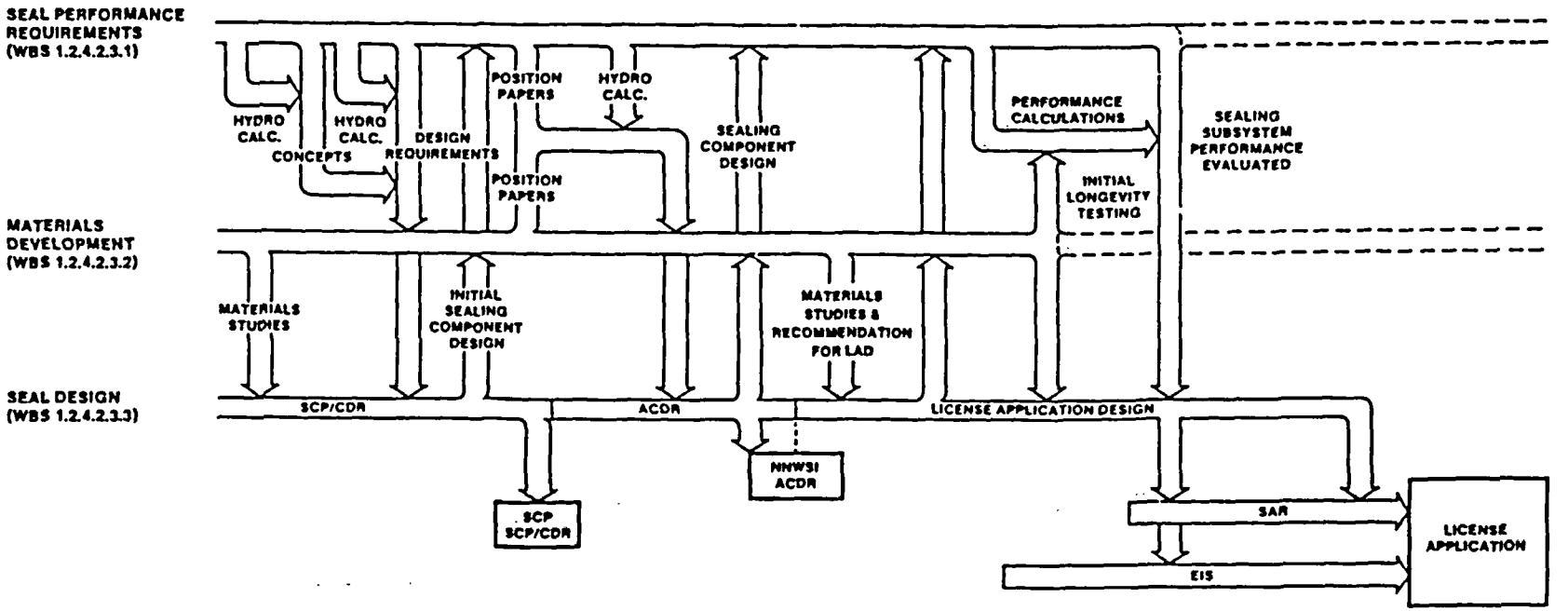


Figure 1-1. Schematic of the NNWSI Repository Sealing Program Through License Application (Modified From Fernandez, 1985, p. 56)

case) and unlikely water-flow conditions (unanticipated case), a broad range of design requirements will be developed that can be applied to a broad range of site conditions that may be encountered. It is also possible that sealing may not be required but may be implemented to give greater assurance that the performance goals can be met.

As (1) additional hydrogeologic data are obtained through site characterization, (2) approaches to allocating performance to various subsystems within the YMMGDS are refined, and (3) the exploratory shafts and the associated testing results are developed, the performance goals, and subsequently the design requirements and material recommendations, may be modified.

1.2 REGULATORY REQUIREMENTS APPLICABLE TO THE NNWSI REPOSITORY SEALING PROGRAM

In understanding the applicable regulatory requirements that impact the NNWSI Repository Sealing Program, it is important to understand how "sealing" is being used in this report, the components that comprise the sealing program, and which subsystems within the YMMGDS incorporate these components. With this understanding, it is possible to determine which regulatory requirements are appropriate for use in the NNWSI Repository Sealing Program.

Within the NNWSI Repository Sealing Program, sealing is defined as the permanent closure of the underground facility, shafts, ramps, and boreholes. It includes emplacing shaft and drift backfill, seals or plugs in shafts, ramps, drifts, and boreholes; and isolating discrete, water-producing zones from the waste packages. The current planning basis for the sealing program is to emplace backfill in the underground facility. Like the seals and plugs, backfill will be emplaced during the decommissioning period.

The sealing subsystem is part of the YMMGDS. The performance objective of the YMMGDS is to ensure that the radionuclide releases from the repository and to the accessible environment are within the limits specified by the U.S. Environmental Protection Agency (EPA). Systems within the YMMGDS are divided into subsystems associated with preclosure waste disposal and postclosure waste disposal. Components associated with the sealing subsystem are included in the postclosure time period. As illustrated in Figure 1-2, the postclosure waste disposal system is divided into natural, engineered, and institutional barriers. All of the sealing components are classified under the repository engineered barriers subsystem. Shaft and borehole seals are distinguished from sealing components located within the underground facility. Therefore, backfill and all seals or plugs in the underground facility are part of the repository engineered barriers subsystem.

In arriving at a strategy for developing hydrologic performance goals, the 40 CFR 191 (EPA, 1986) and 10 CFR 60 (NRC, 1986) standards and regulations were evaluated for their applicability. The EPA standards include isolation requirements for the entire disposal system. The standards do not specify

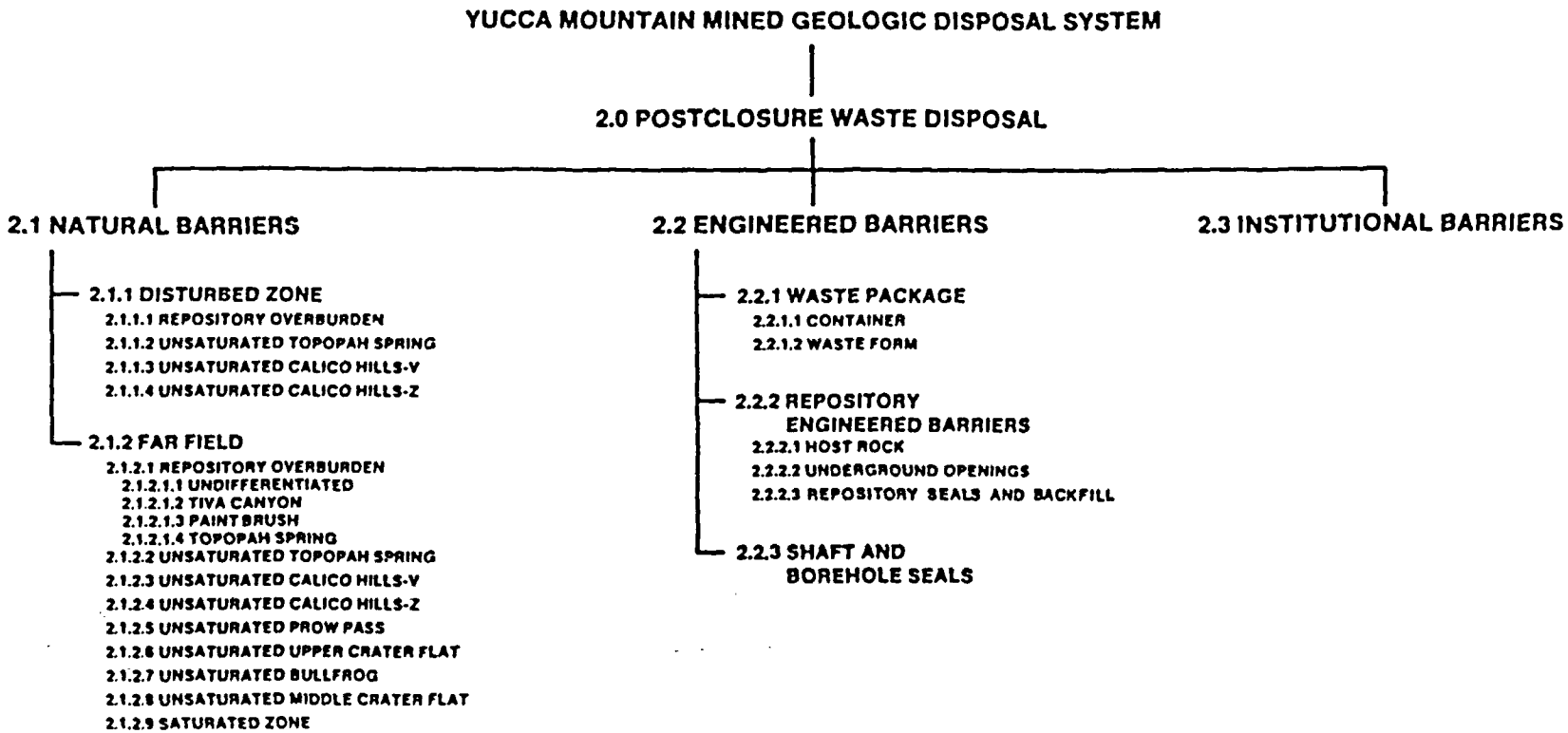


Figure 1-2. Subsystem and Subsystem Components Associated With the Post-Closure Period of the YMMGDS (Modified From SNL, 1987b, Appendix P, p. 11/21)

requirements for individual subsystems that are part of the disposal system. The EPA criteria provide release limits for specific radionuclides considering all releases from significant processes and events that may affect the disposal system. Because the EPA standards apply to the cumulative radionuclide releases from the overall disposal system to the accessible environment, it is inappropriate to use these standards in arriving at performance criteria for the sealing components without considering all of the systems comprising the disposal system. Uncertainties associated with the transport through the geologic system to the accessible environment must be considered together with an assessment of how anticipated and unanticipated processes and events affect the release of radionuclides. Such an evaluation is beyond the scope of this study, but will be performed as part of the Safety Analysis Report which will be part of the License Application.

In 10 CFR 60, performance objectives are given for the engineered barrier subsystem, and design criteria are given for the engineered barriers and the seals for shafts and boreholes. Numerical limits are defined as part of the performance objective, whereas the design criteria are qualitative. Section 60.113 of 10 CFR 60 states the performance objective of the engineered barrier system as follows:

> "(ii) . . ., the engineered barrier system shall be designed, assuming anticipated processes and events, so that: . . . (B) The release rate of any radionuclides from the engineered barrier system following the containment period shall not exceed one part in 100,000 per year of the inventory of that radionuclide calculated to be present at 1,000 years following permanent closure, or such other fraction of the inventory as may be approved or specified by the Commission; provided, that this requirement does not apply to any radionuclide which is released at a rate less than 0.1% of the calculated total release rate limit. The calculated total release rate limit shall be taken to be one part in 100,000 of the inventory of radioactive waste, originally emplaced in the underground facility, that remains after 1,000 years of radioactive decay."

The qualitative design criteria for seals for shafts and boreholes as given in Section 60.134 of 10 CFR 60 are divided in two categories:

> "(a) 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.
>
> (b) Selection of materials and placement methods. Materials and placement methods for seals shall be selected to reduce, to the extent practicable: (1)

> The potential for creating a preferential pathway for ground-water; or (2) radioactive waste migration through existing pathways."

The quantitative criteria given in Section 60.113.ii are used to develop the performance goals for the engineered barrier subsystem and the shaft and borehole seal subsystem (shaft portion only). The qualitative criteria given in Section 60.134.(a) and (b) are used in developing the performance goal for the shaft and borehole seals subsystem (borehole portion only).

1.3 NNWSI ISSUES PERTINENT TO THE REPOSITORY SEALING PROGRAM

As indicated in the Nuclear Waste Policy Act of 1982 (NWPA, 1983, p. 2211) and in 10 CFR 60 criteria (NRC, 1986, p. 603), a Site Characterization Plan (SCP) shall be prepared for candidate sites for a nuclear waste repository. The purpose of the SCP is to identify those issues at a specific proposed site and the plans for resolving those issues. The site characterization efforts at proposed sites are, therefore, centered on issues defined in the SCP.

The issues hierarchy is divided into three levels: key issues, issues (including characterization, performance, and design issues), and information needs. Key issues are broad questions of overall suitability; issues are more specific; and information needs are generally data or analyses about the natural or engineered systems required to address the issues. The key issues, issues, and information needs pertinent to the NNWSI Repository Sealing Program are given in Table 1-1.

As indicated in Table 1-1, issues pertinent to sealing fall under two key issues. These issues deal with the isolation of waste from the accessible environment after closure by the YMMGDS and the feasibility of closing the repository in a cost-effective manner using available technology. In this report, both issues are addressed through the information needs. Available site, waste package, and underground facility information (information need 1.12.1) is used in the development of the design requirements. The recommendations of potential sealing materials, which are based on the design requirements, address information need 1.12.2. Because the design requirements and the recommended materials provide the basis for the advanced conceptual design and all future sealing component designs, information needs 1.12.3, 1.12.4, and 4.4.10 are addressed.

1.4 APPROACH USED IN THIS REPORT

The approach used in this report to establish the design requirements for engineered, sealing components is presented below. The results obtained from this approach represent the current planning basis for the sealing program. As additional site information is obtained, the planning basis for the sealing program will be modified. Therefore, the calculations presented in this report are preliminary and subject to change.

Table 1-1. Selected Key Issues, Issues, and Information Needs Significant to the NNWSI Repository Sealing Program (1-16-87 version)

KEY ISSUE 1:	Will the mined geologic disposal system at Yucca Mountain isolate the radioactive waste from the accessible environment after closure in accordance with the requirements set forth in 40 CFR Part 191, 10 CFR Part 60, and 10 CFR Part 960?
DESIGN ISSUE 1.12:	Have the characteristics and configurations of the shaft and borehole seals been adequately established to (a) show compliance with the postclosure design criteria of 10 CFR 60.134, and (b) provide information to support resolution of the performance issues?
INFORMATION NEED 1.12.1:	Site, waste package, and underground facility information needed for design of seals and their placement methods.
INFORMATION NEED 1.12.2:	Materials and characteristics for seals for shafts, drifts, and boreholes.
INFORMATION NEED 1.12.3:	Placement methods for seals for shafts, drifts, and boreholes.
INFORMATION NEED 1.12.4:	Reference design of seals for shafts, drifts and boreholes.
KEY ISSUE 4:	Will mined geologic disposal system construction, operation (including retrieval), closure, and decommissioning be feasible at Yucca Mountain on the basis of reasonable available technology, and will the associated costs be reasonable in accordance with the requirements set forth in 10 CFR Part 960?
DESIGN ISSUE 4.4:	Are the repository construction, operation, closure, and decommissioning technologies adequately established to support resolution of the performance issue?
INFORMATION NEED 4.4.10:	Identification of technologies for emplacement of seals for accesses, drifts, and boreholes.

The steps involved in arriving at the design requirements are schematically presented in Figures 1-3 through 1-5. These figures illustrate the primary products that are developed in specific sections in this report. The intent of Chapter 3 is to select hydrologic performance goals for the sealing subsystem. Hydrologic performance goals, as used in this report, refer to the allowable amount of water that could enter the waste disposal areas. A performance goal represents the desired performance from a system or subsystem and can incorporate a margin of safety. Further, performance goals can change during the design process if additional information is acquired or the design approach changes.

In selecting performance goals, the results from two radionuclide release models are considered under the assumption that water entering the waste disposal areas contacts the waste. The first model assumes a constant rate of failure of the waste package and release of radionuclides from areas outside of the uranium dioxide matrix. The result from this model is displayed as the maximum-allowable-performance-goals curve in Figure 1-3. This curve represents realistic yet conservative* performance goals. The second model assumes all of the radionuclides are located in the matrix and all waste packages have failed 300 years after closure of the repository. The results from this model are displayed as the design-basis-performance-goal curve shown in Figure 1-4. This curve incorporates the unrealistic assumption of total, waste package failure at 300 years after closure. Both models represent the most stringent hydrologic performance goals for all of the radionuclides investigated.

In contrast to Chapter 3, the focus of Chapter 4 is to compute the expected rather than the allowable amount of water that could enter the waste disposal area and contact the waste. Both anticipated conditions and selected, unanticipated scenarios are evaluated to arrive at volumes of water that could potentially enter the waste disposal areas. Anticipated conditions are those processes and events that are reasonably likely to occur during the time period in which the repository performance objective must be met. For example, an anticipated condition could include periodic runoff in each of the drainage basins associated with the shafts. A portion of this runoff could then enter the upper portion of the shafts. Unanticipated scenarios are those processes and events that are not reasonably likely to occur during the same period but are sufficiently credible to warrant evaluation. An anticipated scenario could include a probable maximum flood (PMF) occurring simultaneously at each shaft location and no sheetflow of the PMF waters occurring from the basins. Unanticipated scenarios that are not sufficiently credible or are without consequence are not considered in this report. The specific

*The authors feel that the rate of failure of the waste packages assumed overpredicts the rate of failure that will actually occur. Therefore, the maximum-allowable-performance goals are lower than what would be computed if a more realistic rate of failure of the waste packages was assumed. In this sense, the maximum-allowable-performance goals in this report are conservative.

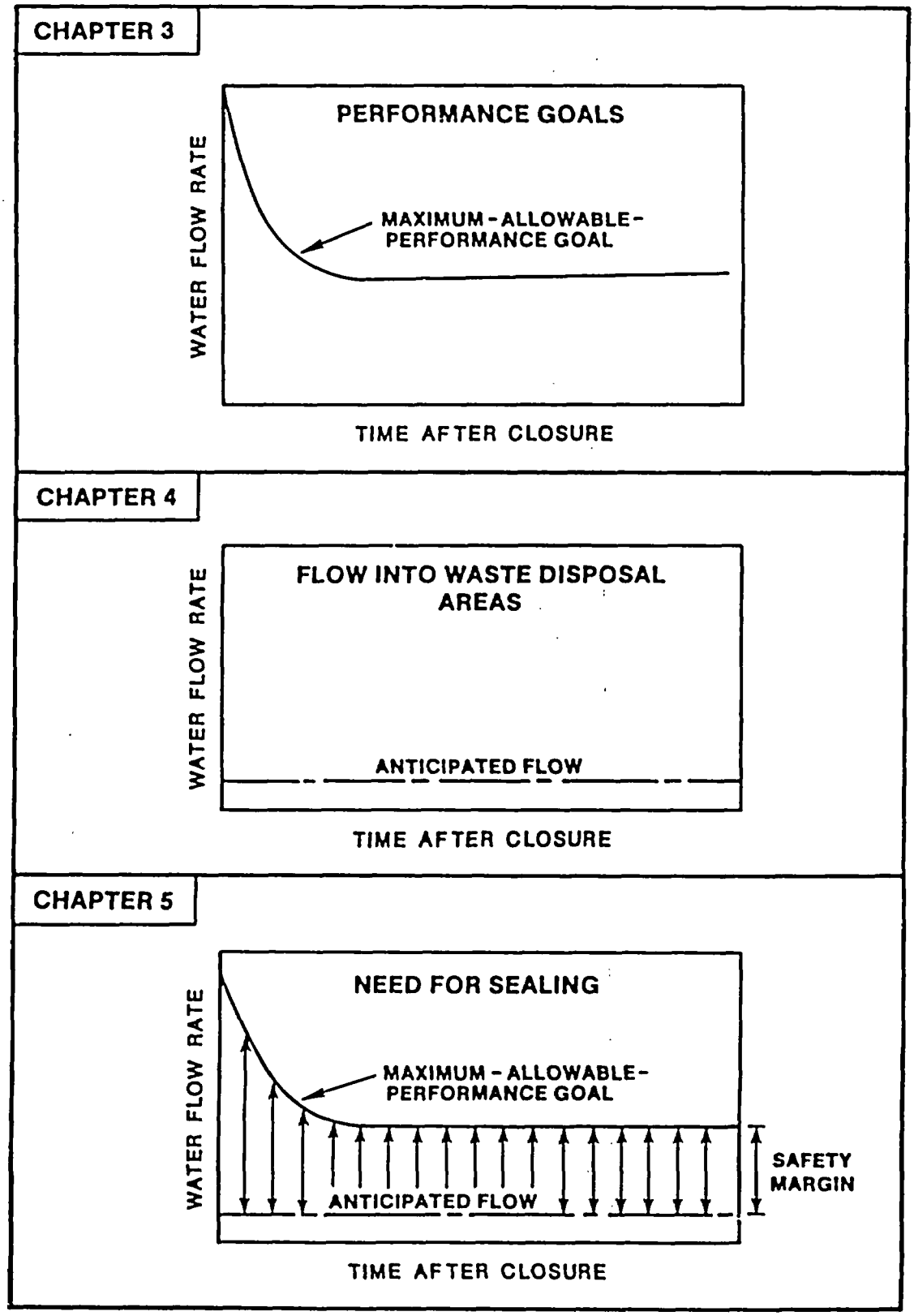


Figure 1-3. Approach Used in Determining Design Requirements

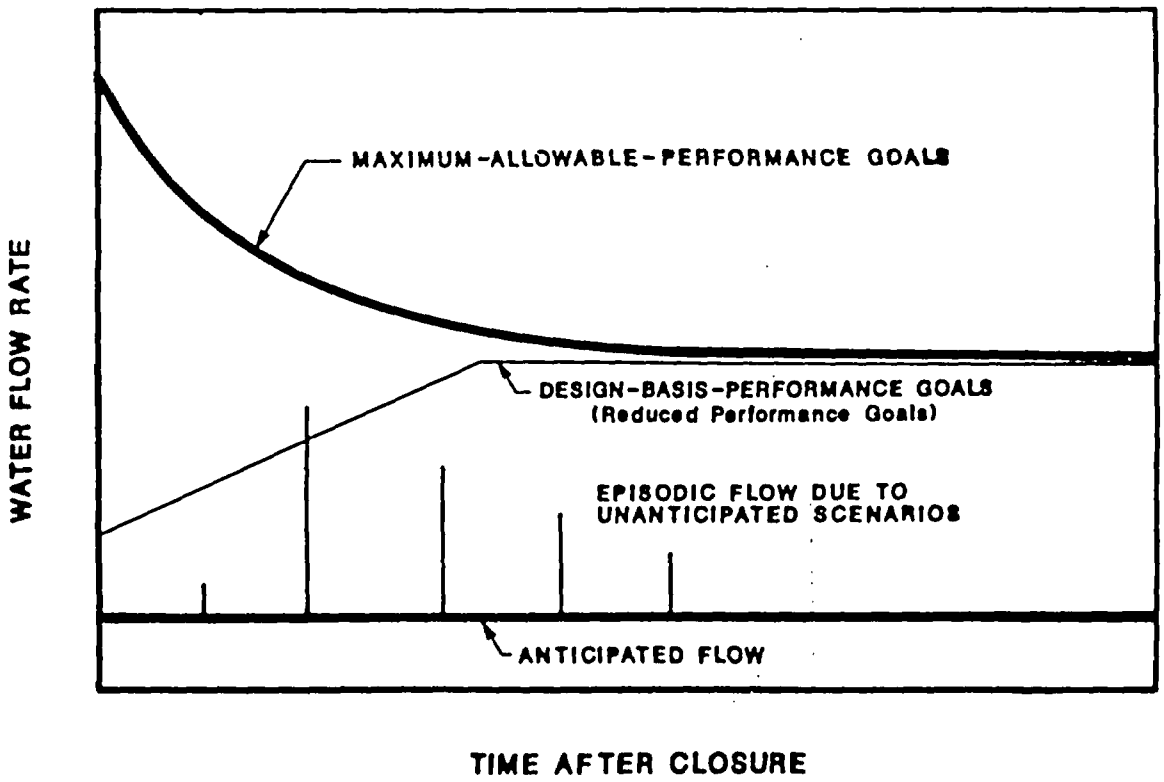


Figure 1-4. Comparison of Maximum-Allowable-Performance Goals and Design-Basis-Performance Goals to Anticipated and Unanticipated Episodic Flows

conditions that are considered in arriving at anticipated and unanticipated flows are described in more detail in Chapter 4.

An assessment of the need for sealing is made (Chapter 5). This assessment includes comparing two relationships: the maximum allowable amount of water entering the waste disposal area and the expected amount of water that could enter the waste disposal area under anticipated conditions. The difference between these curves represents a safety margin in the decision not to seal.

In Chapter 5, it is concluded that only limited sealing measures are necessary to properly isolate the radioactive waste in the underground facility. Nevertheless, design options are proposed in Chapter 6 to provide additional assurance that repository performance objectives will be met. To establish the time period in which sealing should be proposed, the reduced, performance goals developed in Chapter 3 are compared with the volumes of water entering the waste disposal areas for selected, unanticipated scenarios described in Chapter 4. This comparison is schematically shown in Figure 1-4. The reduced-performance-goals curve is selected as our design-basis-performance-goal curve because it reduces the amount of water that is allowed to enter the waste disposal area during the period in which containment is required to be substantially complete, i.e., 300 to 1,000 years following closure. This

reduction of the performance goal represents an additional degree of conservatism in our analysis. It is noted that even when comparing our design-basis curve with the water flow estimated to enter the waste disposal areas under anticipated conditions, our design-basis curve is not exceeded for all time. It is only when considering unanticipated scenarios of low probability that our design-basis curve is exceeded. This excess can potentially occur only up to approximately 1,000 years following closure. This time is selected as the design life for proposed sealing components.

To arrive at a design life for sealing components in the underground facility and the shafts and ramps subsystems, portions of the design-basis-performance goal are allocated to each of these subsystems. Allocation of performance goals considers separately the estimated amounts of water that enter the underground facility and the shafts and ramps under unanticipated conditions. This allocation is schematically shown in Figure 1-5. The allocation is made such that the allocation to the shafts and ramps plus the allocation to the underground facility is equal to the performance goal for each specified year. Mathematically, the relationship may be expressed as

$$T(t) = S(t) + U(t)$$

where

$T(t)$ = performance goals for the entire sealing subsystem,
$S(t)$ = performance goals allocated to seals located in the shaft and ramp subsystem,
$U(t)$ = performance goals allocated to seals located in the underground facility subsystem, and
t = time.

In arriving at the time period in which to allocate a portion of the entire performance goal, the unanticipated scenarios are considered as a continuous flow for all time. These unanticipated flows, we believe, represent an extremely conservative, upperbound estimate of the volume of water that could enter the waste disposal areas and contact the waste in any given year during the post-closure period. The difference between the performance goal and the unanticipated flow lines represents the reduction of flow required to meet the performance goal. This reduction of flow becomes progressively less until the maximum-allowable amount of water that can enter the waste disposal area through a sealing subsystem equals the unanticipated flow volumes. At this point, no reduction of flow is necessary and no specific performance is required from the sealing components.

For a subsystem to achieve an acceptable reduction of the unanticipated flow rates, it is necessary for all or a portion of the components comprising the subsystem to contribute to the reduction. Because there can be a number of sealing components included in the subsystem, a portion of the performance goal for the subsystem is allocated to the sealing components included in the

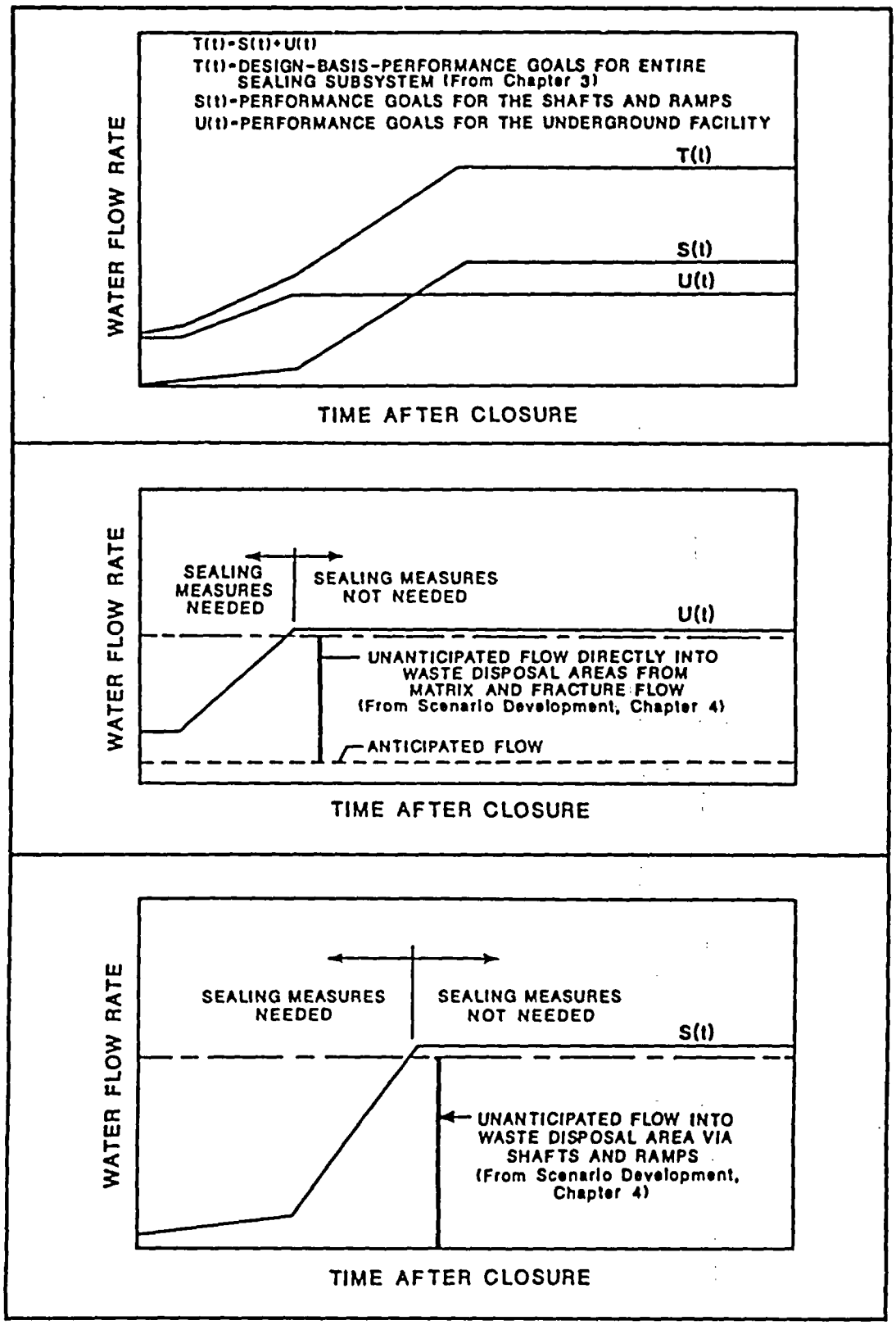


Figure 1-5. Performance Allocation

sealing subsystem. The relationship between the performance goal of the subsystem and the goal established for specific sealing components can be described by the following expressions:

$$S(t) = X_a DG_a(t) - C_s(t)$$

and

$$U(t) = X_a DG_a(t) - C_u(t)$$

where

- $S(t)$ = performance goal for shaft and ramp subsystem,
- $U(t)$ = performance goal for the underground facility subsystem,
- $C_s(t)$ = storage capacity for the shaft and ramp subsystem,
- $C_u(t)$ = storage capacity for the underground facility subsystem,
- X_a = number of a specific sealing component in which some level of performance is required, and
- $DG_a(t)$ = design goal for a specific sealing component in either subsystem.

As indicated by this expression, the design goal can be defined as a specific level of performance required by a specific component. The above expressions are written to take into account storage capacities of the subsystems. Water may enter the repository, and may be stored, and then drained before entering the waste disposal rooms. A storage capacity in the lowest part of the repository is calculated and used in arriving at design goals for sealing components associated with the shafts and ramps. No storage capacity is assumed when computing the design goals for sealing components in the underground facility.

In certain remote areas of the repository where waste is not emplaced, water will not enter waste emplacement rooms and contact the waste if adequate drainage is provided. In these instances, design goals are derived from other considerations. Once design goals for specific sealing components are specified, design requirements can be determined as presented in Chapter 6. Finally, in Chapter 7, sealing materials are selected using the appropriate design requirements.

2.0 REFERENCE CONDITIONS

This chapter summarizes the conditions at Yucca Mountain and the repository design features that may affect seal performance and design. These include site geology, hydrology, repository design, and waste descriptions. These reference conditions will be considered in establishing the need for sealing in Chapters 4 and 5.

2.1 GEOLOGY AND STRUCTURAL SETTING

Th: Yucca Mountain site (Figure 2-1) lies in the Basin and Range physiographic province. This province is characterized by extensional faulting. In general, the faults strike north to northeast. The topographic expression of this faulting is aligned, fault-bounded uplifts (ranges), such as Yucca Mountain, separated by alluvium-filled basins (U.S.G.S., 1984, p. 1).

Yucca Mountain is underlain by Precambrian basement and Paleozoic and Mesozoic sediments, but only Tertiary volcanic rocks outcrop in the vicinity of Yucca Mountain. The Tertiary volcanic rocks consist of a sequence of Miocene silicic ash-flow and ash-fall tuffs, lavas, and breccias that erupted before or during the extensional faulting (U.S.G.S., 1984, p. 3). The degree of welding, devitrification and zeolitization varies throughout the sequence, as does the thickness of individual units. Several members of the Paintbrush Tuff are exposed at Yucca Mountain. The Tiva Canyon Member occurs extensively at the surface, while the older Pah Canyon and Topopah Spring Members outcrop mostly on the steep western slope of Yucca Mountain (Scott and Bonk, 1984, Sheet 1). The sequence of tuff units at Yucca Mountain is shown in Table 2-1. Generally in this report, the simplified hydrogeologic or stratigraphic classifications shown in Table 2-1 are used. The repository, as planned, would be excavated in the Topopah Spring welded unit.

The most significant structural features at Yucca Mountain are the numerous faults mapped at the surface (Figure 2-2). Several faults are continuous, major features that could penetrate deeply to the repository horizon and define a number of distinct structural blocks. As discussed subsequently in Section 4.2, faults that would penetrate from the surface to the repository horizon could become saturated, with a resulting inflow to underground drifts. As currently planned, the repository will be confined to the "central block" (Sinnock et al., 1984, p. 13), which is bounded on the east and west by west-dipping, anticipated faults or fault zones that are downthrown on the west, and to the north by a suspected northwest-southeast-trending fault along Drill Hole Wash (Figure 2-2). The repository could be extended into adjacent blocks if additional disposal area should be needed or as additional site-related data should become available.

Two types of faults occur in the vicinity of the proposed repository location, north-south-trending, anticipated, normal faults and northwest-southeast-trending, strike-slip faults. The major anticipated faults include the

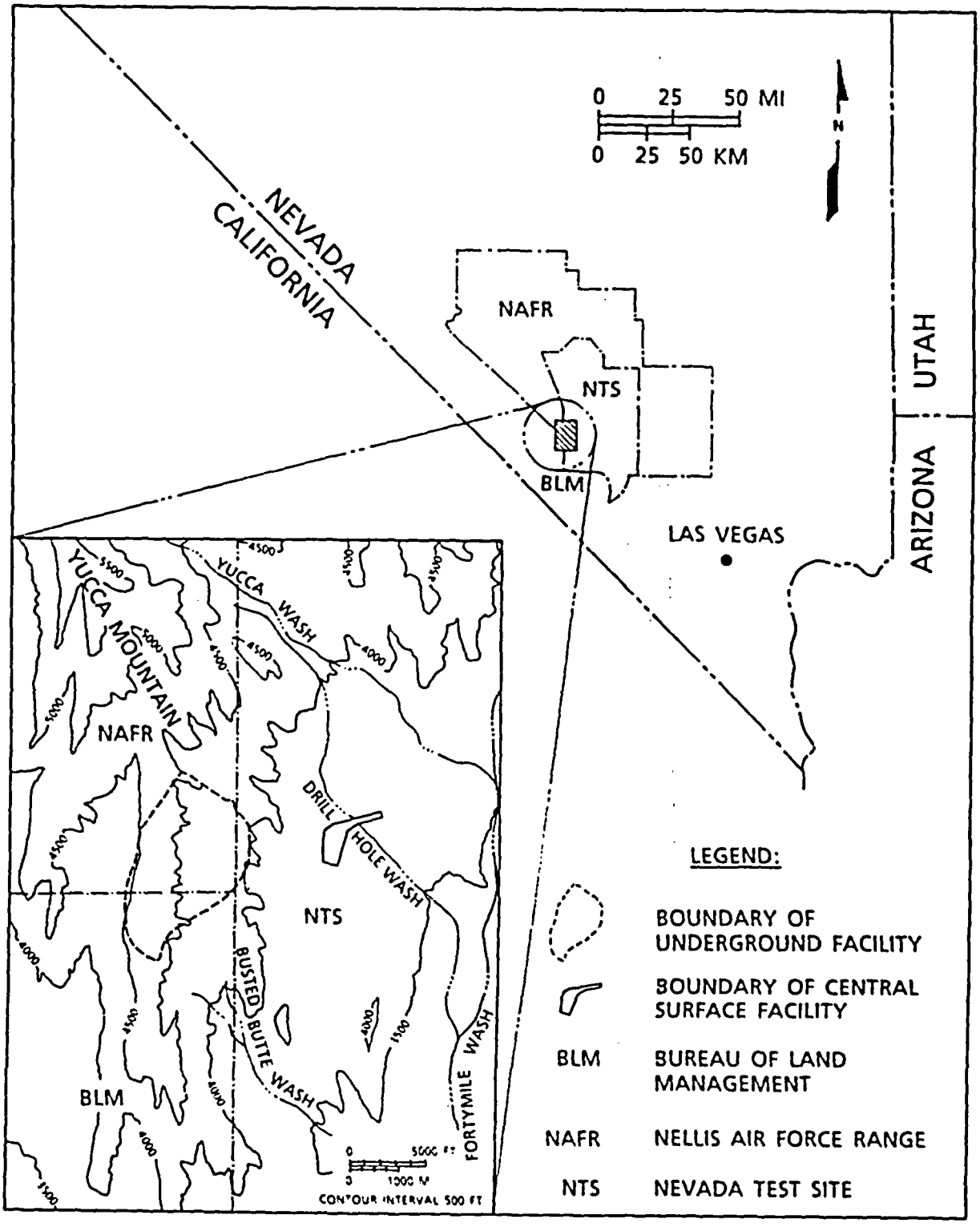


Figure 2-1. Location of the Yucca Mountain Site

Table 2-1. Relation of Stratigraphic and Hydrogeologic Units at Yucca Mountain (Modified From SNL, 1987b, p. 2-25)

STRATIGRAPHIC UNIT		TUFF LITHOLOGY[a]	HYDROGEOLOGIC UNIT[b]	APPROXIMATE THICKNESS (METERS)	COMMENTS
Alluvium		---	Alluvium	0-30	Underlies washes, thin layer on flats
Paintbrush Tuff	Tiva Canyon Member	MD	Tiva Canyon welded unit	0-150	Caprock that dips 5-10° eastward at Yucca Mountain High fracture density
Paintbrush Tuff	Yucca Mountain Member	NP, B	Paintbrush nonwelded unit	20-100	Vitric, nonwelded, porous, poorly indurated, bedded in part. Low fracture density
Paintbrush Tuff	Pah Canyon Member	NP, B	Paintbrush nonwelded unit	20-100	Vitric, nonwelded, porous, poorly indurated, bedded in part. Low fracture density
Paintbrush Tuff	Topopah Spring Member	MD	Topopah Spring welded unit	290-360	Densely to moderately welded. Several lithophysal cavity zones, intensely fractured Central and lower part is potential host rock for repository Bulk hydraulic conductivity in saturated zone east of the site (at Well J-13) about 1.0 m/day
Tuffaceous Beds of Calico Hills		NP, B; Vitric, Devitrified (in part zeolitic)	Calico Hills nonwelded unit; Water table	100-400	Beneath Yucca Mountain, base of units for unsaturated zone determined by water table Calico Hills nonwelded unit is vitric in southwest Yucca Mountain, zeolitic in east and north Zeolitic boundary generally parallels the water table with vitric units above and zeolitic units below a transitional boundary.
Crater Flat Tuff	Prow Pass Member	MD	PP_w		
Crater Flat Tuff	Bullfrog Member	NP, B	PP_n		
Crater Flat Tuff	Bullfrog Member	MD	BF_w		
Crater Flat Tuff	Bullfrog Member	NP, B	BF_n	0-200	
Crater Flat Tuff	Tram Member	NP, B	Crater Flat Member	170-350	None
Lava		Undifferentiated		0-250	Occurs in northeast part of primary area
Lithic Ridge Tuff		Undifferentiated		80-190	None
Older Volcanics		Undifferentiated		---	In USW H-1 hydraulic head about 50 m higher than water table
Pre-Tertiary Rocks		Undifferentiated		---	Occurs 2.5 km east of proposed repository at depth of 1250 m in UE-25p #1, where hydraulic head is about 20 m higher than water table Bulk hydraulic conductivity high, probably due to high fracture density.

(a) NP = nonwelded to partially welded, MD = moderately to densely welded, B = bedded.

(b) PP_w = Prow Pass welded unit, PP_n = Prow Pass nonwelded unit, BF_w = Bullfrog welded unit, BF_n = Bullfrog nonwelded unit.

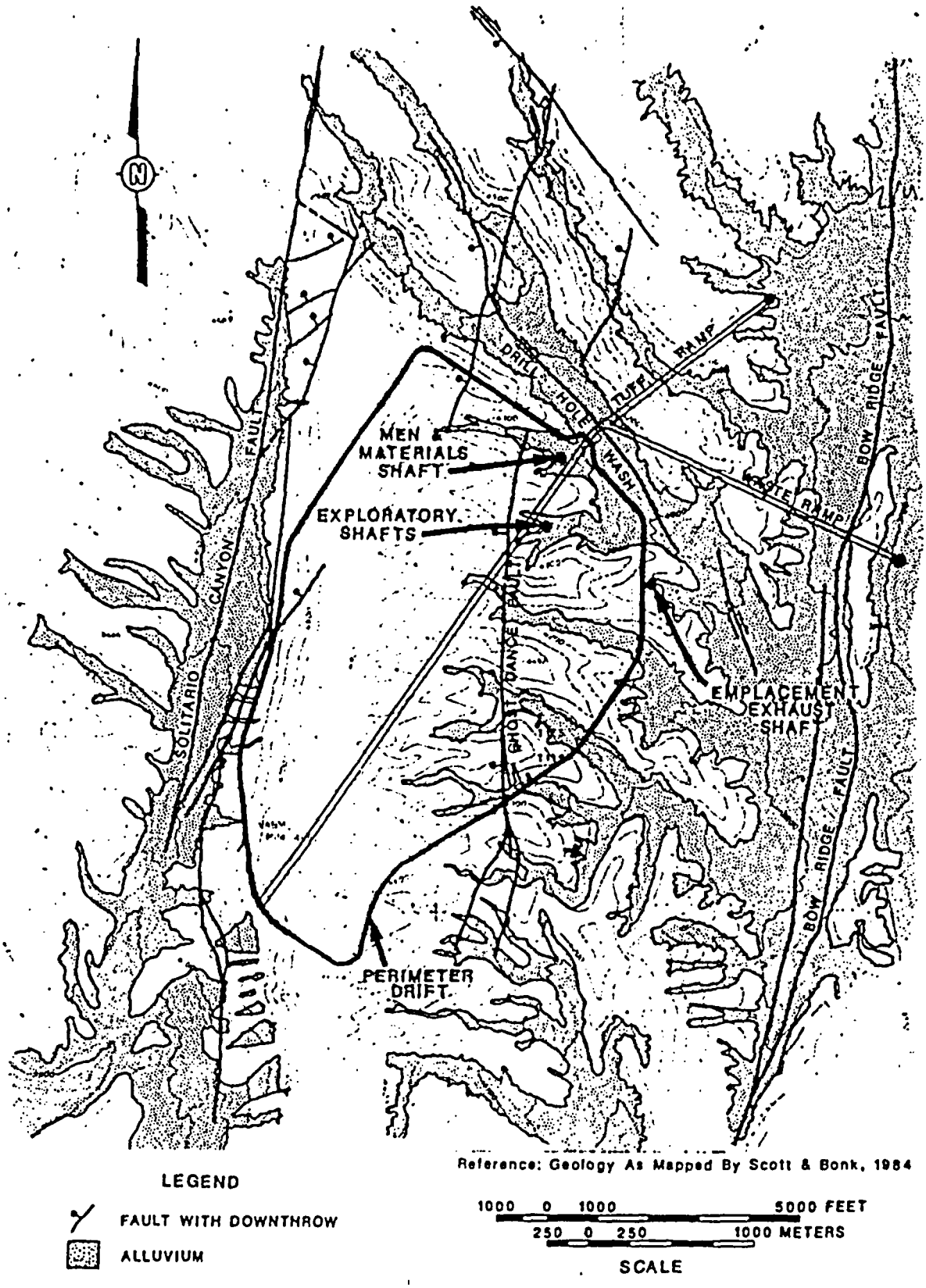


Figure 2-2. Repository Extent, Major Faults and Surficial Geology

Solitario Canyon Fault (or series of faults), which forms the western boundary of the central repository block, and the Bow Ridge Fault, which lies to the east of the repository. Within the central repository block, the most significant (i.e., most laterally continuous) fault is the Ghost Dance Fault (Figure 2-2). This fault dips 84° to 89° to the west and is identified at the surface by small stratigraphic separations and by breccia zones (Scott and Bonk, 1984, Sheet 1). The fault appears to have only a vertical displacement (about 25 m), and it cuts across the east-west-trending washes that drain off Yucca Crest (DOE, 1986, p. 3-15). Thus, it has little or no topographic expression across the block. The nature of this fault (vertical continuity, degree of fracturing, etc.) at depth is uncertain, but the fault is potentially important because it is the only continuous, high-angle, north-south fault known to lie substantially within the proposed repository boundary (Figure 2-2). Other minor faults may occur close to the edge of the repository workings or may be intersected by the ramps used to gain access to the repository (described in Section 2.2).

Several northwest-striking faults lie northeast of the proposed facility and underlie washes in this area, including Drill Hole Wash along the northeast boundary of the repository. These faults appear to have little vertical separation and probably have a major strike-slip component (Scott et al., 1984, p. 18). Although these faults are outside the repository boundary, the proposed alignment of the Tuff Ramp, Waste Ramp, and the main drifts are intersected by a fault in Drill Hole Wash (see Figure 2-2).

In addition to the faults described above and shown in Figure 2-2, many relatively short faults or zones of closely-spaced fractures have been mapped at the surface within the repository boundary. In general, these faults trend north, northwest, or northeast (Scott and Bonk, 1984, Sheet 1). Many of these features may have little displacement and they may not be continuous through the various stratigraphic units to the repository horizon.

In general, the tuff beds strike north-to-northeast and dip at a shallow angle (5° to 8°) to the east (Scott and Bonk, 1984, Sheet 1). The alluvium occurs mostly in the washes and underlying the flats to the east of the repository (Figure 2-2). The maximum thickness of alluvium recorded thus far in drill holes is 45 m in drillhole UE-25a#7, which is located east of the Exploratory Shaft (ES-1) and in the middle of Drill Hole Wash (Fernandez and Freshley, 1984, p. 61).

2.2 HYDROLOGY

2.2.1 Surface-Water Hydrology

The Yucca Mountain site lies in a semiarid region in the rainshadow of the Sierra Nevadas to the west. Annual, average precipitation is 150 mm (Montazer and Wilson, 1984, p. 37) and potential evapotranspiration is approximately 630 mm/year (DOE, 1986, p. 6-140). As in any arid region, rainfall may be highly

variable from year to year. For example, at Las Vegas, where average annual precipitation is 96 mm (3.76 in.), levels have varied from 19 mm to 270 mm (0.76 in. to 10.7 in.) (Eglinton and Dreicer, 1984, p. 2-1). Moreover, much of the rainfall occurs in short-duration, intense storms.

There are no perennial streams at the Yucca Mountain site and vicinity. Major storms, however, may result in significant stream flows and local flooding of washes. Figure 2-3, based on the map of Squires and Young (1984), shows the potential for flooding along washes near the proposed repository shaft and ramp portal locations for 100-year, 500-year, and regional maximum floods. The regional maximum flood in this case is estimated by Crippen and Bue (1977, p. 15) based on major flood events in the region. It does not constitute a rigorous determination of the probable maximum flood (ANSI, 1981). Calculations of the maximum flood flows for each of these access locations are given in Appendix A.

2.2.2 Ground-Water Hydrology

The proposed repository in the Topopah Spring welded unit lies in the unsaturated zone 200 to 400 m above the ground-water table (Figure 2-4a). The water table is essentially horizontal and occurs predominantly within the Crater Flats Tuff below the repository. Figures 2-4b, 2-4c, and 2-4d, respectively, show the thicknesses of Topopah Spring welded tuff and Calico Hills nonwelded tuff (vitric and zeolitic) between the repository and the water table.

Table 2-2 summarizes the hydrogeologic characteristics of some of the hydrogeologic units defined in Table 2-1. Saturated, rock mass, hydraulic conductivities are in the range 10^{-2} to 10^{-6} cm/s for all units, but there is a significant contrast between the properties of welded and nonwelded units. Welded units typically have low matrix conductivities but high conductivity through closely spaced fractures. Nonwelded units have fewer fractures but higher conductivity through the matrix.

The available information regarding the spatial extent and hydrologic properties of hydrostratigraphic units at Yucca Mountain are sufficient to allow preliminary definitions of likely ground-water flow paths under isothermal conditions (Montazer and Wilson, 1984). Some uncertainty exists, however, regarding ground-water flow through faults and flow of water or water vapor under nonisothermal conditions.

The generally arid conditions and the occurrence of most of the precipitation in short-duration, intense events ensure that relatively little water is available for deep recharge. Recharge may be impeded by permeability contrasts and capillary barriers between the welded and nonwelded units (Scott et al., 1983, p. 325; Montazer and Wilson, 1984, p. 26-31). The fractured, welded Tiva Canyon is highly permeable when saturated (Table 2-2), and may allow rapid infiltration through open fractures if saturation could be maintained during heavy rainfall. Water could possibly percolate rapidly down to

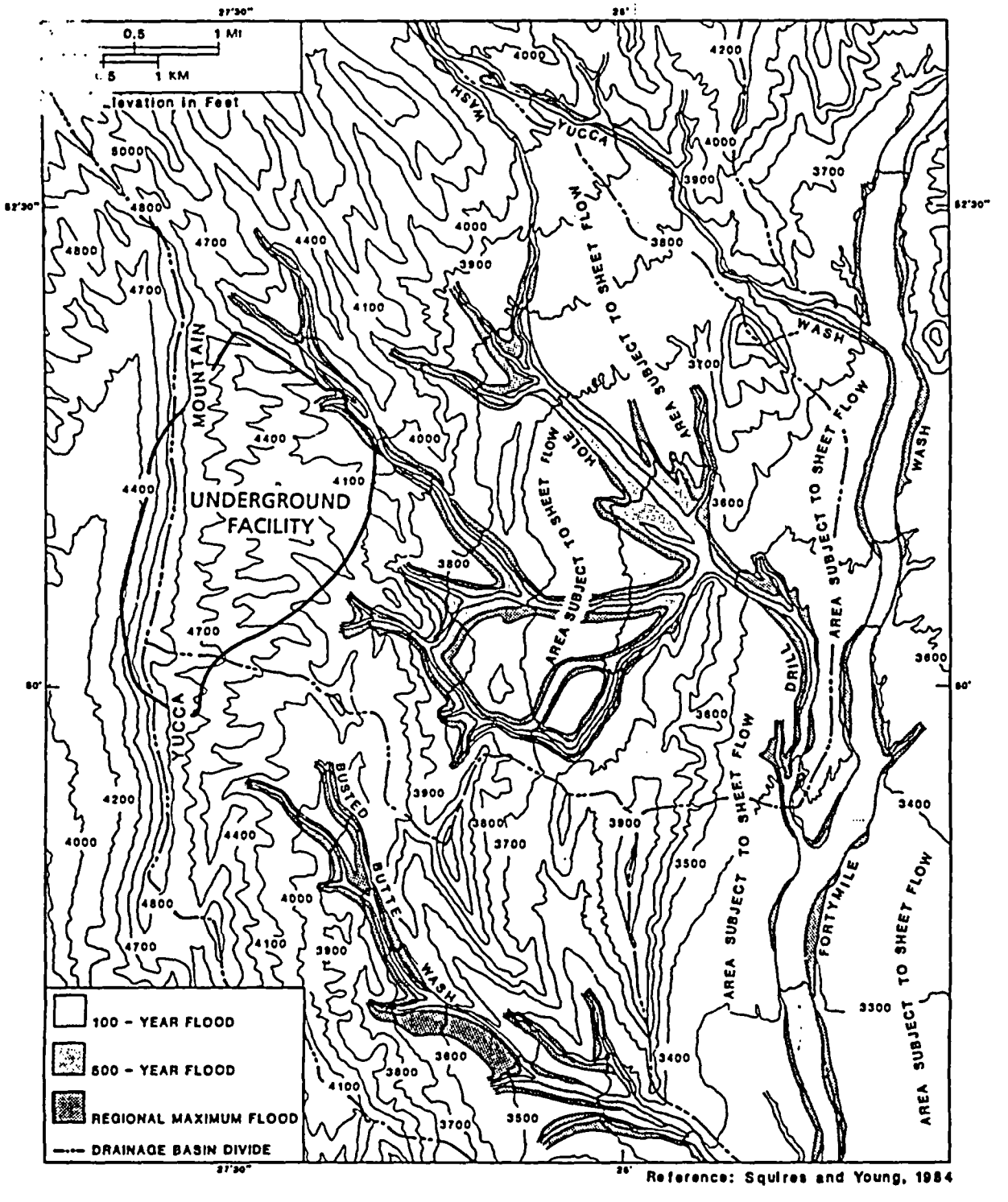
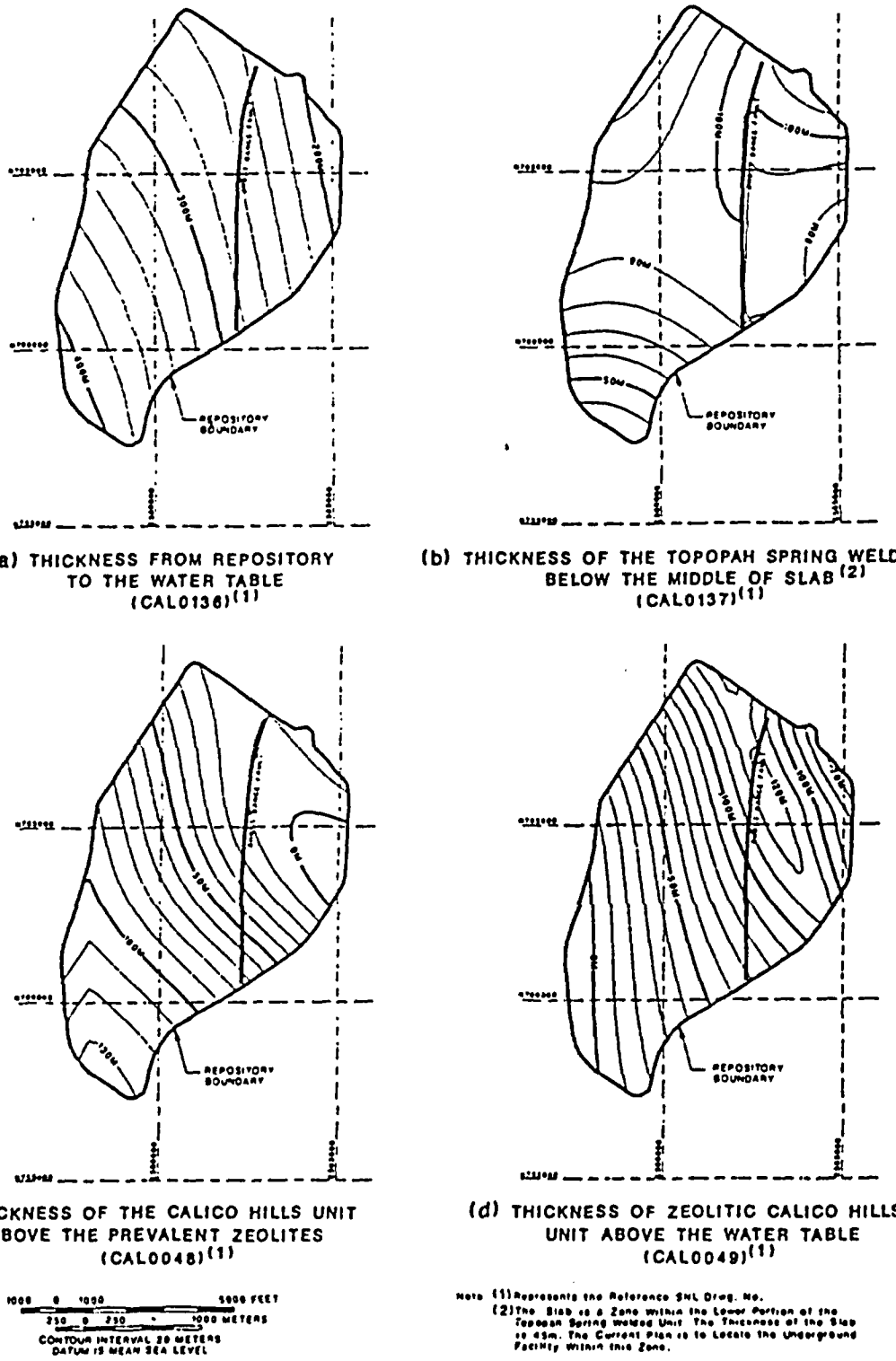


Figure 2-3. Flood Prone Area, Yucca Mountain, Southern Nevada



(a) THICKNESS FROM REPOSITORY TO THE WATER TABLE (CAL0136)[1]

(b) THICKNESS OF THE TOPOPAH SPRING WELDED BELOW THE MIDDLE OF SLAB[2] (CAL0137)[1]

(c) THICKNESS OF THE CALICO HILLS UNIT ABOVE THE PREVALENT ZEOLITES (CAL0048)[1]

(d) THICKNESS OF ZEOLITIC CALICO HILLS UNIT ABOVE THE WATER TABLE (CAL0049)[1]

Note (1) Represents the Reference SNL Dwg. No.
(2) The Slab is a Zone within the Lower Portion of the Topopah Spring Welded Unit. The Thickness of the Slab is 45m. The Current Plan is to Locate the Underground Facility Within this Zone.

Figure 2-4. Contours of the Thickness of Unsaturated Rock Beneath the Proposed Repository Area

Table 2-2. Hydrogeologic Classification and Hydrologic Properties of Stratigraphic Units at Yucca Mountain

Hydrogeologic Unit	Characteristics[a,c]	Matrix Properties: Bulk Porosity,[b] %	Matrix Properties: Average, Saturated Hydraulic Conductivity, cm/s	Rock Mass Properties: Fracture Density,[b] per m^3	Rock Mass Properties: Saturated Hydraulic Conductivity, cm/s
Alluvium	Underlies washes; thin layer on flats		Generally high [a]		Generally high [a]
Tiva Canyon welded unit	Densely welded, high fracture density	10 ±5.3	2.5×10^{-9} [b]	20	1.2×10^{-3} [b]
Paintbrush[d] nonwelded unit	Nonwelded, argillic and vitric, porous, poorly indurated, bedded in part, low fracture density	45 ±11.8	2.4×10^{-6} [b]	10	2.4×10^{-4} [b] 10^{-3} (vitric) [c] 10^{-4} to 10^{-6} (argillic) [c]
Topopah Spring welded unit	Densely welded, high fracture density, several lithophysal (cavity) zones	15 ±5.1 10.6 ±4.6[e]	3.5×10^{-9} [b] 2.3×10^{-9} [e]	40	1.2×10^{-3} [b] 10^{-2} to 10^{-5} [c]
Calico Hills nonwelded unit	Nonwelded, vitric	39 ±7.7 32.4 ±8.8[e]	1.3×10^{-6} [b] 3.4×10^{-7} [e]	5	2.4×10^{-4} [b] 10^{-3} [c]
Calico Hills nonwelded unit	Nonwelded, zeolitic	30 ±8.6 2.69 ±4.7[e]	1.0×10^{-8} [a] 4.2×10^{-9} [b] 1.7×10^{-9} [e]	5	2.4×10^{-4} [b]
Older tuffs	Variable	23 ±?	1.1×10^{-7} [b]	5 to 20	2.4×10^{-4} to 1.2×10^{-3} [b]

[a] DOE, 1986, p. 3-29. Saturated hydraulic conductivity of Tiva Canyon assumed equal to Topopah Spring. Saturated hydraulic conductivity of Paintbrush assumed equal to Calico Hills.
[b] Sinnock et al., 1984, pp. 11-12.
[c] Scott et al., 1983, p. 299.
[d] Includes Pah Canyon and Yucca Mountain Members.
[e] From the Reference Information Base (RIB) (Zeuch and Eatough, 1986).

the bottom of the welded unit, but it would be impeded from infiltrating into the underlying nonwelded Pah Canyon Member of the Paintbrush Tuff by the contrast in permeability, especially if the nonwelded unit were clay-rich (Scott et al., 1983, p. 325-327), or by air entrapment (Montazer and Wilson, 1984, p. 47). According to this hypothesis, water infiltrating from the surface could flow downdip (generally to the east) along the upper contact of the nonwelded Paintbrush Tuff, which is located above the proposed location of the underground facility. A second capillary barrier may occur at the base of the nonwelded member of the Paintbrush Tuff at the contact with the fractured Topopah Spring Member. In this case, downward flow may be impeded because the water-filled pores in the nonwelded unit are smaller than the apertures of the fractures in the welded unit (Montazer and Wilson, 1984, p. 31).

Because of the combined effects of low average rainfall and permeability and capillary barriers, the flux through most of the Topopah Spring Member is probably restricted to about 0.1 mm/year (Sinnock et al., 1984, p. 52). This would represent less than one percent of the estimated annual rainfall, the balance occurring as evapotranspiration, flow in the washes, or lateral, subsurface flow in the units overlying the Topopah Spring. Higher infiltration rates could occur through fault or fracture zones. These zones could have high vertical conductivity in both the welded and nonwelded units (Scott et al., 1983, p. 328), although it is possible that such zones are less conductive in the more ductile nonwelded units than in welded units (Montazer and Wilson, 1984, p. 49). If the faults cut across stratigraphic boundaries and are highly conductive throughout, they may provide flow paths through the permeability and capillary barriers. It is estimated that such zones could be recharged directly from the surface or by subsurface water perched above a capillary or permeability barrier at a maximum rate of 0.5 mm/year (Sinnock et al., 1984, p. 16).

As noted in Section 2.1, several, major, surface faults occur only at the edges of the proposed repository block and are less likely to intersect underground drifts. However, faults lying north and east of the proposed repository boundary, including one fault underlying Drill Hole Wash, are more likely to intersect the repository ramps. Other major faults could also be encountered at the edges of the repository workings that intersect drifts. As discussed subsequently in Section 4.2.2.1, a certain assumption is made of fault occurrences at depth (including the Ghost Dance Fault) that on balance is considered reasonable.

2.3 REFERENCE WASTE DESCRIPTIONS

Information on the type of nuclear waste and the number of waste packages to be emplaced in the repository is presented in this section. This information will be used to place in perspective the areal relationships between the waste packages and the excavated area and the number of waste packages in specified areas of interest.

As stated in the Nuclear Waste Policy Act of 1982, the quantity of spent fuel in the first repository shall be limited to "70,000 metric tons of heavy metal or a quantity of solidified high-level radioactive waste resulting from the reprocessing of such a quantity of spent fuel . . ." (NWPA 1983, p. 2215). The first repository will be designed to accept West Valley high-level waste (WVHLW) and spent fuel. The repository shall also be capable of accepting defense high-level waste (DHLW) (NWPA 1983, pp. 2205-2206). These restrictions provide the basis for the type of waste to be disposed of in the repository.

As illustrated below, some variation of type of nuclear waste and number of waste packages is anticipated. As defined by O'Brien (1985, pp. 2-4), the total amount of WVHLW and DHLW to be disposed of in the first repository could be 650 metric tons of uranium (MTU) and 10,000 MTU, respectively. If the first repository were to dispose of all of the WVHLW and the DHLW, the amount of spent fuel would be no more than 59,350 metric tons of heavy metal (MTHM). More current evaluations into the monitored retrievable storage (MRS) facility have assumed that 62,000 MTHM would be spent fuel while 8,000 MTHM would be DHLW (SNL, 1987a, p. B-45, B-46). The reference waste description used in this report is a mixture of 650 MTU of WVHLW, 7,350 MTU of DHLW, and 62,000 MTU of spent fuel (SNL, 1987b, Appendix P, p. 3-8). Using the assumptions below, the number of canisters to be disposed of in the repository is estimated to be about 42,200.

- Each pressurized-water reactor (PWR) assembly contains 0.4614 MTU and each boiling-water reactor (BWR) assembly contains 0.1833 MTU (O'Brien, 1985, p. 5).

- 60% of spent fuel is in PWR fuel assemblies and 40% is in BWR fuel assemblies (SNL, 1987b, Appendix P, p. 3-2).

- 90% of the fuel assemblies are consolidated while 10% are not consolidated (SNL, 1987a, p. 2-7).

- Each waste disposal canister is 66 cm in diameter and can hold three intact or six consolidated PWR assemblies or six intact or 12* consolidated BWR assemblies (O'Brien, 1985, p. 4).

- Each waste disposal canister of DHLW contains 0.5 MTU (O'Brien, 1984, p. 4).

- Number of canisters containing WVHLW is 310 (SNL, 1987b, Appendix P, p. 3-18).

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*In the SCP-CDR (SNL, 1987b) the number of BWR assemblies that can be consolidated into a canister has been modified from 12 to 18. This change reduces the number of canisters in this report. If it is assumed that the canisters are distributed uniformly in the underground facility, there is no impact of this change on any results in this report.

2.4 UNDERGROUND REPOSITORY DESIGN

The current, underground repository design consists of emplacement drifts accessed by nonemplacement drifts, which are in turn accessed by two ramps, and four shafts. In assessing the need for sealing (Chapter 5), it is necessary to estimate surface flows through shafts, ramps, and faults intersecting underground drifts which are dependent in part on repository geometry. In order to assess the potential for inflows to contact waste for the repository as a whole, consideration is given to the physical layout (size, orientation, and grade of drifts) at the lowest portion of the repository. Furthermore, the design options proposed in Chapter 6 require information on the underground repository design.

The current, underground repository designs have been prepared by Parsons Brinckerhoff Quade & Douglas (SNL, 1987b) and are subject to revision as site development proceeds. The repository is designed on one inclined level at about 305-m depth at the exploratory shafts. The repository drifts slope to the southeast or northeast following the dip of the strata (Figure 2-5).

Two alternative emplacement modes are being considered. In the vertical emplacement configuration (Figure 2-6) which is the reference construction in the Site Characterization Plan-Conceptual Design Report (SCP-CDR), waste containers are placed in boreholes drilled in the floors of long emplacement drifts, one container per hole. In the horizontal emplacement configuration (Figure 2-7), multiple waste containers are placed in long horizontal boreholes drilled from access drifts. The general layout is the same for both cases, with the repository divided into about 18 separate panels. Typical panel layouts for vertical and horizontal emplacement configurations are illustrated in Figures 2-8 and 2-9, respectively. Access to the panels is provided by three main entry drifts (the waste main, the tuff main, and the service main), which connect to two ramps (the waste ramp and the tuff ramp) in the northeastern part of the repository, and the repository is bounded by a continuous perimeter drift.

Shaft access is provided through the Exploratory Shafts (ES-1 and ES-2), the Men and Materials Shaft (MMS), and the Emplacement Exhaust Shaft (EES). Within the repository, there are two support shop areas in which no waste will be emplaced; development support shops are located near the MMS and the waste emplacement support shops are located near ES-1 and ES-2.

The main entry drifts in the repository will grade downward from southwest to northeast, and the panel and midpanel access drifts grade downward from northwest to southeast. The EES (Figure 2-5) and the connecting drifts from this shaft to the perimeter drift is the lowest point in the repository. The maximum grade for drifts is about eight percent.

Currently, it is planned that one of the main entry drifts, the two ramps, and the perimeter drift will be excavated by tunnel boring machines (TBM), while

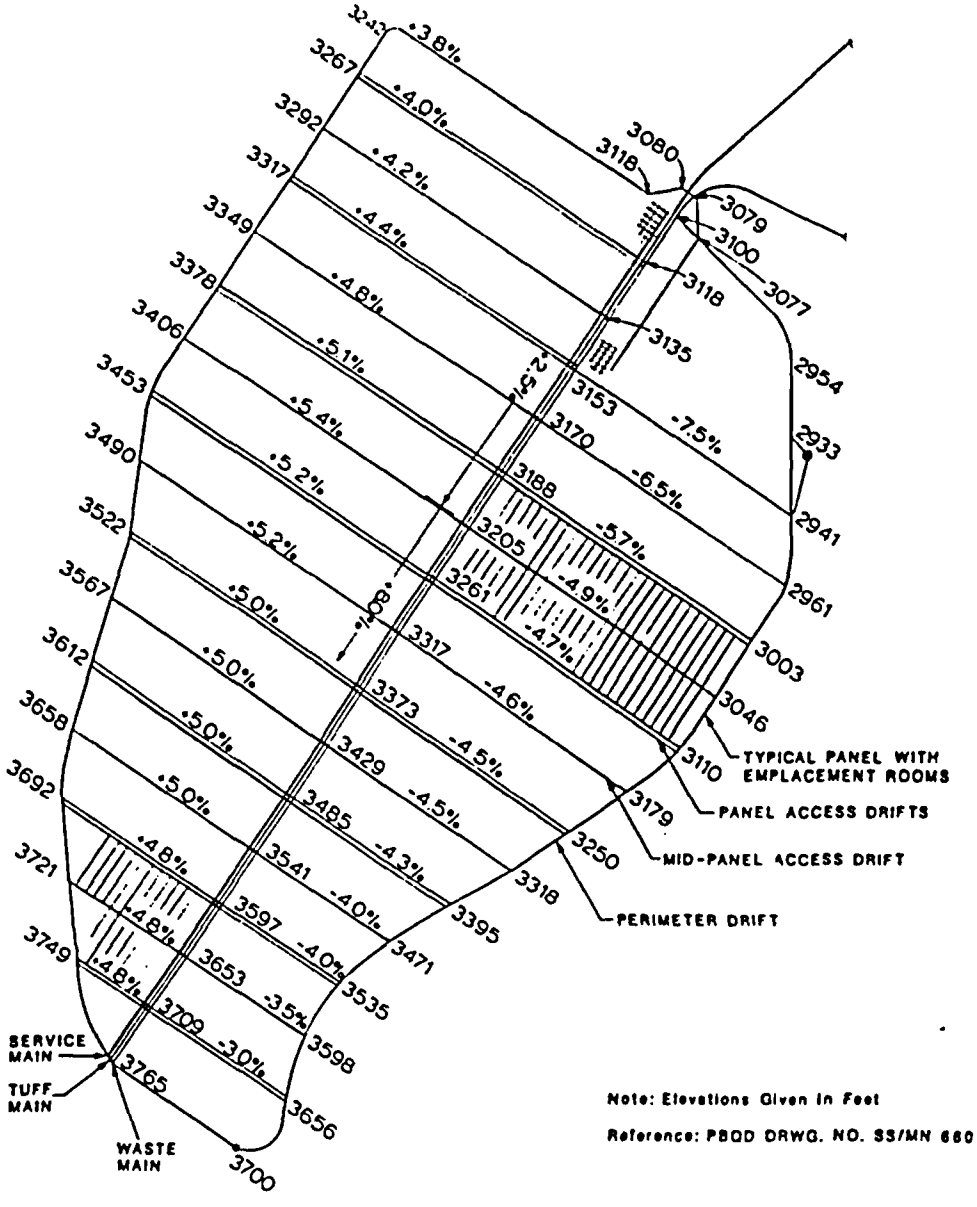


Figure 2-5. General Layout of a Nuclear Waste Repository in Tuff - Vertical Emplacement

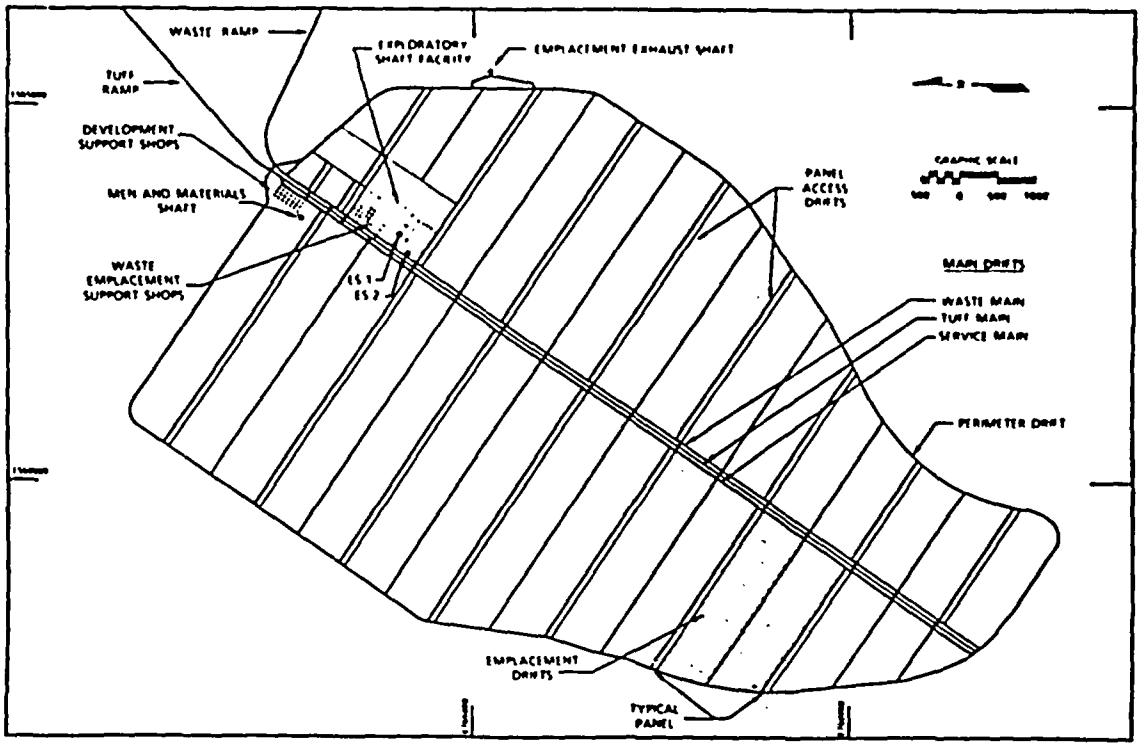


Figure 2-6. Subsurface Facility Design for a Nuclear Waste Repository in Tuff - Vertical Emplacement

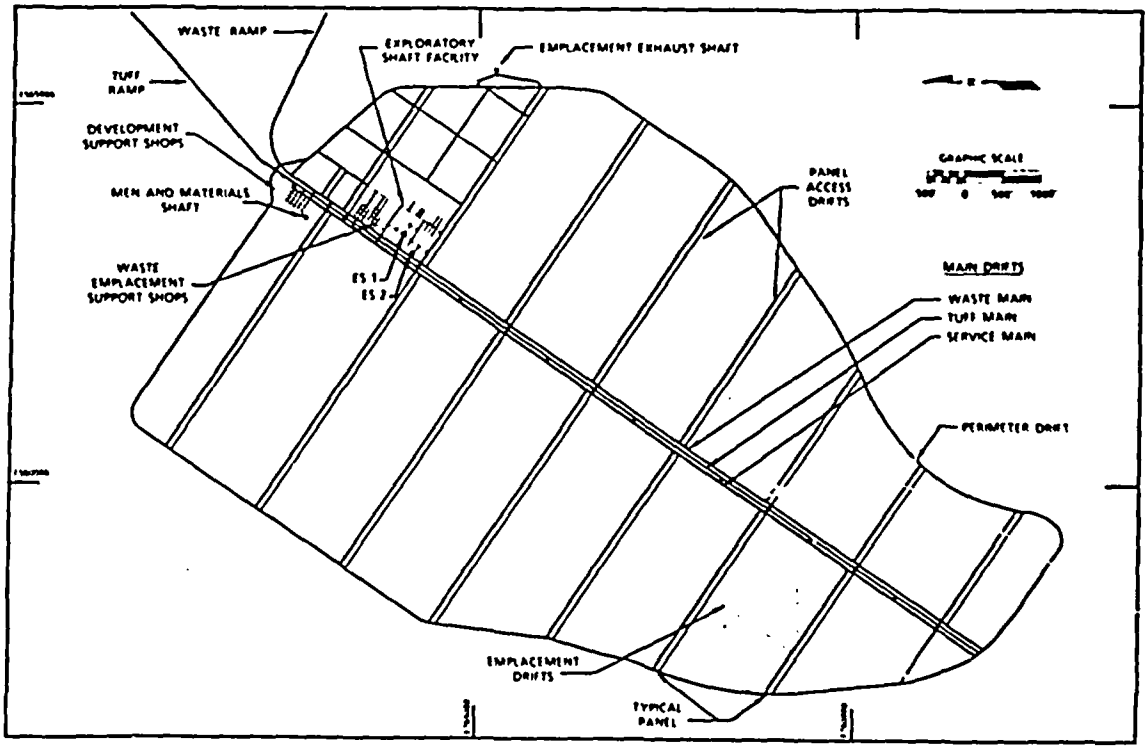


Figure 2-7. Subsurface Facility Design for a Nuclear Waste Repository in Tuff - Horizontal Emplacement

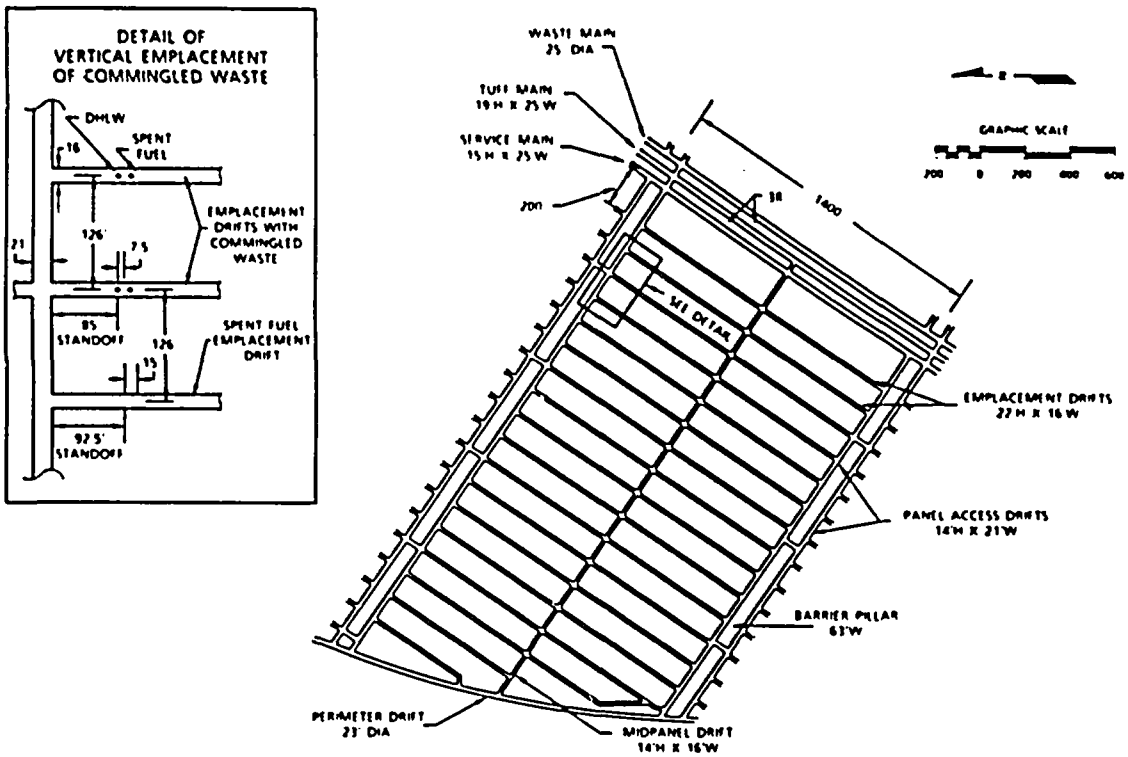


Figure 2-8. Typical Panel Layout - Vertical Emplacement

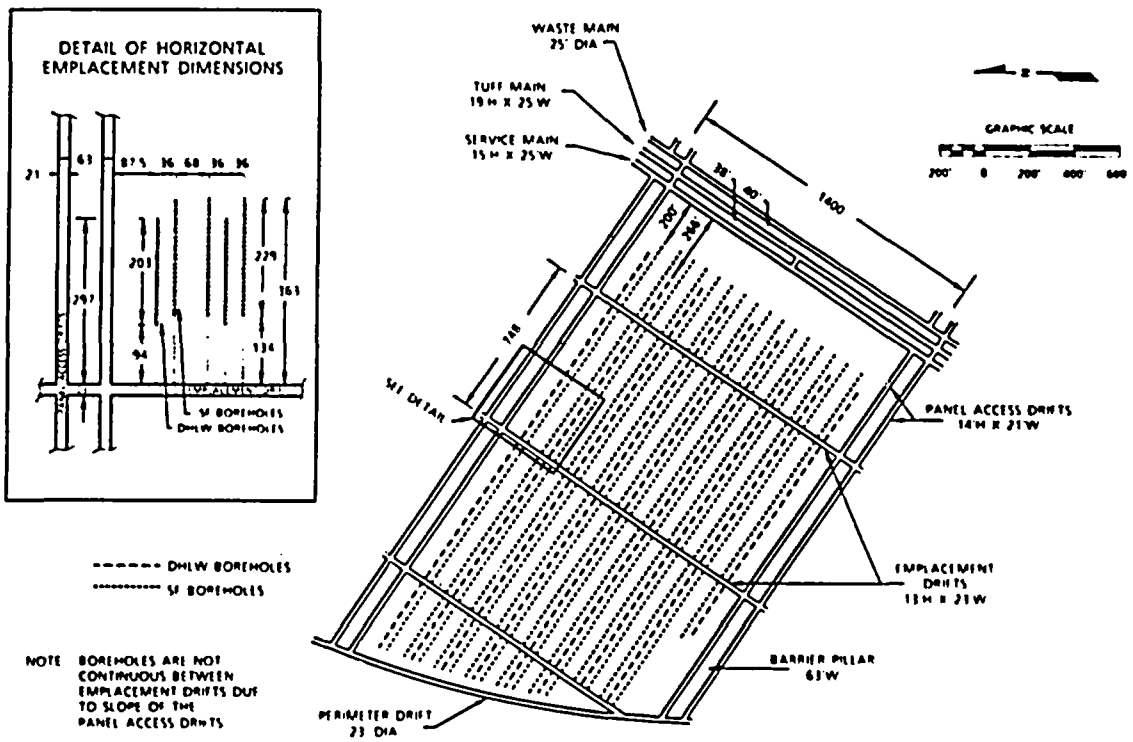


Figure 2-9. Typical Panel Layout - Horizontal Emplacement

the remainder of the drifts will be excavated by drill-and-blast methods. The possibility of excavating all of the drifts using a TBM is being evaluated. Ramps and drifts excavated by TBM will have a circular cross-section, although a roadway may be constructed in the invert. Drifts excavated by drill-and-blast will have vertical walls and arched roofs. Rock support will consist of rockbolts and steel mesh for the conditions expected in most of the drifts. Heavier support such as reinforced shotcrete may be used in areas of poorer rock quality, and steel sets might be used in selected areas.

The repository design incorporates six openings to the subsurface facilities, including four vertical shafts and two inclined ramps. The locations of the shafts and ramps relative to the subsurface workings, faults, and surface areas liable to flooding are shown in Figures 2-2 and 2-3. Dimensions, grades, total depths, excavation methods, and linings are summarized in Table 2-3. Figure 2-10 shows profiles of the shafts superimposed on the stratigraphy at each location. The MMS and EES have shallow sumps extending 24 m and 3 m below the repository level, respectively. In contrast, the sump for the ES-1 is 140 m below the repository level. The ramps connect directly into the main access drifts at the northern end of the repository (Figures 2-6 and 2-7).

Dimensions of all the underground excavations and the floor area and excavated volume are given in Tables 2-4 and 2-5 for the vertical and horizontal emplacement configurations, respectively. The following assumptions are made concerning the types and amounts of waste to be disposed of in the repository in order to arrive at the estimated total length of each design feature in the underground excavation: 650 MTHM of WVHLW, 7,350 MTHM of DHLW and 62,000 MTHM of spent fuel. Should the types or amounts of waste change, the linear footage associated with the emplacement, midpanel, and access drifts will also change. The numbers given in both tables, therefore, represent approximations of the floor areas and volumes excavated for the reference repository design assumed in this report.

Table 2-6 contains summary values that are reduced from values in Tables 2-4 and 2-5. As might be expected, the linear footage associated with the horizontal emplacement configuration is considerably less than the footage for vertical emplacement configuration. Because of this difference, the overall extraction ratio is different, as indicated in Table 2-6, i.e., 18% for vertical emplacement and 9% for horizontal emplacement. For vertical emplacement, the ratio of the area of the emplacement drifts to the floor area for all drifts indicates that there are no wastes disposed of in approximately 39% of the drifts. For horizontal emplacement, about 67% of the drifts are classified as nonemplacement drifts. In the horizontal emplacement configuration, no waste is disposed of in the floors of any of the drifts.

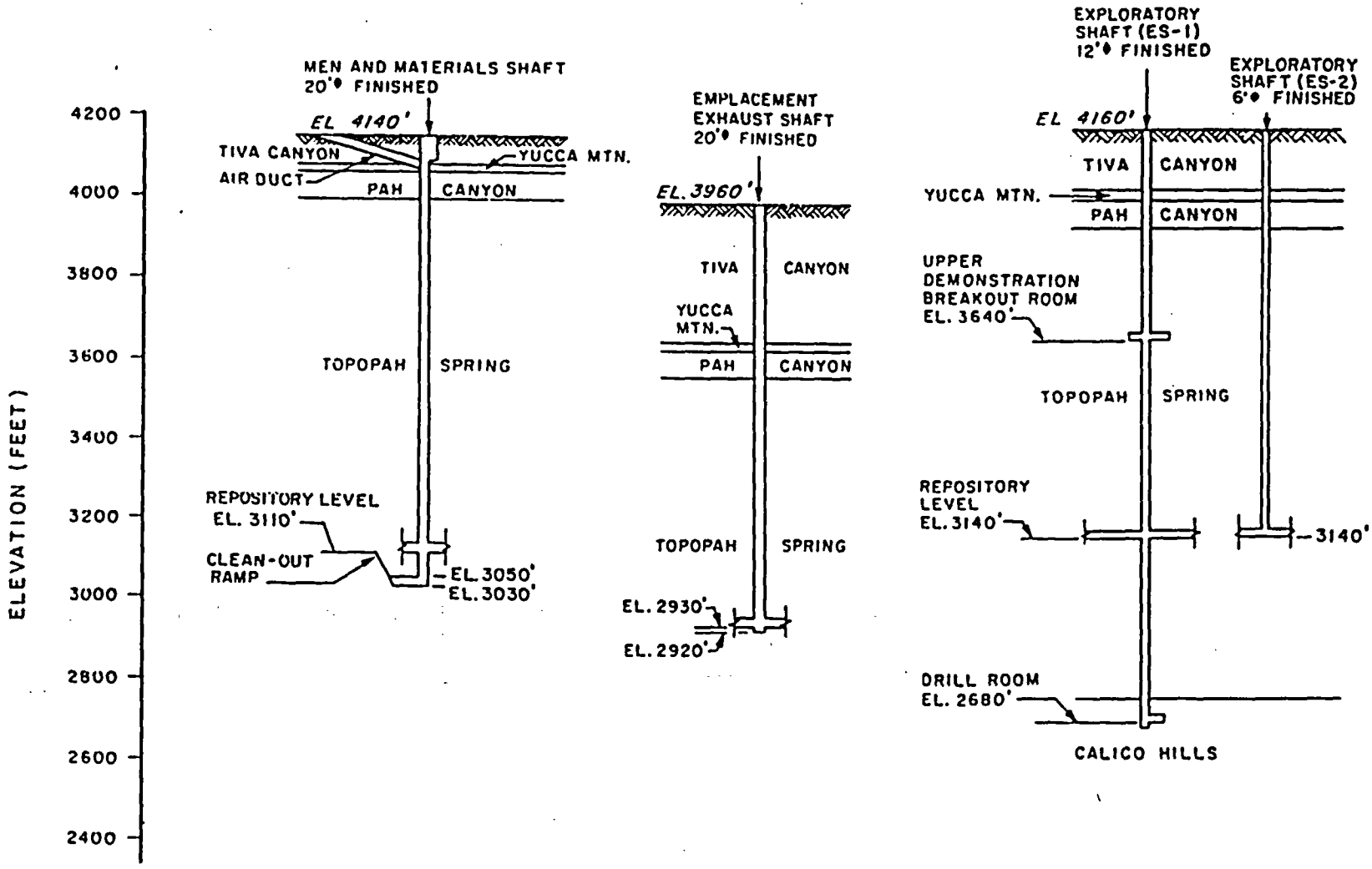
Table 2-3. Summary of Construction Parameters for Shafts and Ramps

Designation	Grade	Excavated Diameter, m (ft)	Finished Diameter, m (ft)	Total Depth,[a] m (ft)	Length, m (ft)	Excavation Method	Lining
Exploratory Shaft (ES-1)	Vertical	4.42 (14.50)	3.66 (12.0)	454 (1490)	-	Drill and blast	31 cm (12-in.) concrete
Exploratory Shaft (ES-2)	Vertical	2.44 (8.0)	1.83 (6.0)	314 (1030)	-	Raise bore	Steel
Men and Materials Shaft (MMS)	Vertical	6.86 (22.50)	6.10 (20.0)	338 (1110)	-	Drill and blast	38 cm (15 in.) concrete
Emplacement Exhaust Shaft (EES)	Vertical	6.86 (22.50)	6.10 (20.0)	317 (1040)	-	Drill and blast	38 cm (15 in.) concrete
Tuff Ramp							
(Vertical)	17.9%	7.62 (25.0)	7.32 (24.0)	-	1410 (4627)	TBM	Rock bolts, wire mesh, shotcrete
(Horizontal)	17.9%	6.40 (21.0)	6.10 (20.0)		1410 (4627)		
Waste Ramp							
(Vertical)	8.9%	7.01 (23.0)	6.40 (21.0)	-	2012 (6603)	TBM	Rock bolts, wire mesh, shotcrete
(Horizontal)	8.9%	6.40 (21.0)	5.79 (19.0)		2012 (6603)		

[a] Includes sumps of 3.05 m (10 ft) for EES and 24 m (80 ft) for MMS.

TBM - Tunnel boring machine
Vertical - Vertical emplacement
Horizontal - Horizontal emplacement

Reference: SNL Drawing Numbers R06916, R07015, R07001, R06886, and R06887



REFERENCES: SNL DRWG. NO. RD7001
GEOLOGY ESTIMATED FROM BENTLEY (1984) AND
SCOTT AND BONK (1984)

Figure 2-10. Elevations of Shafts and Repository in Relation to Stratigraphy

Table 2-4. Dimensions, Areas, and Volumes for the Underground Excavations - Vertical Emplacement

Underground Excavation	Estimated Total Length, m (ft)	Width or Diameter, m (ft)	Height, m (ft)	Floor Area, m^2 (ft^2)[a]	Excavated Volume, m^3 (ft^3)[a]
Waste ramp	2,013 (6,603)	7.0 (23)[b]	--	14,110 (151,900)[c]	77,880 (2,751,000)
Tuff ramp	1,410 (4,627)	7.6 (25)[b]	--	10,750 (115,700)[c]	64,340 (2,272,000)
Waste main	3,420 (11,220)	7.6 (25)[b]	--	26,060 (280,500)[c]	156,000 (5,510,000)
Tuff main	3,420 (11,220)	7.6 (25)	5.8 (19)	26,060 (280,500)	135,400 (4,780,000)
Service main	3,298 (10,820)	7.6 (25)	4.6 (15)	25,130 (270,500)	99,870 (3,528,000)
Emplacement drifts	122,400 (401,500)[c]	4.9 (16)	6.7 (22)	596,700 (6,424,000)	3,779,000 (133,500,000)
Access drifts	26,010 (85,350)[c]	6.4 (21)	4.3 (14)	166,500 (1,792,000)	626,600 (22,130,000)

[a] Areas and volumes rounded to four significant digits.
[b] Circular openings
[c] Horizontal distances

Table 2-4. Dimensions, Areas, and Volumes for the Underground Excavations - Vertical Emplacement (Concluded)

Underground Excavation	Estimated Total Length, m (ft)	Width or Diameter, m (ft)	Height, m (ft)	Floor Area, m^2 (ft^2)[a]	Excavated Volume, m^3 (ft^3)[a]
Midpanel drifts	11,500 (37,750)[c]	4.9 (16)	4.3 (14)	56,110 (604,000)	218,200 (7,707,000)
Ventilation exhaust drift	1,143 (3,750)[c]	7.6 (25)	5.8 (19)	7,204 (77,550)	30,980 (1,094,000)
Perimeter drifts	9,448 (31,000)[c]	7.0 (23)[b]	--	66,240 (713,000)[c]	365,600 (12,910,000)
Development shops	1,118 (3,670)[c]	7.6 (25)	5.8 (19)	7,510 (80,850)	28,710 (1,014,000)
Emplacement shops	687 (2,255)[c]	7.6 (25)	5.8 (19)	4,466 (48,080)	17,410 (614,800)
Training and performance confirmation	305 (1,000)[c]	6.4 (21)	4.3 (14)	1,951 (21,000)	7,339 (259,200)

[a] Areas and volumes rounded to four significant digits.
[b] Circular openings
[c] Horizontal distances

Table 2-5. Dimensions, Areas, and Volumes for the Underground Excavations - Horizontal Emplacement

Underground Excavation	Estimated Total Length, m (ft)	Width or Diameter, m (ft)	Height, m (ft)	Floor Area, m^2 (ft^2)[a]	Excavated Volume, m^3 (ft^3)[a]
Waste Ramp	2,012 (6,603)	6.4 (21)[b]	--	12,880 (138,700)	64,700 (2,285,000)
Tuff Ramp	1,410 (4,627)	6.4 (21)[b]	--	9,027 (97,170)	45,330 (1,601,000)
Waste Main	3,420 (11,220)	6.4 (21)[b]	--	21,890 (235,600)	109,900 (3,882,000)
Tuff Main	3,420 (11,220)	7.6 (25)	5.8 (19)	26,060 (280,500)	135,300 (4,780,000)
Service Main	3,298 (10,820)	7.6 (25)	4.6 (15)	25,130 (270,500)	99,870 (3,528,000)
Emplacement Drifts	22,938 (75,260)[c]	7.0 (23)	4.0 (13)	160,800 (1,731,000)	547,700 (19,340,000)
Access Drifts	26,010 (85,350)[c]	6.4 (21)	4.3 (14)	166,500 (1,792,000)	626,600 (22,130,000)

[a] All areas and volumes rounded to four significant digits.
[b] Circular openings.
[c] Projected floor area.

Table 2-5. Dimensions, Areas, and Volumes for the Underground Excavations - Horizontal Emplacement (Concluded)

Underground Excavation	Estimated Total Length, m (ft)	Width or Diameter, m (ft)	Height, m (ft)	Floor Area, m^2 (ft^2)[a]	Excavated Volume, m^3 (ft^3)[a]
Midpanel Drifts	NA	--	--	--	--
Ventilation Exhaust Drifts	1,143 (3,750)[c]	7.6 (25)	5.8 (19)	7,204 (77,550)	30,980 (1,094,000)
Perimeter Drift	9,449 (31,000)[c]	7.0 (23)	--	66,233 (713,000)	365,200 (12,900,000)
Development Shops	843 (2,765)[c]	7.6 (25)	5.8 (19)	5,651 (61,830)	22,080 (780,000)
Emplacement Shops	687 (2,255)[c]	7.6 (25)	5.8 (19)	4,466 (48,080)	17,400 (614,800)
Training and Performance Confirmation	305 (1,000)[c]	6.4 (21)	4.3 (14)	1,951 (21,000)	7,339 (259,200)

[a] All areas and volumes rounded to four significant digits.
[b] Circular openings.
[c] Projected floor area.

Table 2-6. Summary of Areas and Volumes for Underground Excavations

	Configuration	
	Vertical	Horizontal
Total projected floor area of ramps, m^2 (ft^2)[a]	24,850 (267,500)	21,910 (235,800)
Total projected floor area of the repository, excluding ramps, m^2 (ft^2)[a]	983,700 (10,590,000)	486,000 (5,231,000)
Total volume excavated for ramps, m^3 (ft^3)[a]	142,200 (5,023,000)	110,000 (3,886,000)
Total volume excavated for the repository excluding ramps, m^3 (ft^3)[a]	5,465,000 (193,000,000)	2,008,000 (69,310,000)
Ratio of floor area of emplacement drifts to floor area of the repository, %[b]	61	33
Total area within repository boundary, m^2 (ft^2)[a]	5,564,000 (59,900,000)	5,564,000 (59,900,000)
Approximate extraction ratio based on 1375 acre repository area, %[b]	18	8.7

[a] All areas and volumes are rounded to four significant digits.
[b] All percentages are rounded to two significant digits.

3.0 PERFORMANCE GOALS FOR THE SEALING SUBSYSTEM

In this chapter, performance goals for the sealing subsystem are developed. Performance goals are specific values of a performance measure that a system is designed to achieve. The primary performance goals to be developed in the NNWSI Repository Sealing Program are the hydrologic performance goals presented in Section 3.1. These goals represent the allowable amount of water that could enter the waste disposal areas and apply to the engineered barrier subsystem and the shaft and borehole seals subsystem (shaft portion only). These goals provide the input needed to determine the hydrologic design requirements that are needed for sealing components to achieve a desired performance. The need for airborne performance goals is discussed in Section 3.2, and a performance goal for boreholes is proposed in Section 3.3. The remainder of this chapter describes the approaches used in establishing these goals and the appropriate goals that currently are used in the NNWSI Repository Sealing Program. These performance goals are preliminary and subject to change. The authors believe that the model, used to develop the performance goals and described below, is conservative because the waste packages should remain intact longer than the time periods used in the model. In this sense, the model does not reflect the conditions that we expect to occur.

3.1 DEVELOPMENT OF HYDROLOGIC GOALS FOR THE SEALING SUBSYSTEM

Hydrologic performance goals are derived by computing the volume of water required to release radionuclides in amounts equal to the annual release rate established by NRC in 10 CFR 60.113.a.1.ii.B (NRC, 1986). To compute this volume of water a simple radionuclide release model is defined. It is recognized that the data used to develop this model is limited and radionuclide release modeling is complex. Nevertheless, we are developing this radionuclide release model to provide a link between the performance objective of the repository and the design basis for sealing designs. Further, we believe the model presented below is conservative in that the waste package and fuel rods should not fail as rapidly as the rates predicted below. Because failure of waste packages and fuel rods correlate to radionuclide release, lower failure rates mean lower radionuclide releases.

Because (1) the majority of high-level radioactive waste is in the form of spent fuel rods (~90%) and (2) because calculations by Aines (1986) show that releases from glass waste forms are well below the one part in 100,000/yr even if larger than anticipated fluxes (i.e., 1 mm/yr) through the repository horizon are assumed, the model presented below considers the location of radionuclides within fuel rods only. To compute radionuclide releases, waste package and fuel rod failure rates are considered. Fuel rods consist of a stack of uranium dioxide pellets (UO_2 matrix) encapsulated within a helium environment in a Zircaloy tube or cladding (a small amount of UO_2 pellets are contained within stainless steel cladding). Greater than 90% of the radionuclides are located within the UO_2 matrix (Apted et al., 1986, pp. 5-6). Some radionuclides exist outside the matrix in (1) the cladding and structural parts of

the fuel rods; (2) the gap between the fuel pellets and the cladding; and (3) the boundaries between the UO_2 grains. Because radionuclides are distributed within the fuel rod, radionuclide releases are computed from the UO_2 matrix as well as other portions of the fuel rod.

The approach used in this chapter is to compute radionuclide releases from the matrix and, for select radionuclides, from the cladding (and structural parts) and the gap/grain boundaries. It is assumed that the releases from the matrix are directly proportional to the volume of water that contacts the waste. Releases from gap/grain boundaries are computed by making assumptions on the percentage of their inventory in nonmatrix locations. Releases from the cladding are controlled by the corrosion rate of the cladding. To arrive at the release from the matrix portion of the fuel rods, the SPARTAN code (Lin, 1985) is used. The results from the SPARTAN code are presented in Section 3.1.2. In Section 3.1.3 hydrologic performance goals for those radionuclides located in some portion outside the matrix is presented. Hydrologic performance goals for selected radionuclides located exclusively within the matrix also are presented.

3.1.1 Assumptions Used to Compute Radionuclide Releases

As mentioned above, radionuclides are located within the uranium dioxide matrix and outside the matrix. Their releases are controlled by the accessibility of the radionuclides to the environment outside the waste package and the manner in which the radionuclides are released. The release of the radionuclides to the environment outside the waste package is controlled by the failure behavior of the waste package and the fuel rods. The assumptions describing the failure behavior of the waste package and the fuel rods are given in Section 3.1.1.2. Additional assumptions made to compute releases of radionuclides from matrix and nonmatrix locations are given in Section 3.1.1.1.

3.1.1.1 Assumptions Made to Compute Releases From Matrix and Nonmatrix Locations Within Fuel Rods

The majority of radionuclides are located within the matrix. The following assumptions are made to compute the release of radionuclides contained within the matrix:

- Congruent matrix dissolution is assumed. This mechanism limits the fractional release rate of the radionuclides to the fractional release rate of the matrix.

- Each element contained within the matrix is uniformly distributed. While it is known that segregation of radionuclides can occur during irradiation, this fact is not factored into this analysis because the extent of segregation has not been adequately quantified.

- Water contacting the waste saturates the uranium dioxide matrix, therefore, the UO_2 dissolution rate is determined by its solubility coupled with the quantity of water.

- Water will only contact the matrix within a fuel rod after both the fuel rod and the waste package have failed.

As mentioned above, some radionuclides exist outside the matrix; therefore, their release is not controlled solely by matrix dissolution. Eight radionuclides, that exist in some proportion outside the matrix and that are specifically considered in this report, are: Tc-99, C-14, Cs-135, Cs-137, Sr-90, I-129, Ni-59, and Zr-93. Portions of Tc-99, Cs-135, Cs-137*, I-129 and Sr-90 can be located in the pellet-cladding gap or grain boundaries. Portions of C-14, Zr-93, and Ni-59 are contained in the cladding and structural parts of the fuel rods.

Annual radionuclide releases from Cs-135, Cs-137, I-129, Sr-90, and Tc-99 occur from the matrix and the gap/grain boundary. The portion of the inventory located in the matrix is computed using the same assumption given above. The portion of the inventory located in the gap/grain boundary is released rapidly. Cesium can be released in quantities similar to the fission gas release, typically less than 0.5%. A portion of I-129 may be released similarly (Wilson, 1985, p. 42). This highly mobile portion is assumed to be immediately released in the year when the fuel rods fail. Again, release from the waste packages occurs only after the waste package fails. In this report, it is assumed that 0.5% of the inventory of Cs-135, Cs-137, I-129, and Sr-90 is released immediately once the waste packages and fuel rods have failed.

Experimental works have shown that Tc-99 preferentially dissolves relative to the bulk of the matrix. The preferential dissolution factors reported by Oversby and Wilson (1985, p. 5) were 3.8 and 8.7 for different fuels. The annual release of Tc-99 is equal to the release from the exposed matrix (i.e., the amount released by congruent dissolution) plus an amount immediately released when the fuel rods and waste packages have both failed. The preferential dissolution factor assumed in this report is 10.

For C-14, Ni-59, and Zr-93, it was assumed that their release was controlled in part or total by the uniform corrosion of the Zircaloy cladding. As

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*Other radionuclides may exist in some portion in the pellet-cladding gap or grain boundary, such as Mo-93 and Se-79. The development of a complete source-term model for radionuclides release is an ongoing activity within the NNWSI Project. Plans for the development of the source-term model are given in Section 8.3.5.10 of the NNWSI Site Characterization Plan (DOE, in preparation).

reported in Van Koynenburg et al. (1984, p. 2), the percentage of the C-14 inventory in the structural parts of the fuel rods could be 67% for BWR and 61% for PWR spent fuel rods. In DOE (1979, pp. 3.3.20 - 3.3.21), the percentage of C-14 in the Zircaloy cladding hulls is approximately 7% of the total inventory. It is assumed in this report that C-14, Ni-59, and Zr-93 are located within the fuel cladding in the following percentages: 67% (Van Koynenburg et al., 1984, p. 2), 100% (DOE, 1979, Volume 1, p. 3.3.40), and 5% (DOE, 1979, Volume 1, pp. 3.3.20 - 3.3.21).

Because zirconium and Zircaloy cladding are extremely corrosion resistant (Uhlig, 1967, p. 324; Woodley, 1983, p. A-3; Rothman, 1984, pp. 1, 11; Oversby and McCright, 1984, p. 6), the release rate of the radionuclides contained within the fuel cladding is expected to be extremely slow. Rothman used Hillner's equation for long-term oxidation behavior and computed the depth of oxidized Zircaloy to be 17 μm at a constant temperature of 180°C for 10,000 years. If the minimum thickness for Zircaloy cladding is assumed to be 600 μm (Rothman, 1984, p. 2; Woodley, 1983, p. 12) and the release of radionuclides contained within the cladding was controlled by uniform corrosion, it would take approximately 350,000 years to release all of the radionuclides contained within the cladding. This time period would correspond to a release rate of 2.8×10^{-6} of the total inventory per year.

The time to release all the radionuclides would be longer if temperatures were lower. Using Hillner's long-term oxidation equations at a temperature of 100°C, the depth of oxidized Zircaloy after 10,000 years is calculated to be 0.0469 μm. If we assumed that the release of radionuclides were controlled by this uniform corrosion, then the release rate per year for radionuclides contained within the fuel cladding would be approximately 7.8×10^{-9} of the total inventory. Failure of the waste package as a result of other failure mechanisms such as pitting or stress corrosion could breach the fuel cladding and allow water entry within the gap region. Because water would be on both sides of the cladding, uniform corrosion could occur at a rate twice that calculated. This factor (twice the computed amount) is used in the computation of yearly radionuclide release rates. Further, it is assumed that the corrosion rate of the fuel cladding should not be affected by the sporadic nature of the water flows assumed in these analyses.

From the preceding discussion all of the inventory of Ni-59 is assumed to be released at a rate of 7.8×10^{-9} per year as computed from the corrosion of the Zircaloy cladding. A portion of the total inventory of C-14 and Zr-93 also can be released through the corrosion of the cladding. Additional assumptions associated with the corrosion of cladding include

- corrosion of outside surface of the cladding begins when failure of the waste package occurs, and

- corrosion of the inside surface of the cladding begins when the fuel rod fails.

In addition to this release from the cladding, C-14 and Zr-93 also can be released from the matrix. The release is controlled by the congruent dissolution assumption given above. A third component of release is also considered for C-14. This component is the release of C-14 as a gas from the surface of the fuel rods when the waste package is breached. The percent of C-14 inventory is assumed to be 0.3% (Van Koynenburg, 1984, p. 6). Because this portion of C-14 is released as a gas, it is assumed that this amount of C-14 is released immediately when the waste package fails.

3.1.1.2 Assumptions Concerning the Failure of the Waste Package

In addition to the assumptions given in the previous section, the radionuclide release model presented in this report included several assumptions on the failure of the waste packages and the fuel rods. The assumptions made in this section control, in part, the release of radionuclides from the matrix, the gap/grain boundary, and the cladding. The following assumptions are made:

- During the first 300 years following emplacement it is assumed that containment is substantially complete.

- At 301 years after closure, the waste packages begin to fail at a linear rate of 0.1% per year. Therefore, 1,300 years after closure, all the waste packages have failed.

- At the time of closure, 0.02% of the fuel rods have failed. Woodley (1983, p. 14) cites that failure levels for typical LWR fuel is approximately 0.01% to 0.02%.

- Fuel rods will begin to fail immediately after closure at a rate of 0.01% . Therefore, at 300 years after closure, 3% of the fuel rods have failed.

- Fuel rods, contained within intact waste packages will continue to fail at a rate of 0.01%. Once the waste package has failed, however, the remaining, unfailed fuel rods will fail at an accelerated rate of 0.1% per year. Use of these values suggests that all of the fuel rods will be breached 2,300 years after closure of the repository.

- Failed waste packages and fuel rods are uniformly distributed throughout the repository. This is a reasonable assumption because no segregation of failed fuel rods is currently planned when they are emplaced in waste packages.

Because the failure rates presented above for fuel rods are speculative, caution should be exercised in their use. Nevertheless, the authors feel that the use of the failure rates for the waste package and the fuel rods are reasonably conservative because we do not expect failure rates to be that high. For example, it is estimated that containment will be substantially complete from 3,000 to 30,000 years (DOE, 1986, p. 6-372). Even if the lower estimate were achieved, only limited radionuclide releases could occur for the first 3,000 years after closure.

The assumptions given above are used to develop two factors, a matrix release factor and an immediate release factor. The matrix release factor represents the portion of the UO_2 matrix that is available for dissolution. This suggests that both the waste package and fuel rods must fail in order for the UO_2 matrix to be available for dissolution. This fact was used to compute the radionuclide releases from the matrix. The immediate release factor is that portion of the total inventory that fails in any given year. This factor is applied to those radionuclides that exist in some portion outside the matrix and can be released easily once the fuel rods are breached. This factor is applied to Cs-135, Cs-137, I-129, and Sr-90. Implicit in this factor is the assumption that the failed fuel rod must be located within a failed waste package. Carbon-14 is also released immediately, but is controlled by the failure rate of the waste package, i.e., once the waste package fails, a portion (0.3%) of C-14 in the form of a gas is released immediately.

The matrix and immediate release factor are displayed in Figures 3-1a and 3-1b. Mathematically, the immediate release factor (IRF) can be described as follows:

$$IRF_y = \frac{frf_y}{tfr} \qquad (3\text{-}1)$$

where

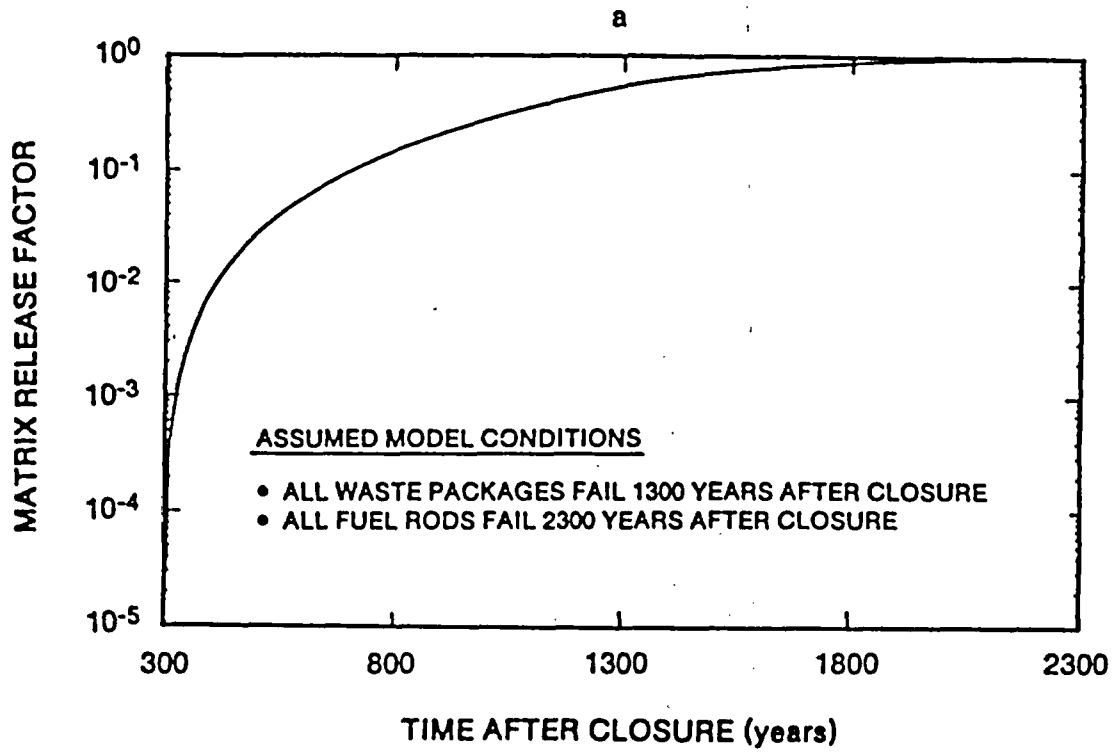
y = year after closure (range is 301 to 2,300 years after closure),

frf_y = number of newly breached fuel rods available for release in year 'y', and

tfr = total number of fuel rods in the repository.

The variable 'frf_y' is dependent on the time after closure that is being considered. The relationships used for different time periods is defined below.

when $y = 301$, $frf_{301} = c_y$;



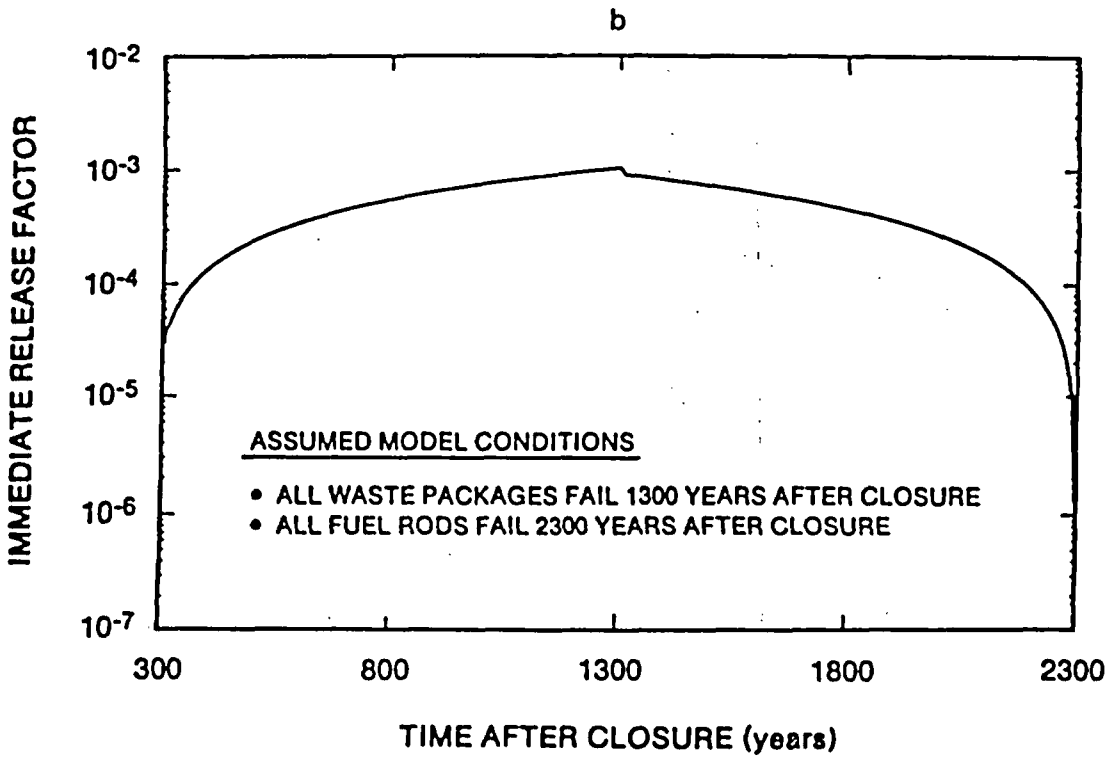


Figure 3-1. Matrix and Immediate Release Factors Following a Containment Period of 300 Years

$$302 \leq y \leq 1{,}300, \qquad frf_y = \sum_{n=301}^{n=y} b_n + c_y;$$

$$1{,}301 \leq y \leq 2{,}300, \qquad frf_y = \sum_{n=y-1{,}000}^{n=1{,}300} b_n;$$

where

b_n = 0.1% of unfailed fuel rods at time 'n' in failed waste packages, and

c_y = number of failed fuel rods contained in waste packages failed in year 'y'.

The matrix release factor (MRF) can be defined as

$$MRF_y = \sum_{n=301}^{n=y} IRF_n \quad . \tag{3-2}$$

Because the MRF represents the yearly accumulation of the IRF, its value ranges from the IRF value at year 301 after closure to 1 at year 2,300 after closure. The range is reflected in Figure 3-1a. At 1,000 years following closure, the model predicts 28% of the matrix is exposed so that dissolution can occur. This assumed value of exposed waste is much higher than the amount of fuel expected to be available for dissolution. The values for the IRF factor increase from 300 to 1,300 years after closure. As indicated above, the IRF is dependent on the failure of fuel rods in both previously failed and newly failed waste packages up to 1,300 years after closure. Because the waste package failure rate is constant and because it is assumed that fuel rods continue to fail even while in intact waste packages, failure of a new set of waste packages each year exposes a greater number of failed fuel rods than the year before. The offset at 1,301 years after closure represents the last contribution of failed fuel rods that have remained in the last set of waste packages remaining intact up to 1,300 years after closure. Beyond 1,301 years, the number of failed fuel rods decreases each year until all have failed at 2,300 years after closure.

3.1.2 Radionuclide Releases From the UO_2 Matrix Using the SPARTAN Code

The assumptions given in Sections 3.1.1.1 and 3.1.1.2 are used to compute the hydrologic performance goals. To compute radionuclide releases from the UO_2 matrix the computer code SPARTAN was used. The results from SPARTAN were then coupled with the assumed release rates of those radionuclides located in nonmatrix locations. These yearly radionuclide releases were then adjusted considering the immediate and matrix release factors described in Section 3.1.1.2.

The various steps mentioned previously are described below in more detail. The remainder of this section describes the releases of radionuclides from the UO_2 matrix. Section 3.1.3 presents performance goals for select radionuclides and Section 3.1.4 discusses the sensitivity of the assumptions made on the percentages of radionuclides located in nonmatrix locations and the manner in which they are released.

SPARTAN is a computer code that can calculate radionuclide transport. It incorporates Darcy flow, radionuclide decay, and convective transport with constant retardation of radionuclides with respect to water flow. In the analysis presented below, the release of radionuclides at the edge of the waste package is computed. This approach is implemented so that a direct comparison can be made to the NRC performance criterion for the release from the engineered barrier system. The release of radionuclides is computed using a mass transport formulation. The primary formulas used in SPARTAN are given below.

Because congruent leaching is assumed in SPARTAN, the release of individual radionuclides is controlled by the dissolution of the uranium matrix. The dissolution of the uranium matrix 'D_μ' is defined as

$$D_\mu(t) = q * S_\mu \tag{3-3}$$

where

$D_\mu(t)$ = dissolution of uranium at time t (kg/yr),
q = amount of water contacting the waste package (m^3/yr), and
S_μ = solubility limit of the uranium matrix (kg/m^3).

The dissolution of an individual radionuclide 'D_j' will then be

$$D_j(t) = D_\mu(t)\ [m_j(t)/m_\mu(t)] \tag{3-4}$$

where

$D_j(t)$ = dissolution of the 'jth' radionuclide at time 't' (kg/yr),

$m_j(t)$ = mass inventory of the 'jth' radionuclide at time 't' (kg/unit of waste considered), and

$m_\mu(t)$ = mass inventory of uranium at time 't' (kg/unit of waste considered).

Because 'q' is directly proportional to the annual release of any radionuclide, an amount of water can be computed that represents the maximum amount of water, 'q_{max}', that can contact the waste form and not violate the NRC criteria. To compute 'q_{max}' the ratio of the annual release of any radionuclide to its NRC limit should be equal to one as indicated below.

$$\frac{D_j(t)}{\text{NRC limit}_j} = \frac{(q_{max} \cdot S_\mu)\ [m_j(t)/m_\mu(t)]}{\text{NRC limit}_j} = 1 \qquad (3\text{-}5)$$

Because S_μ, $m_j(t)$, $m_\mu(t)$, and NRC limit_j are specified, q_{max} can be computed.

Differences in the values for $m_j(t)$, $m_\mu(t)$, and the NRC limits associated with specific radionuclides lead to different values of 'q_{max}'. As implied in Equations 3-3 through 3-5, additional information is necessary to compute 'q_{max}'. This information includes the radionuclide inventory and the solubility limit of the uranium matrix. The radionuclide inventory used in SPARTAN is the 10-year, out-of-reactor inventory specified in DOE (1979). The assumed, solubility limit of the uranium matrix is 5 μg/ml (5×10^{-3} kg/m^3). This value corresponds to the highest uranium concentration (unfiltered solution), determined experimentally for bare fuel in J-13 well water (Oversby and McCright, 1984, p. 9; Wilson and Oversby, 1985, p. 9).

Because 'q_{max}' is inversely proportional to the ratio of the release of a specific radionuclide to its NRC limit, 'q_{max}' for the jth radionuclide can be computed using the following equation:

$$q_{max_j}(t) = \frac{q}{D_j(t)/\text{NRC limit}_j} \qquad (3\text{-}6)$$

The value 'q' represents any assumed volume of water contacting the waste (see Appendix B for typical SPARTAN results). The denominator represents the ratio of the release of a specific radionuclide to its NRC limit associated with the amount 'q'. Knowing the ratio of $D_j(t)/\text{NRC limit}_j$ for a specific 'q' value, '$q_{max_j}(t)$' can be computed for all the radionuclides at any time. The value '$q_{max_j}(t)$' represents the hydrologic performance goal for a radionuclide 'j' at any time 't'. The greater the value of $D_j(t)/\text{NRC limit}_j$, the lower is the hydrologic performance goal, i.e., the maximum amount of water that can contact the UO_2 matrix and not exceed the NRC criterion of 1 part release in 10^5 of the 1,000-year inventory of a specific radionuclide.

3.1.3 Computation of Hydrologic Performance Goals for Select Radionuclides

In this section hydrologic performance goals are developed for these radionuclides considering an assumed failure of the waste packages and fuel rods. In Figure B-1 (Appendix B), Pu-238, Am-241, and Pu-242 are identified as those radionuclides that have the lowest and most restrictive hydrologic performance goals compared with other radionuclides contained totally within the matrix. The hydrologic performance goals of those radionuclides located in part or totally outside the UO_2 matrix are also computed. These are: Cs-135, Cs-137, Sr-90, Tc-99, C-14, I-129, Zr-93, and Ni-59. For these radionuclides, releases from nonmatrix locations must also be considered. The formulas used to compute the hydrologic performance goals are given in Table 3-1. The

Table 3-1. Formulas Used to Compute Hydrologic Performance Goals for Selected Radionuclides

$$q_{max_j}(t) = \frac{\text{NRC limit}_j \cdot q}{MRF(t) \cdot R_j(t)}$$

(Pu-238, Pu-242, Am-241)

$$q_{max_j}(t) = \frac{\text{NRC limit}_j - [DF \cdot IRF(t) \cdot INV_j(t)]}{MRF(t) \cdot (1 - DF) \cdot R_j(t)} \cdot q$$

(Cs-135, Cs-137, Sr-90, I-129)

$$q_{max_j}(t) = \frac{\text{NRC limit}_j \cdot q}{IRF(t) \cdot PDF \cdot R_j(t) + R_j(t) \cdot MRF(t)}$$

(Tc-99)

$$q_{max_j}(t) = \frac{\text{NRC limit}_j - [COR \cdot INV_j(t) \cdot DF] \cdot [IRF(t) + MRF(t)]}{MRF(t) \cdot (1 - DF) \cdot R_j(t)} \cdot q$$

(Zr-93)

$$q_{max_j}(t) = \frac{\text{NRC limit}_j - [COR \cdot DFC \cdot INV_j(t)]\,[IRF(t) + MRF(t)] - [0.001 \cdot INV_j(t) \cdot DF]}{MRF(t) \cdot (1 - DFC - DF) \cdot R_j(t)} \cdot q$$

(C-14)

$$MCOR(t) = \frac{\text{NRC limit}_j}{IRF(t) \cdot INV_j(t) + MRF(t) \cdot INV_j(t)}$$

(Ni-59)

where

- DF = decimal fraction of 'jth' radionuclide available for immediate release,
- $INV_j(t)$ = inventory of the 'jth' radionuclide at time 't', (Ci),
- $R_j(t)$ = dissolution of the 'jth' radionuclide at time 't' for a specific volume of water, 'q' (Ci/yr),
- PDF = preferential dissolution factor,
- MCOR(t) = maximum allowable corrosion rate at time 't' (decimal fraction of available inventory that can be relased per year),
- COR = assumed corrosion rate (decimal fraction of available inventory that can be released per year), and
- DFC = decimal fraction in cladding.

results are presented in Figure 3-2. The curves presented in this figure use the assumptions given in Section 3.1.1.1. The time period displayed on Figure 3-2 goes from 300 to 3,000 years after closure. This incorporates the time period where the waste package and fuel rods are assumed to fail, i.e., from 300 to 2,300 years after closure. By our radionuclide model, all waste packages and fuel rods are failed beyond 2,300 years; therefore, all of the UO_2 matrix is available for contact with water after 2,300 years after closure.

For specific radionuclides indicated in Figure 3-2, the hydrologic performance goals are always greater than 1.3×10^5 m^3 of water/year. For those radionuclides with short half-lives, (i.e., Sr-90, Cs-137, and Pu-238) the lowest performance goal occurs soon after waste package and fuel rod failure occurs. This occurs because their inventory is still high 300 to 400 years after closure. Beyond 400 years after closure their inventory available for release decreases substantially. For the radionuclides with longer half-lives, their hydrologic performance goal reaches a near steady state after the waste packages and fuel rods have all failed at 2,300 years. Again, total failure of the waste packages and fuel rods at 2,300 years after closure is not expected.

3.1.4 Sensitivity of the Assumptions Used to Compute the Hydrologic Performance Goals

As mentioned earlier, the hydrologic performance goals computed in Figure 3-2 used reasonably conservative and realistic assumptions given in Section 3.1.1.1. The assumptions concerning inventories of radionuclides outside of the matrix and release rates of these inventories are based on available laboratory data. To illustrate how these assumptions affect the hydrologic performance goals, these assumptions are varied. If the assumptions on waste packages and fuel rod failures are modified, the results from the sensitivity study presented below will change.

Figure 3-3 shows how the performance goals for I-129 and Cs-135 are affected if the percentage of the inventory* available for immediate release is increased. This figure illustrates that increasing the amount available for immediate release decreases the performance goals. The assumption of immediate release controls the total release of the radionuclides. Increasing the

*A study on the leaching of Cs-137, Cs-134, and I-129 from CANDU fuels (Johnson et al., 1983, p. 474) illustrated a correlation between fission gas release and the fraction of rapidly released cesium and iodine. For the higher fission gas releases, the fraction of cesium release was about 50% of the gas release. This conclusion together with the range of percentages of xenon gas release from the gap (Johnson et al., 1983, p. 472) would suggest a potential range of cesium and iodine that could be released rapidly. It should also be noted that segregation of fission products from UO_2 matrix occurs to a greater extent in the CANDU fuels than in the PWR or BWR fuels (DOE, in preparation, p. 7-191).

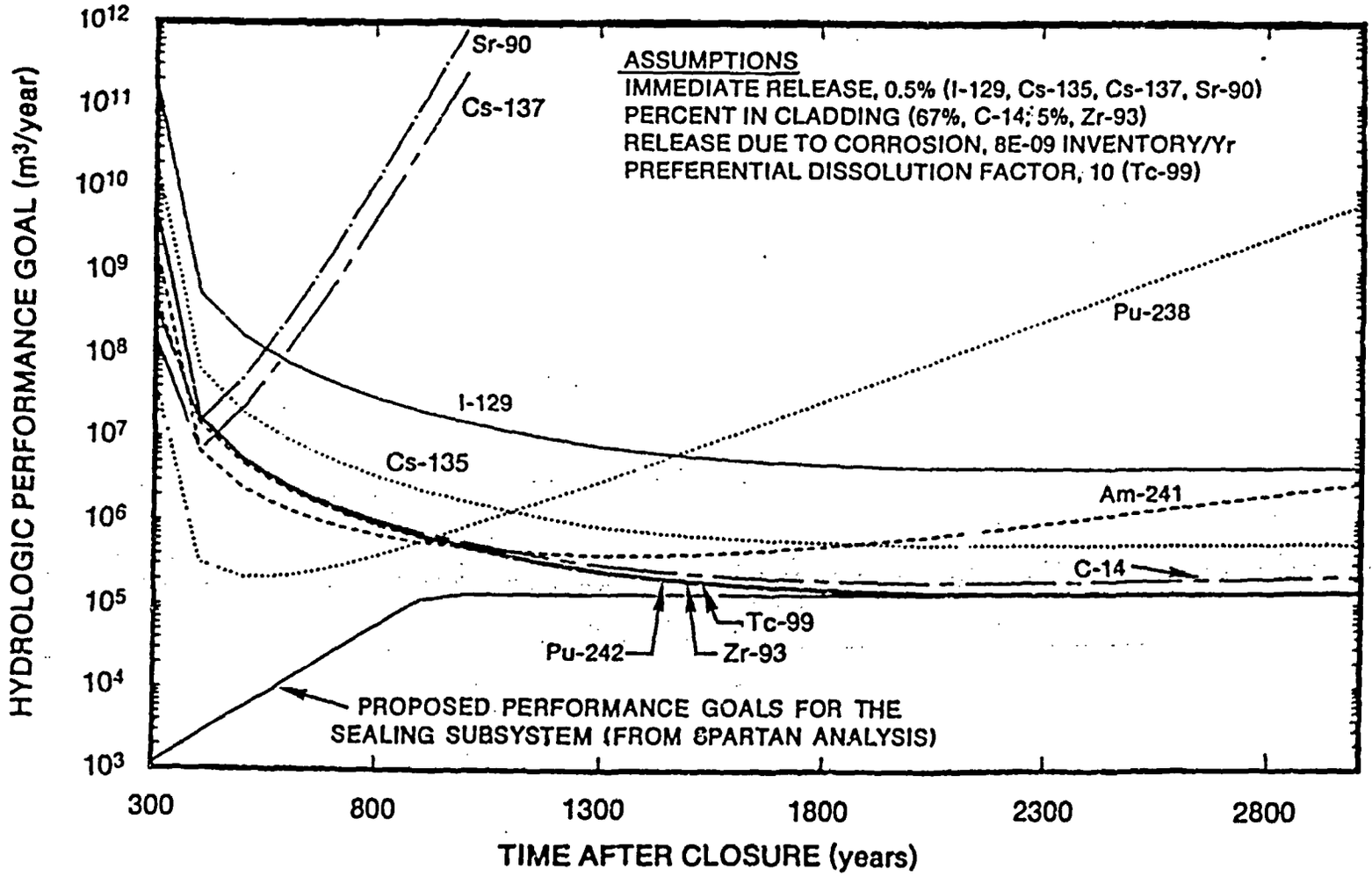
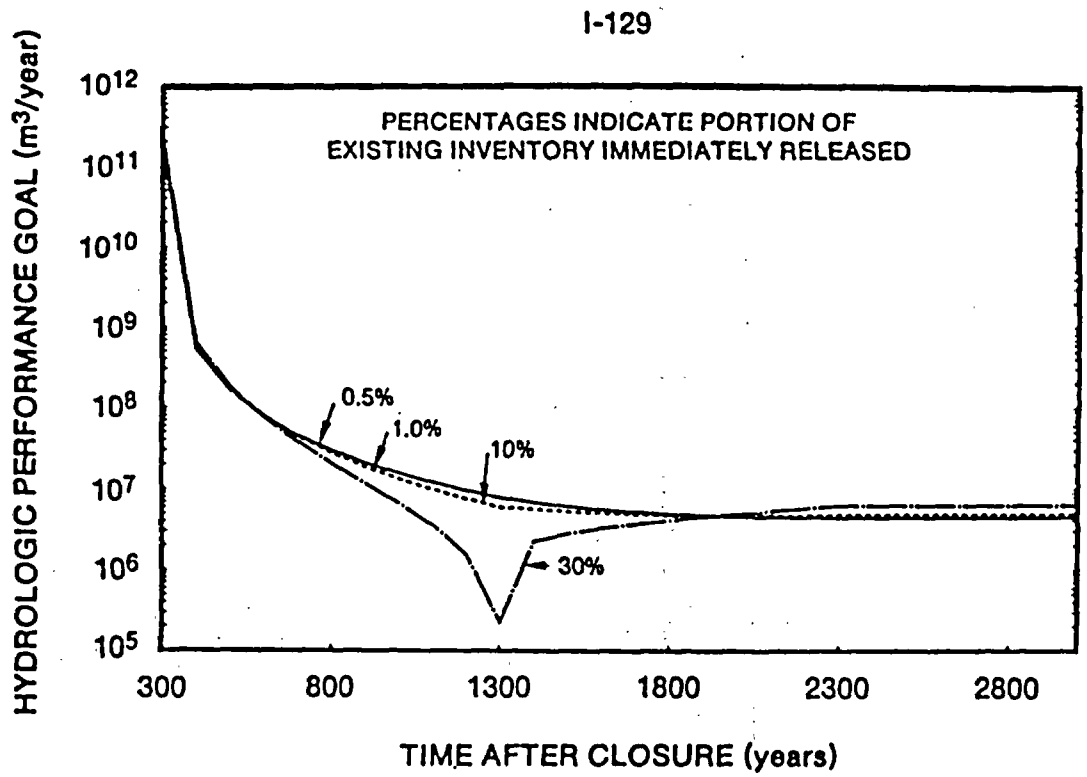


Figure 3-2. Maximum-Allowable-Hydrologic-Performance Goals for Select Radionuclides and Proposed-Performance Goals for the Sealing Subsystem



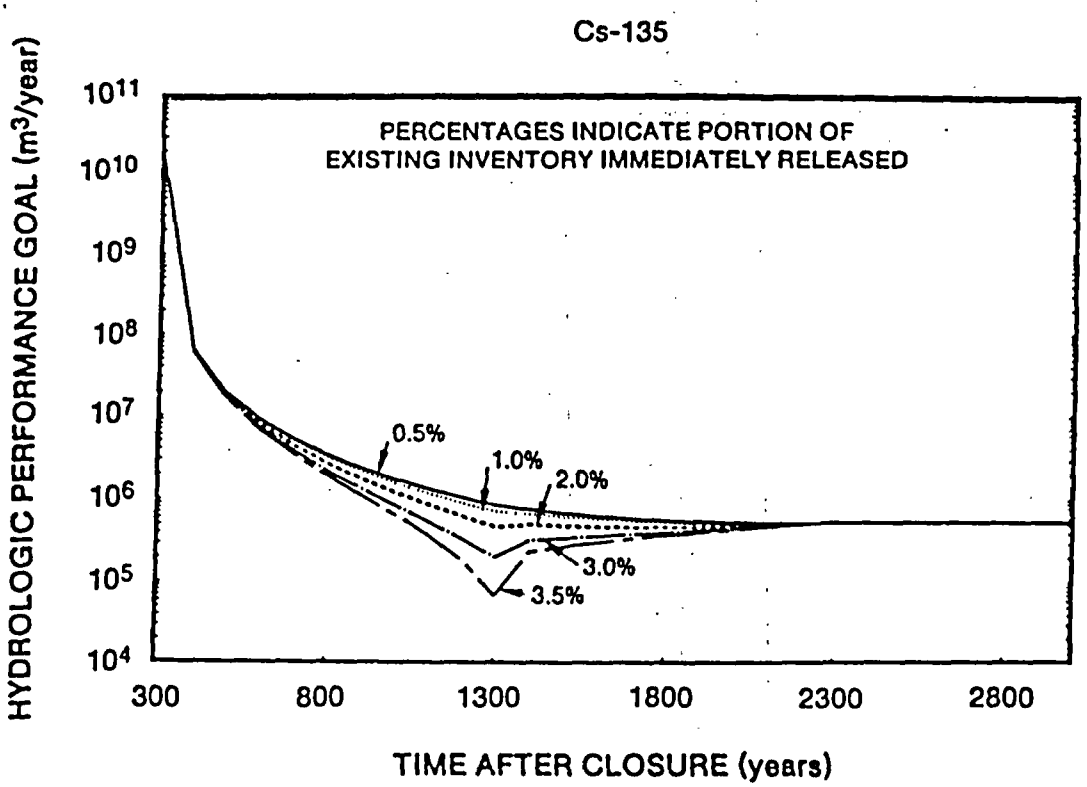
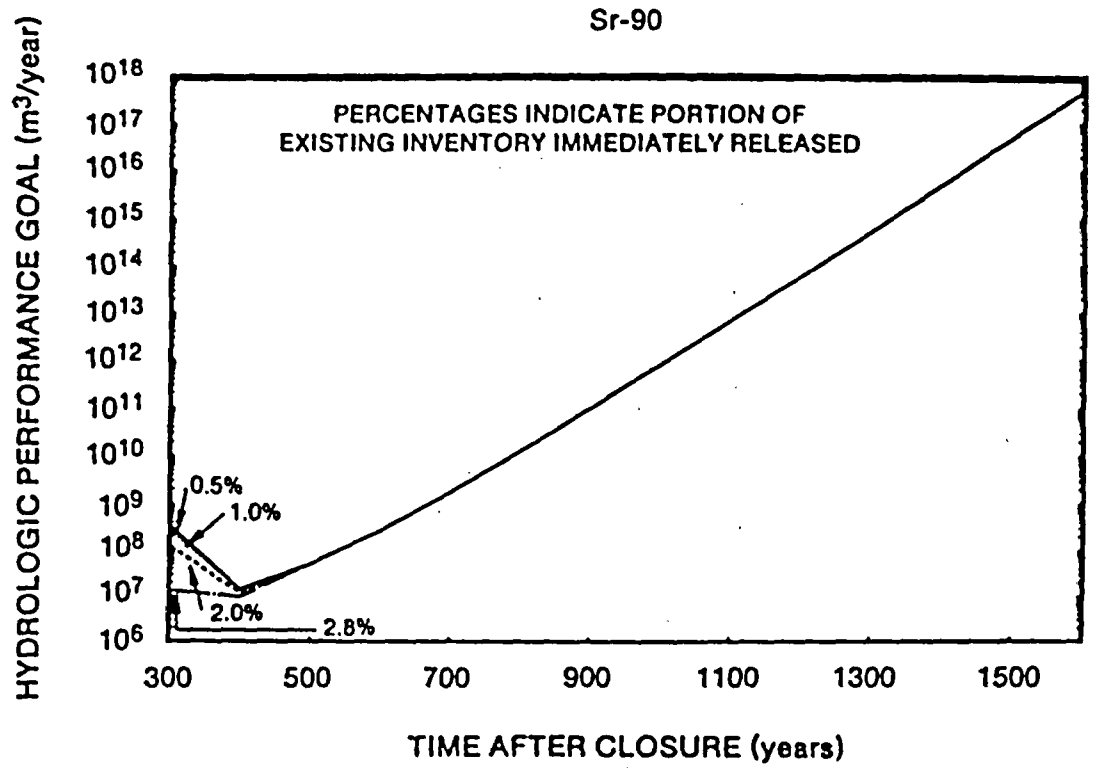


Figure 3-3. Sensitivity to the Assumptions Used to Compute the Hydrologic Performance Goals for I-129 and Cs-135

percentage of I-129 from 0.5% to 30% and Cs-135 from 0.5% to 3.5% reduces the performance goal. The pronounced discontinuity in the performance goal curves occurs at 1,300 years because the half-lives of these radionuclides are long, therefore, the inventory has not decreased substantially from the time of emplacement. Therefore, if we assume that the percentage of the radionuclides available for immediate release increases, a greater amount of radionuclides are released as the fuel rod and waste package fail. Because the immediate release of these radionuclides occurs at the same year the fuel rods are in failed waste packages are breached, and because the largest number of fuel rods in failed waste packages are breached at 1,300 years after closure, the largest release occurs at this time. As the radionuclide releases from non-UO_2-matrix sources increases, the release from the UO_2 matrix must be reduced to assure that the NRC criteria of 1 part in 10^5 is met. Because radionuclide release from the matrix is controlled by water contacting the waste, a reduction in the allowable amount of water contacting the waste is necessary. This reduction in the hydrologic performance goal is observed at 1,300 years.

Similar alterations in performance goal curves are observed for Sr-90 and Cs-137. However, modification of performance goal curve occurs immediately after the containment period of 300 years ends. These modifications occur from 300 to 400 years after closure because the inventory of both of these radionuclides is high. Beyond 400 years after closure their inventory, and subsequently their releases, decrease rapidly. If the waste package remained intact for up to 500 years after closure, the inventory of these radionuclides would have decayed sufficiently where the assumption of the percent of immediate release would not noticeably modify the performance goals as indicated in Figure 3-4.

Figure 3-5 illustrates how varying the corrosion rate and percent of the inventory in the fuel cladding affects the performance goals for Zr-93 and C-14. Two corrosion rates are selected to develop these curves 0.0469 μm/10,000 years to 17 μm/10,000 years. The former corrosion rate was given in Section 3.1.1.1 and the later corrosion rate was that computed by Rothman (1984, p. 11) for the depth of oxidated Zircaloy that would occur over a 10,000 year period at a temperature of 180°C. These depths of oxidized Zircaloy would correspond to a value of 7.8×10^{-9} and 2.8×10^{-6} of the total available inventory of a radionuclide in any year. Using these values and varying the percentage in the cladding, variations in the performance goal curves are observed in Figure 3-5. The greater the amount of Zr-93 in the cladding, the greater will be the performance goals. This suggests that release of Zr-93 from the matrix will have a greater influence on the performance goals. For C-14 four variables were changed. The corrosion rate, the percentage of C-14 in the cladding and matrix, and the amount available for immediate release as a gas were changed. The percentages of 61% and 67% of C-14 in the cladding represent the amounts of C-14 in PWR and BWR fuel respectively (Van Koynenburg et al., 1984, p. 2). All combinations of variables in Figure 3-5 illustrate little variation in the performance goals. By contrasting curves 1 and 2, the effect of corrosion rate can be observed. By



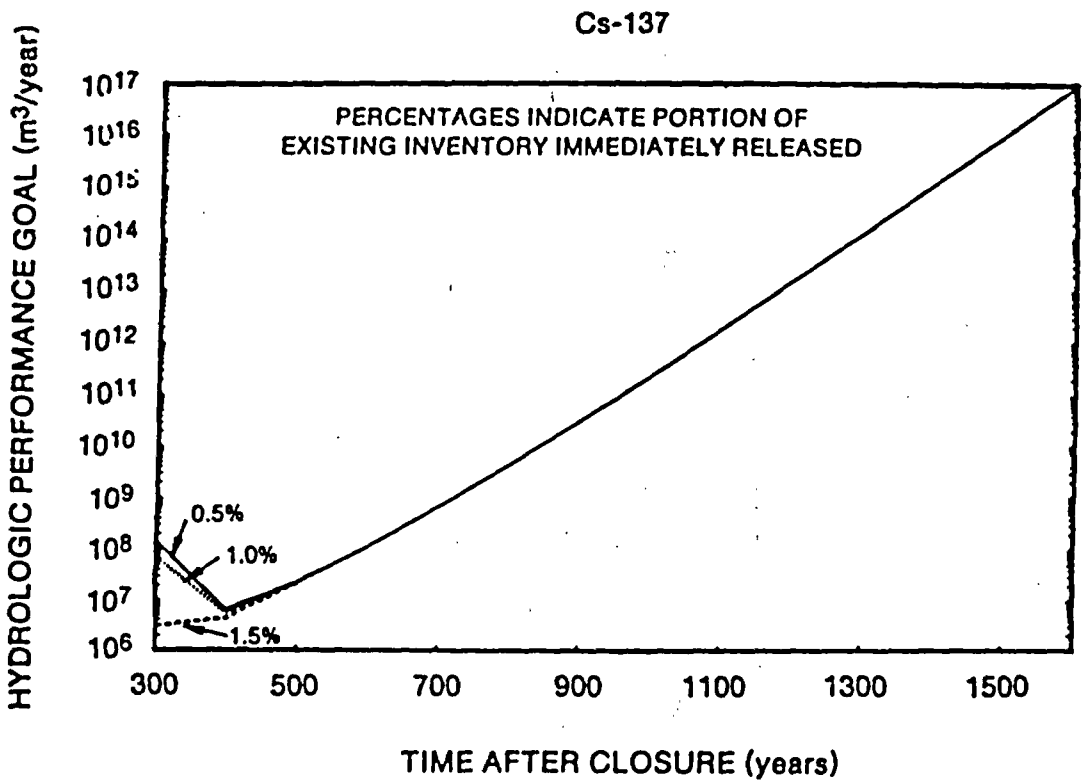
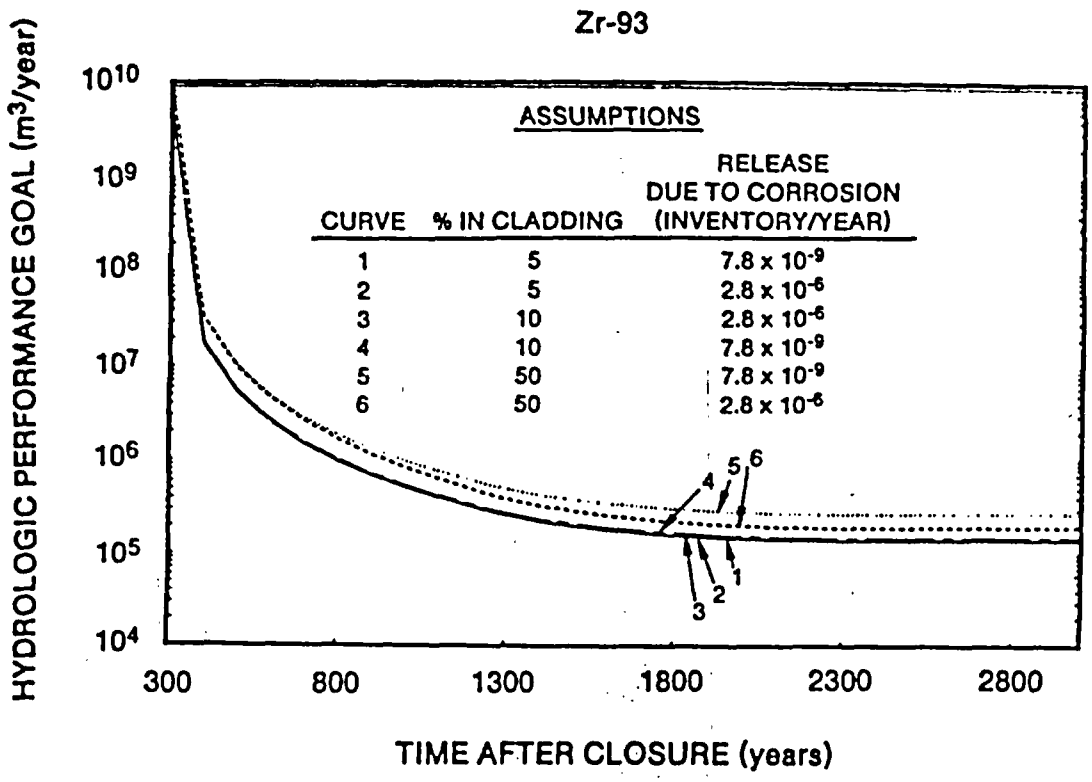


Figure 3-4. Sensitivity of the Assumptions Used to Compute the Hydrologic Performance Goals for Sr-90 and Cs-137



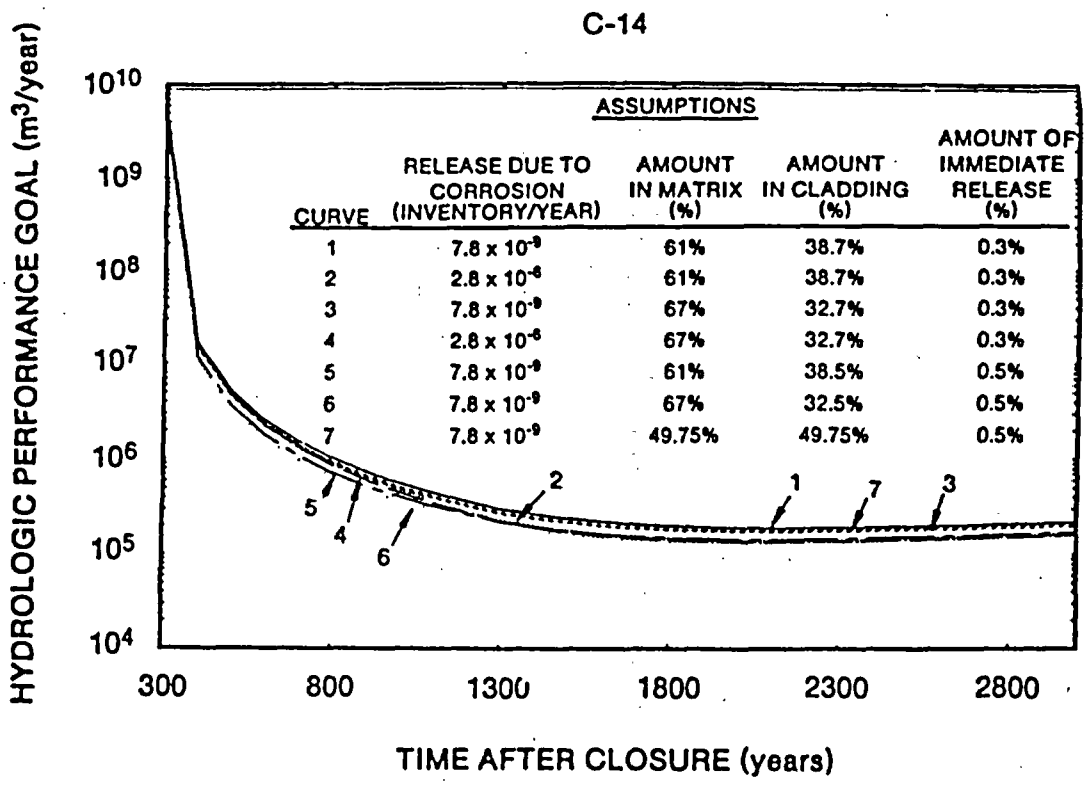


Figure 3-5. Sensitivity of the Assumptions Used to Compute the Hydrologic Performance Goals for Zr-93 and C-14

increasing the corrosion rate by greater than two orders of magnitude, the performance goals are reduced by approximately 25%. Increasing the percentage of C-14 in the cladding from 61% (curves 1 and 2) to 67% (curves 3 and 4) decreases the performance goals by less than 10%. A similar reduction is noticed by comparing curves 5, 6, and 7. By increasing the percent of C-14 in the cladding from about 50% to 67%, the performance goal decreases approximately 25%. A final comparison was made by arbitrarily increasing the amount of C-14 that could be immediately released from 0.3% to 0.5%. Curves 1 and 5 reflect the difference that occurs in the performance goal curves by making this change. By increasing the C-14 that can be immediately released by 0.2%, the change in performance goals is about 25%. From the preceding discussion on C-14, the variable that would most significantly change the hydrologic performance goals is the amount of C-14 that could be instantaneously released once the waste package fails.

The last two radionuclides considered in this sensitivity analysis are Tc-99 and Ni-59 (see Figure 3-6). It is assumed that the release of Tc-99 is controlled by the amount released congruently from the matrix plus a preferential amount dissolved from the matrix. This preferential amount is defined by the preferential dissolution factor (PDF). In the analysis presented in Section 3.1.1.1, a PDF of 10 was used. The PDF, when increased to a value of 400, reduces the performance goals, but their values are still greater than the steady state, performance goal of about 1.3×10^5 m^3/year.

The most pronounced difference in the release of Tc-99 occurs immediately after the containment period of 300 years. The reason for this is that the amount of waste available for release using the congruent leaching assumption is small in comparison to that amount that is preferentially released from the matrix. This preferential release occurs the same year that failed fuel rods in failed waste packages occur.

Because Ni-59 is assumed to be located totally within the cladding and structural parts, hydrologic performance goals were not computed. Rather, the maximum-allowable, inventory loss per year was computed. The computed inventory loss of a specific radionuclide contained within the cladding was assumed earlier as 7.8×10^{-9} of the inventory/year. The difference between this value and the values given in Figure 3-6 represents a factor of safety that can be associated with the release of Ni-59.

From the sensitivity analysis presented above, it is apparent that some variation in the modeling assumptions, dealing with the location of radionuclides and that portion that can be immediately released, can occur and not significantly reduce the hydrologic performance goals presented in Figure 3-2. Therefore, we believe that the hydrologic performance goals presented for specific radionuclides in Figure 3-3 are reasonably conservative and realistic. Nevertheless, to add an additional degree of conservatism and to simplify the future analysis in this report, one set of hydrologic performance goals will be used. The proposed set of hydrologic performance goals is taken from the

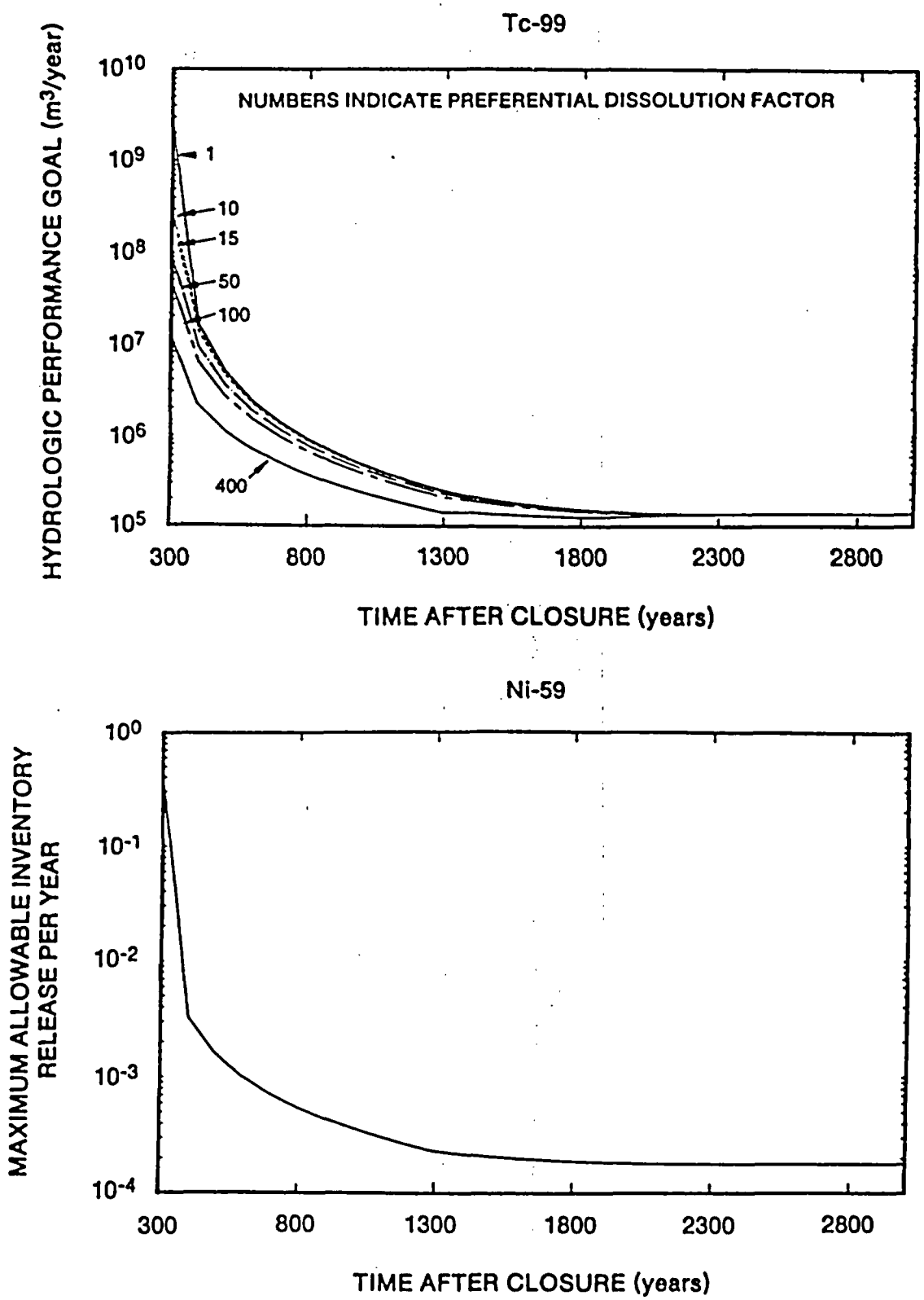


Figure 3-6. Sensitivity of the Assumptions Used to Compute the Hydrologic Performance Goals for Tc-99 and the Maximum-Allowable-Inventory Releases per Year for Ni-59

SPARTAN analysis (see Figure 3-2). Specific values of the design-basis-performance goals are given in Table 3-2. In column 2, the maximum-allowable-performance goals are indicated. The values in column 2 are based on the lowest value of the maximum-allowable goal for any specific radionuclide given in Figure 3-2. The difference between the design-basis-performance goal and the maximum-allowable goal for a specific radionuclide represents a safety margin.

3.2 DEVELOPMENT OF THE AIRBORNE PERFORMANCE GOAL FOR THE SEALING SUBSYSTEM

Shafts represent potential pathways that could "compromise the geologic repository's ability to meet the performance objectives for the period following permanent closure" (NRC, 1986, p. 625). The performance objective, defined in 10 CFR 60.112 (NRC, 1986, p. 618), indicates that the shafts and their seals shall be designed to assure that releases to the accessible environment should comply with the release limits specified in 40 CFR 191 (EPA, 1986). Release to the accessible environment can occur through water transport or air transport. Air flow out of the repository can be induced by convection (as temperatures in the repository rises because of waste decay) and by changes in barometric pressure at the ground surface.

Some potential gaseous species (Xe isotopes, Rn, Kr-85, and H-3) can be eliminated from concern because of their short half-lives, assuming that the containment period will be 300 to 1,000 years. The radionuclides that could potentially enter the repository in a gaseous state are C-14 and I-129 (Van Koynenburg et al., 1984, p. 1). As indicated by Van Koynenburg, 0.3% of the C-14 inventory in a stored canister of spent fuel might be released as a gas. Portions of the I-129 inventory in spent fuel may be concentrated in the pellet-cladding gap and/or on the grain boundaries of the fuel (Oversby and McCright, 1985, p. 10). The percentage of the total inventory is assumed to be 0.5% (Oversby and Wilson, 1985, 0. 9). Release of the gaseous forms of C-14 and I-129 have been considered in the development of the hydrologic performance goals in Section 3.1.3.

In determining whether or not the shafts represent a preferred pathway, it is important to consider the total inventory of C-14 that can be released as a gas through the shafts. The total allowable C-14 and I-129 inventories that can be released to the accessible environment up to 10,000 years after closure are 100 curies/1,000 MTHM. The total inventories of C-14 and I-129 that are released from the waste packages, using the radionuclide release model presented earlier in this chapter, are approximately 4 Ci and 0.2 Ci, respectively. Because these values are a small percentage of the total allowable release, we adopt the position for our current planning basis that the requirements given in 10 CFR 60.134 are satisfied if the radionuclide releases of C-14 and I-129 as a gas out of the shaft is restricted to one percent of the allowable release limit for each radionuclide. Because the allowable release limit is 100 Ci/1,000 MTHM, the performance goal would be 1 Ci released for each radionuclide. This would mean reducing the total gaseous

Table 3-2. Hydrologic Performance Goals for the Sealing Subsystem[a]

Time After Repository Closure, years	Maximum-Allowable-Performance Goal,[b] m^3/year	Design-Basis-Performance Goal,[c] m^3/year
0 - 300	3.88×10^7 [d]	1,180[d]
400	3.12×10^5	2,600
500	2.09×10^5	5,690
600	2.18×10^5	12,300
700	2.74×10^5	26,500
800	3.83×10^5	55,900
900	5.62×10^5	116,000
1,000 - 10,000	3.79×10^5 to 1.36×10^5	136,000

(a) Applies to selected sealing components located within the underground facility, shafts, and ramps.

(b) Represents the lowest, hydrologic performance goal for any radionuclide in Figure 3-2. The curve for Pu-238 displays the lowest performance goals from 300 to approximately 1,000 years. The curve for Pu-242 displays the lowest performance goals beyond 1,000 years.

(c) Proposed volume of water that can enter the waste disposal areas each year (see Figure 3-2).

(d) During this period it is assumed that containment is achieved and no release occurs. The hydrologic performance goal is selected to be that computed at 300 years.

release of C-14 from 4 Ci/1,000 MTHM to 1 Ci/1,000 MTHM. This reduction of 4 Ci/1,000 MTHM to 1 Ci/1,000 MTHM suggests that no more than 25% of the total air flow should exit from the shafts. This value of 25%, therefore, represents our performance goal for air flow out of shafts. No reduction of I-129 is required if we apply a similar logic to the release of I-129 because the potential release is less than 1 Ci.

It is recognized that the release of airborne radionuclides can be reduced by the emplacement of seals or backfill. To better understand the magnitude of reduction, air flow calculations are performed and are described in detail in Appendix C. Convective air flow models are evaluated in Appendix C. The first model, Mechanism A, assumes no flow through the tuff rock above the drifts. Therefore, all flow occurs only through the shafts, ramps, and drifts. The second model, Mechanism B, assumes that flow also can occur upward through the undamaged tuff rock. Primary conclusions reached from these calculations are synopsized below:

- Total, air-flow rates are dependent on the air conductivity of the shaft fill (Mechanisms A and B, see Table C-2) and the undamaged tuff (Mechanism B, see Figures C-3, C-4, C-5, and C-6).

- Little reduction in air flow rate is achieved by backfilling drifts. Filling shafts and ramps is most effective in reducing the air flow rate (Mechanism A, runs 1 - 3 compared with runs 4 - 6, Table C-2).

- When the shaft fill, air conductivity is less than 3×10^{-4} m/min, the percentage of flow through the shaft fill and the modified permeability zone (MPZ)* is always less than five percent of the flow through the rock above the repository drifts, regardless of the emplacement configuration or the presence of a modified permeability zone (Mechanism B, see Figures C-7, C-8, C-9, and C-10).

- When the air conductivity of the overlying tuff rock is high (i.e., for welded tuff 3×10^{-4} m/min and for the nonwelded tuff 3×10^{-5} m/min), varying the air conductivity of the shaft fill does not affect the total flow rate over a broad range of shaft fill air conductivities (Mechanism B, see Figures C-3, C-4, C-5, and C-6).

*The modified permeability zone is the zone immediately surrounding an underground excavation in which the permeability of the rock mass has been altered due to stress redistribution and blast damage effects.

- The presence of an MPZ around the shaft and ramp excavations does not significantly affect the flow rates out of the shafts (Mechanisms A and B, see Table C-2).

3.3 DEVELOPMENT OF THE PERFORMANCE GOAL FOR BOREHOLES

Boreholes could potentially provide pathways for radionuclide migration from the repository to the water table. As discussed in Section 6.7, as of May 1986, there are four existing boreholes within the proposed repository perimeter that penetrate to the water table. These boreholes provide a potential vertical pathway from the repository to the water table. Additional boreholes, many of which penetrate to the water table are located within 5 km of the repository perimeter. Among these holes, a potential pathway exists for those holes, which are downdip from the repository. Water that drains through the repository floor could become perched at a stratigraphic contact, flow downdip, and enter a borehole that penetrates through the stratigraphic contact to the water table. However, if a fully saturated condition does not exist, the borehole would act as a capillary barrier. This conclusion is reached based on the analyses of a shaft in the unsaturated zone that penetrates a welded-nonwelded tuff interface.

Generally, boreholes that occur to the east and northeast of the repository are more likely to act as pathways, whereas boreholes that occur to the west of the repository, as well as probably those to the south and north, cannot act as pathways for radionuclide migration (Section 6.7). This is true because water passing through a stratigraphic discontinuity will pass through the contact or flow downgradient along the contact. At Yucca Mountain, the contours defined by stratigraphic contacts indicate a dip or direction of flow generally to the east or northeast.

Performance goals and requirements for borehole seals are developed from the NRC 10 CFR 60 requirement (Section 1.2) that "boreholes shall be designed so that following closure they do not become pathways that compromise the geologic repository's ability to meet the performance objectives." We adopt the position that this requirement is satisfied if the potential for vertical flow through boreholes is only one percent or less of the potential for vertical flow through the entire rock mass over which lateral flow along stratigraphic contacts is assumed to occur. In Section 6.7, we develop a performance requirement by considering the potential total flow through all boreholes within 5 km of the repository (for various areas on the eastern side of the repository) and the potential total flow through the area of the rock mass also extending to 5 km, under the same unit gradient.

4.0 ESTIMATES OF WATER FLOW INTO SHAFTS AND THE UNDERGROUND FACILITY

In this chapter, both anticipated and unanticipated conditions are postulated for water flow into shafts and the underground facility. Because hydrologic data are limited, engineering judgment is used in defining the conditions that are evaluated. Anticipated hydrologic conditions in this report include (1) matrix flow over the entire repository area, (2) annual, limited, and localized fracture flow, and (3) limited, surface flow into shafts. Precipitation events sufficient to cause sheet flow, i.e., flooding events, are assumed to occur annually to introduce water into the fractures and shafts. The durations of these events are limited. Unanticipated hydrologic conditions include (1) continuous fracture and matrix flows over the entire repository area and (2) extensive surface flow into shafts from major flooding events. The duration of these flooding events is limited only by the supply of water at the shaft location.

Descriptions of these conditions are provided in this chapter. To obtain estimates of water flow into shafts/ramps and the underground facility analytical solutions, together with engineering judgments, are used. Additionally, the horizontal and vertical emplacement configurations are discussed for both flow conditions.

4.1 WATER FLOW INTO REPOSITORY SHAFTS

Potentially, the shafts could act as conduits for surface runoff or ground water to enter the repository. The scenario is that if a large volume of water were to enter a shaft, the sump would fill and the excess flow would enter the drift (or drifts) connecting the shaft to the repository. Flow within the repository would then be determined by the room layout, by the grade of the drifts, by drainage through the floors of the drifts, and by any measures taken to impede or divert flow. Figures 2-6 and 2-7 show the locations of the four shafts in the present conceptual design for the repository. All of the shafts are located in the northeastern part of the repository. The Emplacement Exhaust Shaft (EES) intersects the repository at its lowest point; any water in the repository could drain eventually to this point, if it does not drain through the floor.

4.1.1 Surface-Water Inflow

During operation of the repository, flood-control structures illustrated in Section A-1.1 will be designed to prevent surface water from entering the shafts. After decommissioning, however, the effectiveness of the flood-control structures would probably diminish. Therefore, over the long term (i.e., 10,000 years) water flow into the upper portion of a shaft would be potentially limited by (1) the volume of surface water, (2) the topography in the vicinity of the shaft entry point, (3) the hydraulic conductivity of sealing components placed in the shaft, (4) the hydraulic properties of the alluvium and Tiva Canyon Member, and (5) the extent and nature of the modified

permeability zone (MPZ) adjacent to the shaft wall. These factors are discussed below to define the assumptions for anticipated and unanticipated flow conditions and arrive at estimates for flow into shafts.

All of the proposed shaft locations are in valleys near washes (Figure 2-2) that may be subject to occasional flooding during heavy rainfall. Such flooding constitutes a source of water that could enter the shafts. Computations of the total runoff for various flooding events are presented in Appendix A-1. Existing formulas, techniques and various assumptions are used to compute flood volumes for the 100-year, 500-year, and probable maximum floods (PMFs) (Table 4-1). The purpose in calculating multiple flood events is to present a comparison of the different flood volumes and to evaluate by using different techniques the reasonableness of the values obtained. As expected, the total flows associated with the 100-year, 500-year, and the PMFs are progressively larger. The PMF is computed using the procedure given in ANSI/ANS-2.8-1981 (ANSI, 1981) and represents the largest flood anticipated at a specified location.

The second item affecting water entry into the shafts is the role of topography. Three scenarios involving the role of topography are discussed below: (1) debris slides, or flood debris deposition blocking water flow down the wash, (2) subsidence of shaft fill and inward collapse around the upper portion of the shaft, and (3) periodic sheet flow over the shaft entry point.

The first scenario would involve a large debris slide downstream from the shaft location, or erosion of alluvium around the shaft (perhaps enhanced by settlement, as described below), and deposition downstream, forming a dam across the wash. At the Exploratory Shaft (ES-1) location, the width of the valley containing the wash is about 107 m*, while at the Men and Materials Shaft (MMS), the valley width is about 244 m, and at the EES, it is about 107 m (Appendix A-1). At all three locations, a landslide large enough to completely block the valley and impound the total flow in the wash is not credible given the thin cover of colluvium and weathered rock on the adjacent slopes. For example, for the ES-1, a dam would have to be about 12 m high across the 107 m wash to impound the peak 500-year discharge of 24 m^3/s for one hour. At Yucca Mountain there is at present no evidence that surface impoundments, of the size needed to contain this flood volume, and formed by landslides are in existence (DOE, 1986, p. 6-232). A more probable scenario

*At the conclusion of this study the ES-1 and ES-2 locations were moved to the northeast of the locations indicated in the final Environmental Assessment for Yucca Mountain. The proposed locations for ES-1 and ES-2 are shown in Figure A-1.1. Because the shafts have been relocated to areas that are out of the wash and because both shafts will be collared in bedrock, the impact that erosion and flooding could have on water entering the shafts is substantially reduced at these exploratory shaft locations.

Table 4-1. Flood Volumes Associated With Various Flooding Events

Shaft	100-yr Flood		500-yr Flood		1-Hour Thunderstorm[d]		PMF, 16 hr	PMF, 14 hr
	Peak[b] Discharge, m^3/s (ft^3/s)	Total[c] Flow, m^3 (ft^3)	Peak[b] Discharge, m^3/s (ft^3/s)	Total[c] Flow, m^3 (ft^3)	Peak Discharge, m^3/s (ft^3/s)	Total Flow, m^3 (ft^3)	Duration[a] Total Flow, m^3 (acre-ft)	Duration[a] Total Flow, m^3 (acre-ft)
Exploratory Shafts[e]	5.3 (190)	4,800 (170,000)	24 (850)	22,000 (770,000)	22 (750)	55,000 (1,960,000)	159,000 (129)	93,000 (75)
Men and Materials	7.1 (250)	6,400 (230,000)	32 (1,130)	29,000 (1,014,000)	37 (1,320)	99,000 (3,500,000)	226,000 (183)	132,000 (107)
Emplacement Exhaust	3.1 (110)	2,800 (97,000)	14 (480)	12,000 (434,000)	9.9 (350)	22,000 (780,000)	51,000 (41)	30,000 (24)

(a) Probable maximum flood (PMF) value taken from Bullard (1986) and 13.8 inches for a 6-hour storm and 8.5 inches for a 14-hour storm.
(b) Peak discharge calculated from Squires and Young (1984).
(c) Total flow computed by taking peak discharge values and scaling flood hydrograph from Eldorado Flood hydrograph (Glancy and Harmsen, 1975, p. 15).
(d) Computed using the SCS unitgraph method as described in Appendix A-1 and precipitation of 7 inches in one hour.
(e) Note that ES-1 and ES-2 are in the same drainage basin.

is that of a small obstruction blocking a portion of the wash, slowing down the flow and diverting runoff into the alluvium or fill surrounding the shaft.

The second scenario would involve subsidence adjacent to the shaft or settlement within the shaft, resulting in inward collapse of the upper part of the shaft and formation of a depression. In the conceptual design for the repository, a network of drifts housing decontamination and emplacement equipment is excavated immediately to the south of the ES-1 at the repository horizon (Figures 2-5 and 2-6). A similar network of drifts housing maintenance facilities is located near the MMS. The extraction ratio in these areas (about 33%) may be high enough to provide the potential for collapse and subsidence over the long term. The amount of subsidence would be small, however, particularly if the rooms are backfilled. The current conceptual design for the repository includes plans for backfilling all drifts during decommissioning. If the underground workings are backfilled, a more likely mechanism for collapse around the shaft would involve settlement of the shaft fill and inward collapse of only the upper part of the shaft, particularly if no measures are taken to control consolidation of the shaft fill. If the settlement were 15 m, equivalent to 3.4% over 451 m depth, the area affected would be about 11 m in diameter, assuming an angle of draw of 45°. The volume created within the depression would be about 160 m^3.

The third scenario would involve periodic sheet flow over the entry of the shaft. In this scenario, water entry into the shaft would be governed by the water available to infiltrate the shaft fill which in turn is dependent on the degree to which shaft flow exceeds the ground infiltration capacity upstream of the shaft. For certain hydrologic events, saturation of the ground upstream may not occur, and the shaft fill would not likely saturate by direct precipitation; in other instances, sheet flow may develop in which case the water depth is expected to be small (less than 10 cm). Under these anticipated conditions, the water available for direct infiltration due to sheet flow is expected to be low.

The preceding discussion has been presented to demonstrate that water can potentially enter the shafts under extreme rainfall conditions. In the short term, the likelihood of these occurrences will be greatly reduced by appropriate, surface flood control features, but over hundreds of years, these structures could be destroyed by weathering or erosion. Even in the long term, water infiltration into the shaft will probably be low due to the limited duration and frequency of major precipitation events and, as previously discussed, the low likelihood of impounding large amounts of water within the washes.

4.1.2 Ground-Water Inflow

Previous work by Fernandez and Freshley (1984) has shown that ground-water inflow to a shaft will be insignificant provided that unsaturated conditions prevail. The conditions discussed below are inflow from the rock matrix,

perching of ground water in the alluvium or at a lower stratigraphic contact, and inflow from a discrete fault that intersects the shaft.

4.1.2.1 Inflow From the Rock Matrix

Fernandez and Freshley (1984, p. 35) and Freshley et al. (1985, p. 62) analyzed matrix flow around a shaft using TRUST, a computer program for variably saturated flow in a multidimensional, deformable media. The calculations were performed assuming ambient thermal conditions.

One of the primary objectives was to evaluate whether or not a perched water table condition might occur at a stratigraphic boundary between welded and nonwelded tuff. A second objective was to study the effects of clay and sand shaft fill on the flow system around the shaft. Five cases were analyzed: three having welded tuff overlying nonwelded tuff and two having nonwelded overlying welded tuff.

When sand was used for shaft fill, the calculated matrix inflow to the shaft was approximately 2×10^{-14} m^3/s per meter of depth. For clay, the inflow was approximately 10^{-10} m^3/s per meter of depth. The stratigraphic sequence did not affect the inflow rates. It was postulated that perched water might form at the contact between welded and nonwelded tuff because of the large contrast in conductivities and porosities. Some transient buildup of water (i.e., saturation) at the contact was simulated, but it dissipated as the steady-state flow system was approached.

4.1.2.2 Inflow Under Perched Water Conditions

The expected hydrologic conditions at Yucca Mountain are that the rock mass will remain unsaturated under an average ground-water flux of less than 0.1 mm/yr (Sinnock et al., 1984, p. 52). As described in the previous section, under unsaturated conditions the ground-water inflow to a shaft will be very small. Higher inflows to a shaft could occur if local saturation, i.e., perched water conditions, could develop in the vicinity of a shaft. Theoretically, perching can occur where downward infiltration is impeded by permeability contrasts or capillary barriers. Simple analyses presented in Appendix A-2 show that perching is theoretically possible at the alluvium-bedrock contact near a shaft located in a wash following heavy rainfall and flow in the wash if the hydraulic conductivity of the alluvium is high. Perching is also theoretically possible at the contact between the Tiva Canyon and the underlying, less permeable, Paintbrush nonwelded unit.

There is no conclusive evidence that perched water currently exists at Yucca Mountain. Water was encountered above the water table in boreholes USW UZ-1 and USW H-1. In USW UZ-1, water was encountered in the Topopah Spring welded unit and was believed to be the drilling fluid used to drill an adjacent well USW G-1; however, no determination was made concerning the presence of naturally occurring waters (DOE, 1986, p. 6-139). In USW H-1, water seepage into

the borehole was observed in the Topopah Spring welded unit. Because water was used to drill the borehole, and no water samples were taken to determine whether these water seeps were drilling fluids or naturally occurring waters, a determination of the presence of perched waters in the Topopah Spring Member could not be made (DOE, 1986, pp. 6-139 and 6-140). Currently no evidence exists for the presence of perched water above the Topopah Spring Member. Observations are planned during Exploratory Shaft Facility construction and operation to evaluate the occurrence of water inflows.

4.1.2.3 Inflow From a Discrete Fault

Higher inflows than those calculated by Fernandez and Freshley (1984) could possibly occur if a shaft was intersected by a high-permeability fault or fracture zone. It is noted, however, that no significant fault was found in borehole USW G-4 (Bentley, 1984) near the ES-1 location. Moreover, none of the other shafts is located near a major fault (Section 2.1). It is considered unlikely, therefore, that inflows would occur into shafts from a major fault. Minor faulting, however, does occur throughout the Yucca Mountain area. The hydrologic significance of faults in general will be investigated through testing associated with the Exploratory Shaft Facility.

4.1.3 Assumptions for Water Flow Into Shafts

Two scenarios are selected to represent anticipated and unanticipated flows to the shaft. These scenarios include:

- periodic sheet flow over shaft entry points for the anticipated water flow condition (third scenario in Section 4.1.1), and

- restriction of all flood waters in the alluvium above the shaft locations for the unanticipated water flow condition (first scenario in Section 4.1.1).

The assumptions used in evaluation of flow under anticipated and unanticipated conditions are given in Table 4-2.

For the anticipated scenario, it is assumed that no restriction of flood waters in the alluvium occurs. Therefore, a perched water system does not form that would result in radial flow towards the shaft. Any fault systems that intersect the shaft are assumed to be dry.

For the unanticipated scenario, it is assumed that flood water would be partially impeded from flowing down the wash, and that the alluvium would be saturated under perched conditions near the shaft. The scenario does not assume a massive impoundment of flood volumes, but it is assumed that the storage capacity in the vicinity of the shaft location is sufficient to retain

Table 4-2. Assumptions Made for Anticipated and Unanticipated, Yearly Water Flow Into Shafts

Anticipated Water Flow	Unanticipated Water Flow
• No restriction of flood waters in alluvium at shaft locations, i.e., no perched water system at surface.	• Restriction of all flood waters in alluvium at shaft locations, i.e., perched water system at surface.
• Shafts filled with a granular material having a saturated hydraulic conductivity of 10^{-2} cm/s.	• Shafts filled with a granular material having a saturated hydraulic conductivity of 10^{-2} cm/s.
• No seals in shafts.	• No seals in shafts.
• Water supply defined by four thunderstorms >1.3 cm (0.5 in.).	• Water supply defined by PMF volume and 500-year flood volume.
• Duration for each thunderstorm is 1 hour. Sheet flow over shaft and fault locations lasts for one hour. Water depth of sheetflow is 1.25 ft.	• No restriction on the duration of flow into shafts.

all the flood volume from selected flood events. The alluvium is assumed to become instantaneously saturated following the flood event.

The flow under unanticipated conditions is assumed to occur in one of three ways; these include vertical infiltration into the bedrock, lateral flow down gradient through the alluvium, or radial flow towards the shaft. To provide conservatism, the water is assumed to be isolated from the surface evapotranspiration zone and evapotranspiration is neglected. As discussed in Section 4.1.5, the amount of water entering the shaft is dependent on the hydrologic properties of the near-surface geologic units, the shaft components, and the MPZ associated with the shaft.

4.1.4 Estimates of Water Flow Into Shafts Under Anticipated Conditions

As discussed in the previous section, the water entering the shafts under anticipated conditions is expected to occur from periodic sheet flow over the shaft entry points. Water supply is controlled by duration and frequency of the surface flooding event.

Typically, the flow rate during a flash flood in a semiarid environment will rise and decrease rapidly, with the peak flow period having a short duration (Glancy and Harmsen, 1975, p. 15). In our analysis, the duration of flow corresponding to the duration of infiltration is assumed to be approximately 1 hour, corresponding to the duration of a probable thunderstorm (U.S.B.R., 1974, p. 52). The frequency is controlled by the number of precipitation events having sufficient precipitation to cause surface runoff. Using limited field observations by the U.S.G.S. in the Yucca Mountain area, Bullard, 1986 concluded that "one to two inches of rainfall is required before any runoff occurs". For the purpose of this report, it is conservatively assumed that runoff will occur if the precipitation amount is greater than 1.3 cm (0.5 in.). A recently published report summarizing meteorological information in the Yucca Mountain area (Eglington and Dreicer, 1984, pp. 1-16, 1-17) indicated, for the climatological records between 1962 and 1971 for Yucca Flat, Nevada, and between June 1978 and May 1983 for Desert Rock, Nevada, that on the average four thunderstorms having a precipitation greater than 1.3 cm (0.5 in.) occur per year. Therefore, it is assumed that infiltration into the shaft fill occurs four times during the year and the duration for each event is 1 hour.

To compute the amount of water entering the shaft, a simple method for calculating vertical infiltration, i.e., the Green and Ampt approach (Hillel, 1971, pp. 140-143), is employed. Details of this approach are given in Appendix A-3. For a shaft fill having a saturated hydraulic conductivity of 10^{-2} cm/s, the depth of penetration into an initially dry, shaft fill over a 1-hour period is 2.4 m. The total amount of water entering all shafts is given in Table 4-3 for an assumed porosity of 0.3 for the shaft fill. For shaft fills having low hydraulic conductivities (less than or equal to 10^{-2} cm/s) the initial flow rate computed using the Green and Ampt approach is

Table 4-3. Yearly Inflow Through All Shafts Under Anticipated Conditions

	Units	Shafts			
		Emplacement Exhaust	Men and Materials	Exploratory Shaft (ES-1)	Exploratory Shaft (ES-2)
Finished Diameter	m (ft)	6.1 (20)	6.1 (20)	3.7 (12)	1.8 (6)
Nominal Excavated Diameter	m (ft)	6.86 (22.5)	6.86 (22.5)	4.42 (14.5)	2.4 (7.9)
Water Entry Into Shaft					
Green and Ampt Solution[a]	m^3 (ft^3)	106.0 (3,740)	106.0 (3,740)	44.2 (1,560)	13.4 (470)
Steady-State Flow[b]	m^3 (ft^3)	53.1 (1,880)	53.1 (1,880)	22.1 (780)	6.7 (240)

[a] Assumes (1) a porosity of 0.3 for shaft fill and zero initial saturation, (2) four events sufficiently large to create free-flowing water at surface, (3) each event is 1 hour in duration at the surface, and (4) -100 cm H_2O initial pressure head at the wetting front. Using these assumptions and the Green and Ampt solution, the saturation front advances 2.4 m.

[b] Darcy flow assuming a unit gradient with a hydraulic conductivity of the shaft fill of 10^{-2} cm/s (presented for comparison purposes only).

higher than that computed by a steady-state approach. In this sense, a greater amount of water will be computed using the Green and Ampt approach. The total flow into the shafts calculated using the Green and Ampt, and steady-state approaches is presented in Table 4-3.

4.1.5 Estimates of Water Flow Into Shafts Under Unanticipated Conditions

In arriving at the flow into shafts, it is assumed that all of the waters from selected, major-flood events are restricted in the alluvial basins adjacent to each shaft, allowing water to drain into the shafts (see Section 4.1.3, Table 4-2). The approach used to compute flows into the shafts is described below.

Regardless of surface processes, water entry into shafts can be controlled by the hydrologic nature of the near-surface geologic units and the shaft sealing components. Because there can be a broad range in the hydrologic properties of the geologic units, this range should be considered in assessing the volume of water entering the shaft. For example, the saturated, bulk hydraulic conductivity of the Tiva Canyon Member is assumed to range from 10^{-2} to 10^{-5} cm/s (Table 2-2). Values for alluvium could range from 10^{-5} to 100 cm/s (Freeze and Cherry, 1979, p. 29).

To arrive at an upper bound of water that could potentially enter the underground facility through the shafts, a flow model was developed. A computer program was developed to facilitate the numerical calculations. To aid in conservatism, the flow model assumed that all of the water from a flooding event would be available for drainage into the shafts. The total amount of the flow into the shaft would be controlled by several types of flow, as well as the geometry used in the model, as described below.

The geometry of the model is illustrated in Figure 4-1. Alluvium overlies the welded, highly fractured Tiva Canyon Member. The upper portion of the shaft through the alluvium is modeled as being filled with a coarse fill to minimize restriction of flow into the shaft. The lower portion of the shaft is modeled as containing fill having a saturated hydraulic conductivity of 10^{-2} cm/s, extending to the outside diameter of the shaft. (In reality, a shaft liner, potentially having a lower hydraulic conductivity than the shaft fill, may remain in place. By ignoring the presence of the shaft liner, a higher flow through the shaft is computed.) The modified permeability zone (MPZ) model extends out one radius from the shaft wall. Two cases for the MPZ are considered in which the bulk rock, hydraulic conductivity is either 20 or 60 times the undisturbed, hydraulic conductivity of the Tiva Canyon Member (Case and Kelsall, 1987, p. 59). The selection of these values for the MPZ is discussed later. Flow progresses in three phases: an initial desaturation phase, a steady-state phase, and a final desaturation phase. These phases of flow are schematically illustrated in Figure 4-2. Prior to initiation of Phase I, it is assumed that the alluvium becomes fully saturated and the water in the shaft above the alluvium-Tiva Canyon contact enters the lower portion of the shaft. Desaturation of the alluvium occurs first at curve "1" and

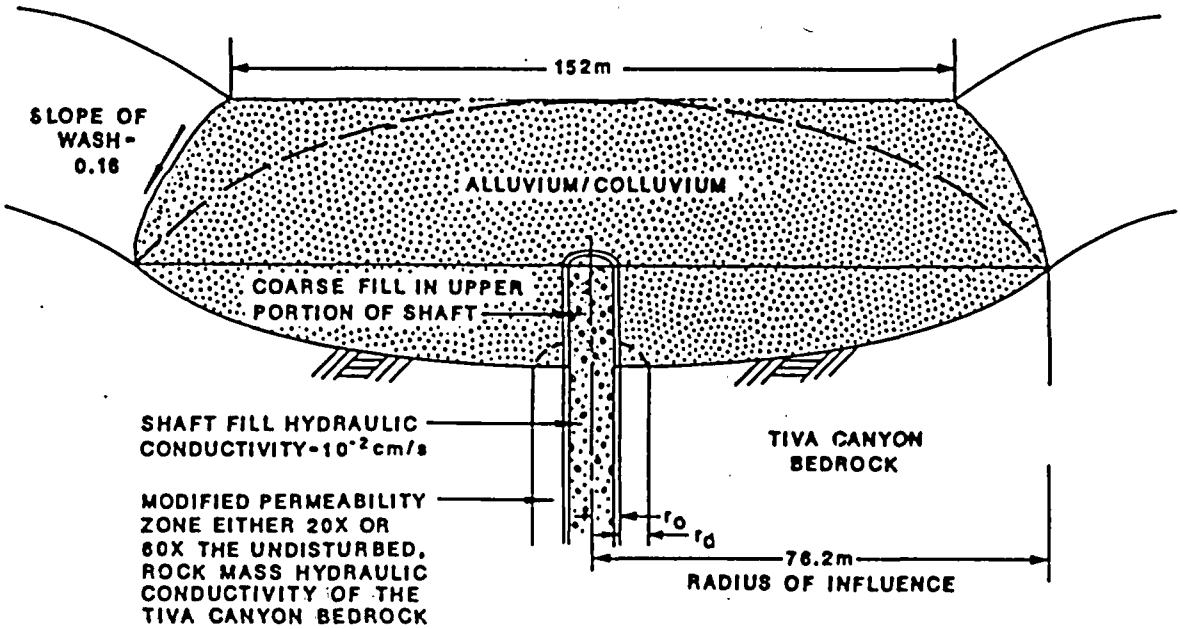


Figure 4-1. Geometry of Model Used to Estimate Flow Into a Shaft From Saturated Alluvium

progressively to curve "n" (Figure 4-2a). As the radius of influence is changed in response to desaturation, the radius of influence associated with curve "n" represents steady-state conditions that are held constant until the supply of water replenishing the alluvium no longer exists (Figure 4-2b). As Phase III begins, the only water remaining is that water contained under curve "n." Desaturation then proceeds from curve "n" to curve "m."

During each phase of drainage, four types of flow are considered: unconfined radial flow under the Dupuit flow assumption, alluvial flow, Tiva Canyon flow, and flow through the MPZ and the shaft fill. Each of the flows are discussed below.

Radial flow is computed using the following equation:

$$Q_s = \frac{\pi K \, (H^2 - H_o^2)}{\ln(\frac{R}{r_o})} \qquad (4\text{-}1)$$

where

R = radius of influence,
Q_s = flow rate into the shaft,
K = hydraulic conductivity,
H = piezometric level at radius R,
H_o = piezometric level at radius r_o, and
r_o = shaft radius.

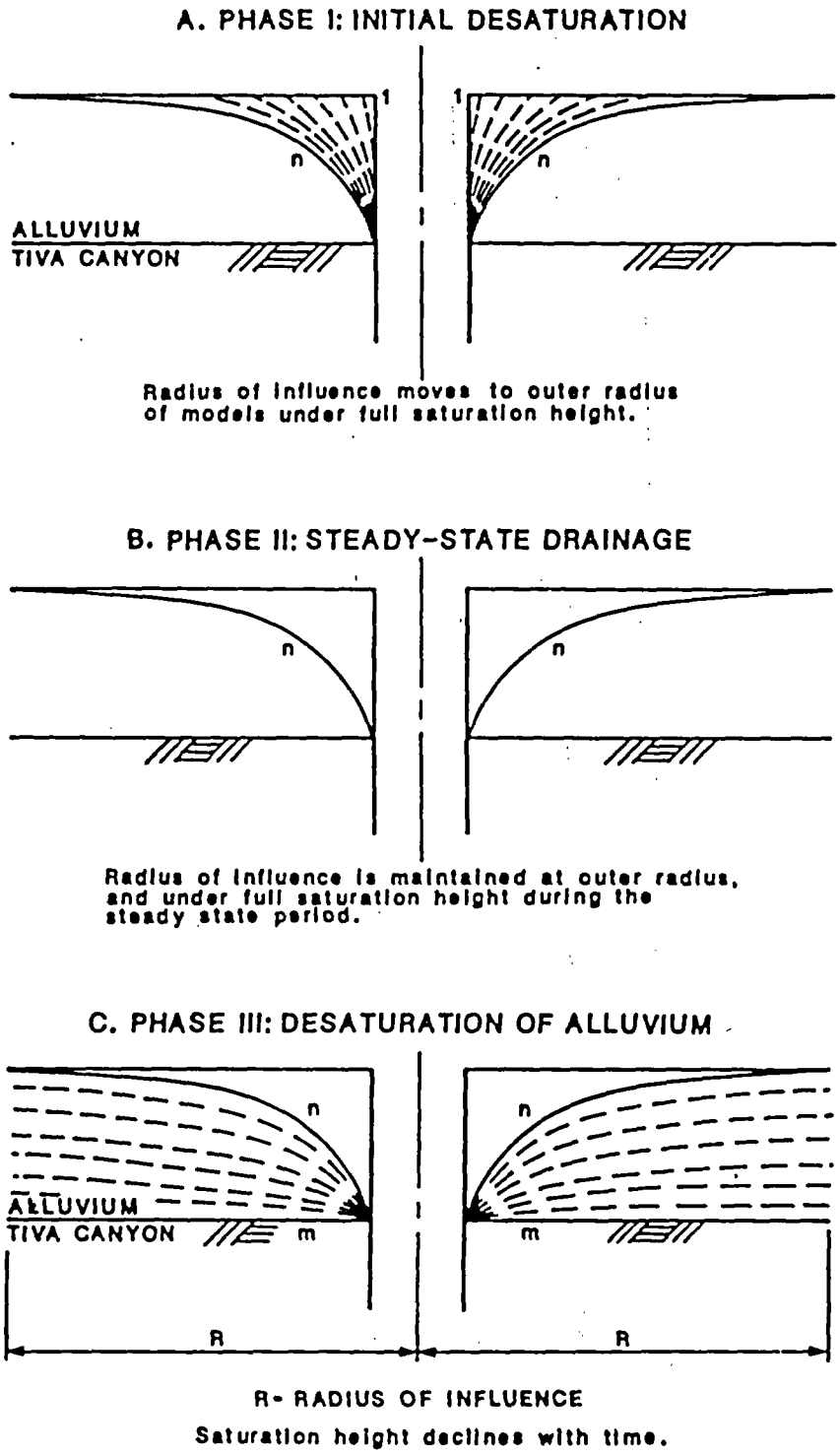


Figure 4-2. Phases of Flow for Flow Into a Shaft From Saturated Alluvium

This equation taken from Todd (1980, p. 118) assumes steady-state flow in the horizontal direction under unconfined conditions. Radial flow is illustrated in Figure 4-3a.

Alluvial flow is computed assuming Darcy's law, as follows:

$$Q = KiA \tag{4-2}$$

where

- Q = flow,
- K = hydraulic conductivity of alluvium,
- i = alluvial grade, and
- A = cross-sectional area through which flow occurs.

Alluvial flow is schematically illustrated in Figure 4-3b. Alluvial flow occurs within the crosshatched area and occurs parallel (or nearly parallel) with the surface between the alluvium and the Tiva Canyon Member.

Tiva Canyon flow is computed assuming Darcian flow and a unit gradient over the entire flow area. This type of flow is illustrated in Figure 4-3c and occurs through the crosshatched area. The vertical infiltration through the Tiva Canyon unit is assumed to occur through the crosshatched area under a unit gradient as might occur for fractured rock that is nearly saturated. It is recognized that the bedrock is unsaturated and that infiltration rates are likely to be higher; nevertheless, the flow calculation is conservative in underestimating this component of flow (greater proportion of flow is directed to the shaft).

Flow through the MPZ and shaft fill is also be computed assuming Darcian flow and a unit gradient. Note that for material within the interior portion of the shaft, the degree to which infiltration would occur at unit gradient depends on the level of saturation, and initially, the hydraulic gradient could exceed unity. These high infiltration rates would be associated with the saturation of voids and not transmission of water to the base of the shaft. As the infiltration front reaches the base of the shaft at which point water could potentially enter the repository, the hydraulic gradient would be approximately one.* Two models are considered for the hydraulic conductivity of the MPZ, as discussed below.

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*This can be shown by the Green and Ampt solution for vertical infiltration. At the base of the shaft, the hydraulic gradient is given by $1 + [(H_o - H_f)/L_f]$ where H_o equals the pressure head at the surface, H_f equals suction head at wetting front, and L_f equals the length over which the wetting front has moved. If we assume the pressure head at the surface is 9.1 m (height of saturated alluvium above bedrock), the suction head for the backfill is -1.0 m (a typical value for coarse material) and the length over which the wetting front has moved (311 m), then the calculated hydraulic gradient is nearly one.

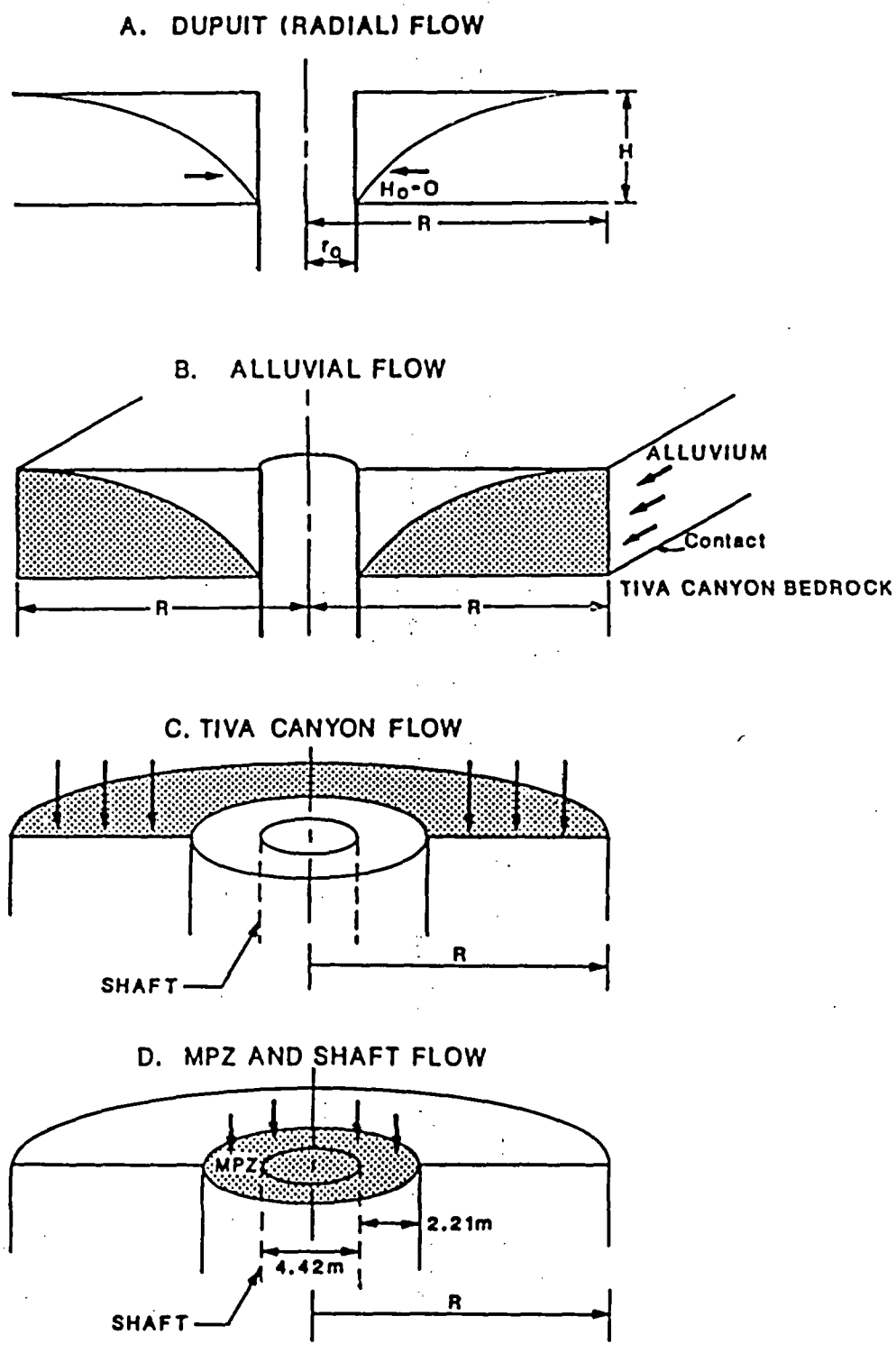

Figure 4-3. Types of Flow Considered in Estimating Flow Into a Shaft

These flows are superimposed such that flow can occur as Tiva Canyon flow, alluvial flow, or shaft flow. Therefore, as a volume of water is computed for each portion of each phase, flow occurs proportionately through the Tiva Canyon Member, alluvium, or the shaft as determined by the flow rate computed for each. Flow through the shaft is either the amount computed using the radial formula or the amount computed by the MPZ and shaft fill model, whichever amount is lower. The entire process of desaturation continues until the supply of water is depleted. The potential supply of water is assumed to be the waters associated with specific flood events.

To maximize the amount of inflow into the shaft, it is assumed that retainment of the entire amount of water associated with a flooding event occurs above each shaft location. This implies that the alluvium has a sufficient storage volume to retain all the water from the flood event. This retention is an overly conservative assumption that involves no losses by evapotranspiration or sheet wash downgradient from the shaft locations. In reality, a high percentage of the precipitation will exit the drainage basin, with only a small part percolating into the alluvium or exposed bedrock. Further, it is assumed that water flow is directed vertically downward inside the shaft liner or in the shaft fill as the water percolates to the base of the shaft. It is further assumed that flow occurs through fractures within the MPZ and that water is not absorbed within the tuff matrix. In computing the flows into shafts, the PMF and 500-year flood volumes are considered. These flood volumes are presented in Table 4-1, Section 4.1.1.

In applying this model, it was necessary to develop assumptions and evaluate specific conditions for water flow. The following assumptions were used in applying the model:

- The MMS and EES have a radius of 3 m at the surface and the ES-1 has a radius of 1.83 m at the surface.
- The MMS and EES each have a radius of 3.4 m in the Tiva Canyon and the ES-1 has a radius of 2.21 m in the Tiva Canyon*.
- The thickness of the MPZ in the Tiva Canyon Member is one radius and extends from the shaft wall.
- Hydraulic conductivity of the alluvium varies from 10^{-5} cm/s to 100 cm/s (Freeze and Cherry, 1979, pp. 29, 147).

*The selection of a larger radius in the Tiva Canyon was made to allow more flow to enter the shaft and MPZ model. Therefore, this assumption is more conservative.

- Hydraulic conductivity of the Tiva Canyon Member varies from 10^{-5} cm/s to 10^{-2} cm/s (Table 2-2).

- Alluvial grade at each site is 0.16 (based on alluvial grade in Coyote Wash).

- Radius of influence is 76.2 m (based on a typical width of alluvium at shaft locations, see Appendix A-1).

- Depth of alluvium is 9.1 m (based on depth of alluvium at borehole USW G-4).

- Porosity of alluvium is 0.30 (Appendix D).

The models for the extent and hydraulic conductivity of the MPZ are obtained from Case and Kelsall (1987, p. 59). These models considered two locations in the Topopah Spring Member, one located 100 m from the surface and another 310 m from the surface. Expected conditions for strength, stress, and sensitivity of permeability to stress were evaluated. Also, upper-bound conditions were evaluated. These conditions included lower-bound strength, upper-bound in situ stress, and greatest sensitivity of permeability to stress. MPZ models also included models without blast damage, a 0.5-m-wide, blast damage area, and a 1-m-wide, blast damage area.

Two MPZ models were selected for evaluation. The first MPZ model represents the expected conditions at 100- and 310-m depths and has a 0.5-m-wide, blast damage area. The bulk, saturated hydraulic conductivity of this model is 20 times the undisturbed, bulk, hydraulic conductivity of the Tiva Canyon Member. A second MPZ model represents the upper-bound condition and has a 1-m-wide, blast damage area. The bulk, saturated hydraulic conductivity of this model is 60 times the undisturbed, bulk, hydraulic conductivity of the Tiva Canyon Member. This value of 60 is the average of two values, 40 and 80, associated with MPZ models at 100- and 310-m depths. Because the Tiva Canyon Member is a densely welded, highly fractured tuff, the strength values from the Topopah Spring Member are considered applicable. Further, because the Tiva Canyon Member is located closer to the surface, the in situ stress state is less severe than for either the 100- or 310-m depth models. Therefore, application of the MPZ models to the Tiva Canyon Member is considered conservative in that more flow can pass through the shaft.

The preceding discussion addresses the model used, the volume of water associated with each shaft site, and the assumptions made in applying the model. Three conditions were evaluated:

1. A volume of water equal to the PMF at each shaft site is assumed, and the saturated hydraulic conductivity of the shaft fill is 10^{-2} cm/s.

2. A volume of water equal to the PMF at each shaft site is assumed, and the saturated hydraulic conductivity of the shaft fill is 10^{-6} cm/s.

3. A volume of water equal to the 500-year flood at each shaft site, and the saturated hydraulic conductivity for the shaft fill is 10^{-2} cm/s.

The results for each condition are illustrated in Figures 4-4, 4-5, and 4-6, respectively. A broad range of flows computed by the model are presented above. For the PMF conditions evaluated, the inflow into shafts can range from approximately 200 to 83,700 m^3. The influence of shaft fill can be evaluated by observing differences between Figures 4-4 and 4-5. As might be expected, the influence of a lower hydraulic conductivity for the shaft fill is greater when the Tiva Canyon Member hydraulic conductivity is lower. This is true because the flow through the shaft fill is proportionately greater when the hydraulic conductivity of the MPZ is low. To illustrate this relationship, the percentage of flow through shaft fill as a function of the hydraulic conductivities of the Tiva Canyon Member and the shaft fill is presented in Figure 4-7. Using the relationship presented in this figure, the results displayed on Figures 4-4 and 4-5 can be contrasted. When the Tiva Canyon Member has a hydraulic conductivity of 10^{-2} cm/s, the percentage of flow through shaft fills, having hydraulic conductivities of either 10^{-2} cm/s or 10^{-6} cm/s is low (see Figure 4-7). Therefore, the effect on the total flow into the shafts is small, and the flow occurs mainly through the MPZ. Consequently, the annual flow into all the shafts is nearly identical for the cases where the shaft fill, hydraulic conductivity is 10^{-2} cm/s (Figure 4-4) or 10^{-6} cm/s (Figure 4-5). However, when the hydraulic conductivity of the Tiva Canyon Member is 10^{-5} cm/s, the percentage of total flow through the shaft fill is considerably different. Flow through the shaft fill having a hydraulic conductivity of 10^{-2} cm/s represents approximately 95% and 85% (depending on the assumption of the MPZ) of the total flow into the shaft, whereas the percentage of flow through the shaft fill is extremely low for a fill hydraulic conductivity of 10^{-6} cm/s. This suggests that the shaft fill can control water flow to the base of the shaft under some circumstances. This reduction is noticed by contrasting Figures 4-4 and 4-5. When the hydraulic conductivity of the Tiva Canyon Member is either 10^{-5} or 10^{-4} cm/s, differences in total flow into shafts are noted. An additional parameter that can be contrasted between Figures 4-4 and 4-5 is the maximum flow computed for both MPZ model assumptions. When the shaft hydraulic conductivity is 10^{-2} cm/s, the computed maximum flows are 59,000 and 83,700 m^3/year for the 20X MPZ and 60X MPZ models, respectively. When the shaft hydraulic conductivity is 10^{-6} cm/s, the computed maximum flows are 22,900 and 45,900 m^3/year for the 20X MPZ and 60X MPZ models. This indicates that decreasing the hydraulic conductivity of the shaft fill by four orders of magnitude, achieves less than a one order of magnitude decrease in the maximum flow into the shafts. For each MPZ model, some reduction of flow is observed, as well as a reduction of flow that can be achieved by reducing the hydraulic conductivity of the shaft fill.

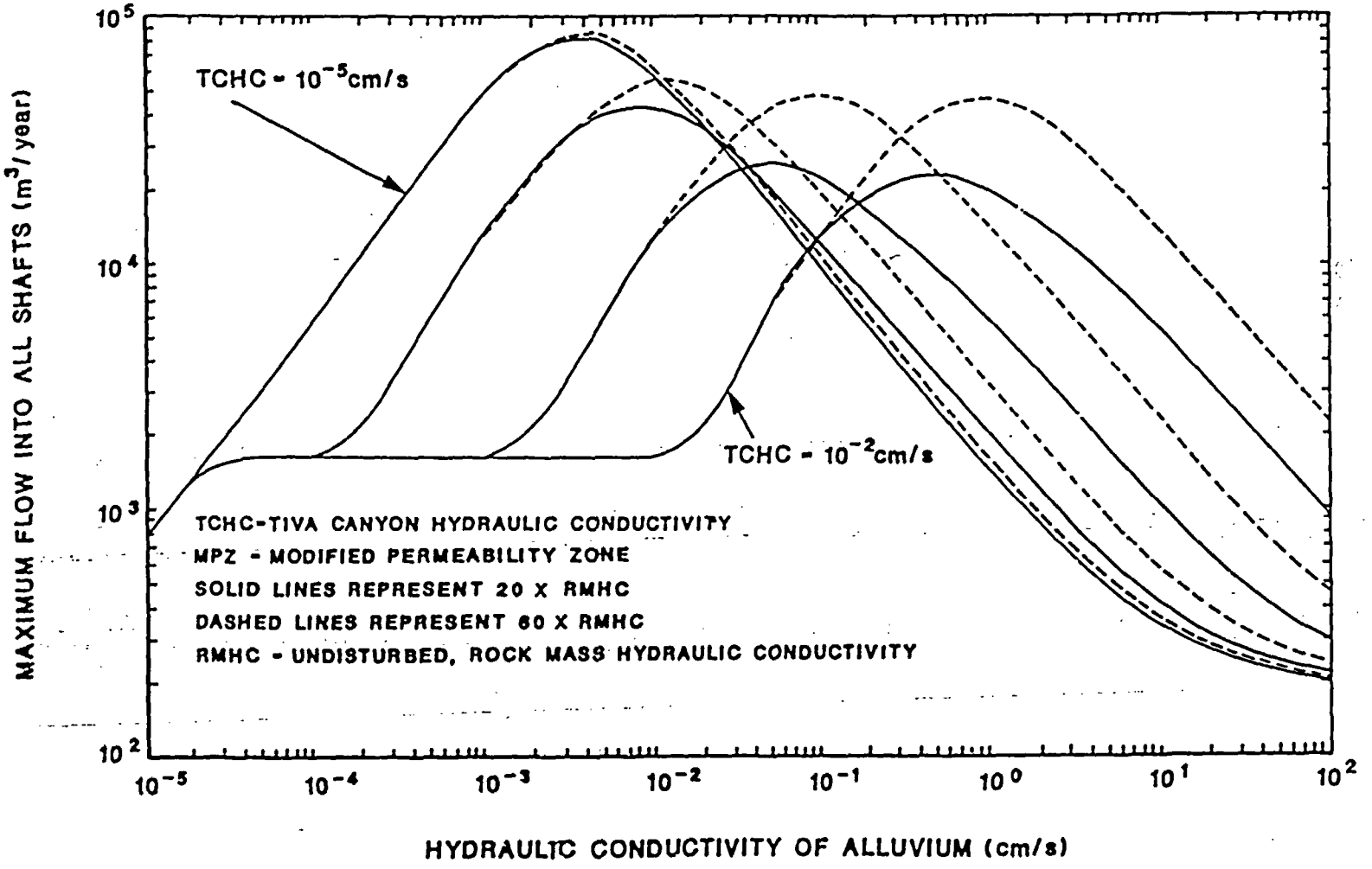

Figure 4-4. Estimated Volume of Surface Water Entering a Shaft (PMF; Shaft Fill, Hydraulic Conductivity = 10^{-2} cm/s)

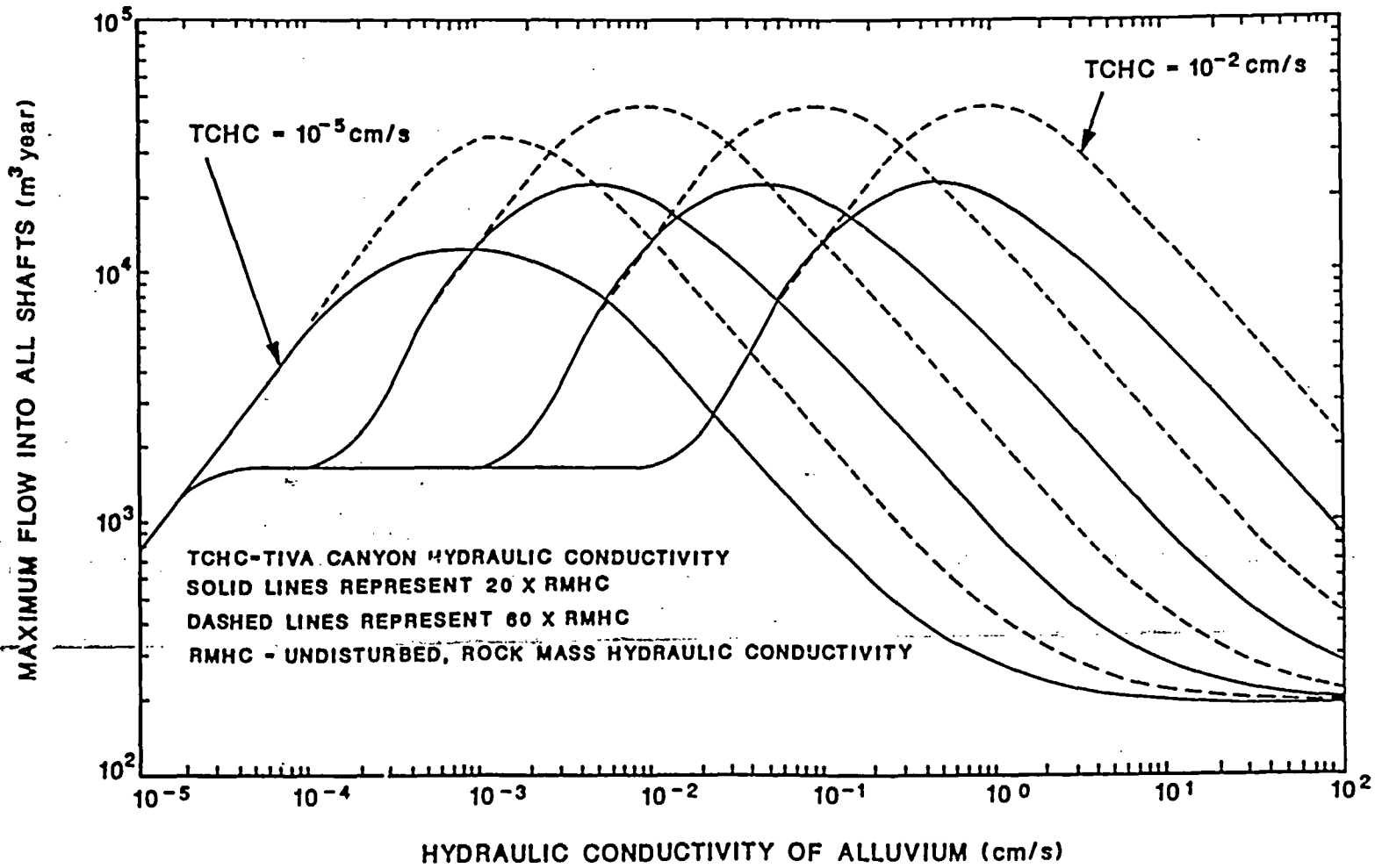

Figure 4-5. Estimated Volumes of Surface Water Entering a Shaft (PMF; Shaft Fill, Hydraulic Conductivity = 10^{-6} cm/s)

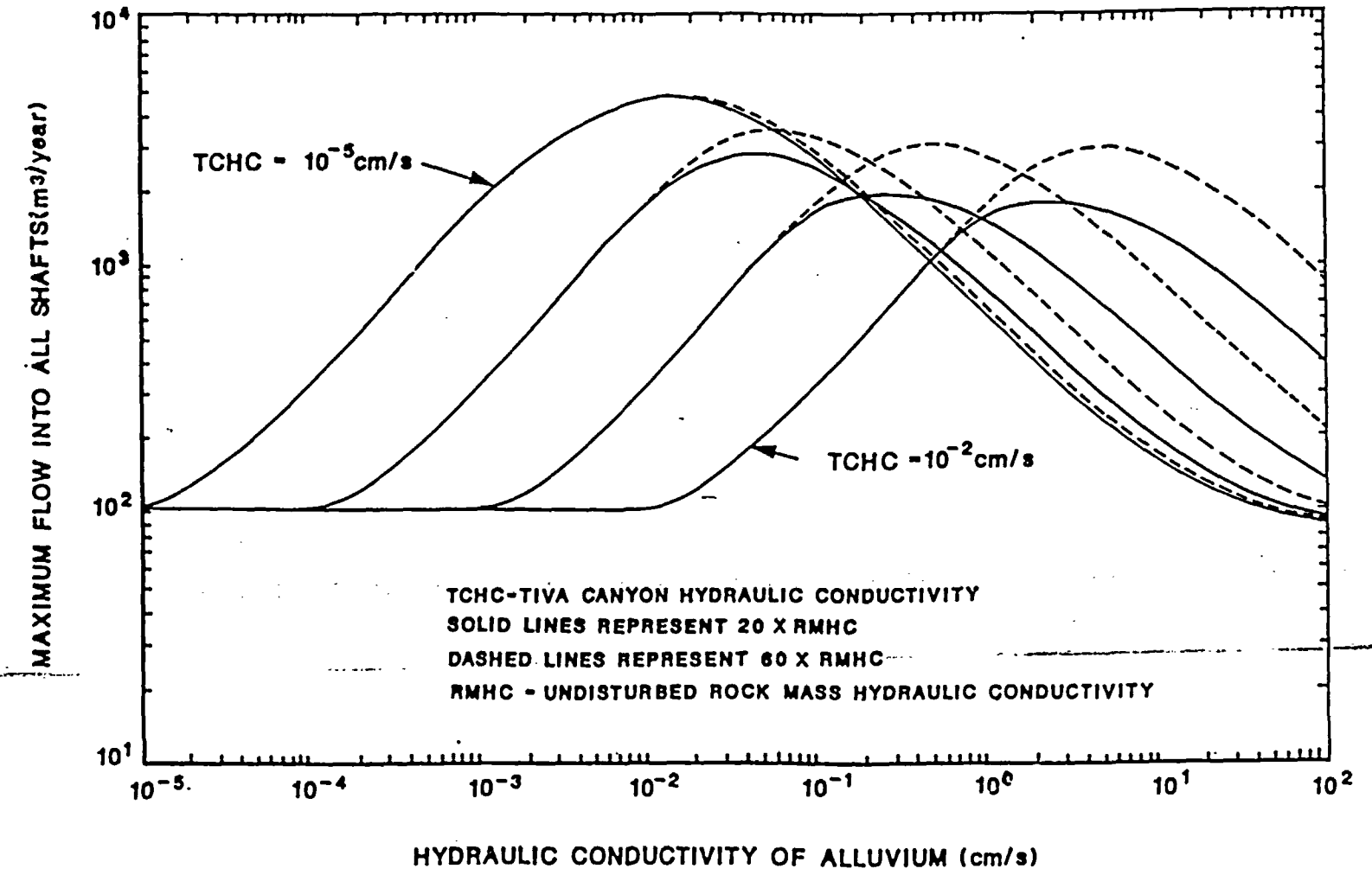

Figure 4-6. Estimated Volumes of Surface Water Entering a Shaft (500-Year Flood; Shaft Fill, Hydraulic Conductivity = 10^{-2} cm/s)

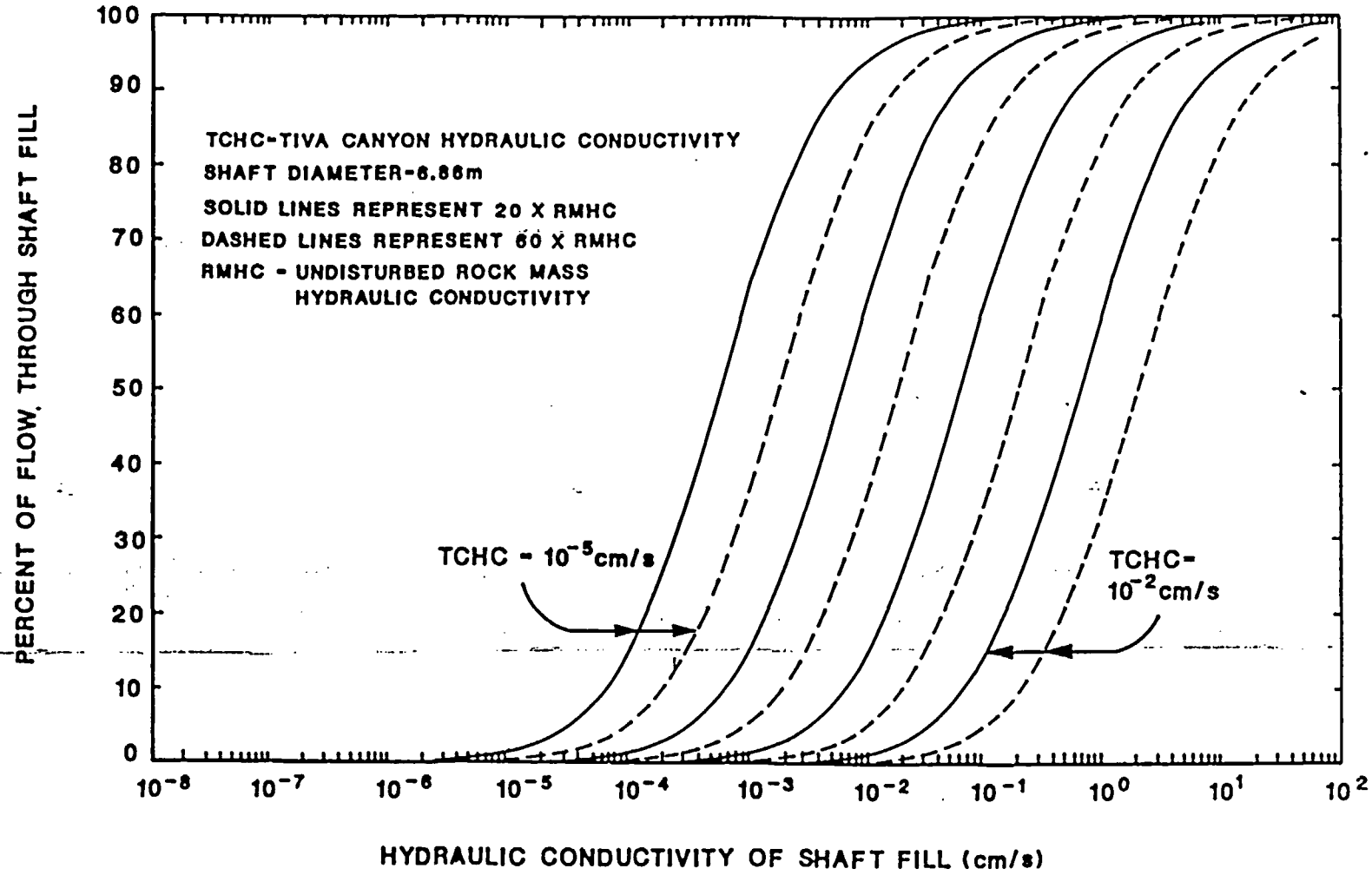

Figure 4-7. Percent of Flow Through Shaft Fill as a Function of Fill and Tiva Canyon Hydraulic Conductivity

Similar results have also been computed for a 500-year flood (Condition 3). The results are displayed on Figure 4-6. The maximum amount of water entering the shaft computed for this condition is 4,950 m^3. The maximum percentage of total flow into the shafts as compared to the available flood water supply is 8% for the 500-year flood and 19% for the PMF-flood.

In order to verify the numerical results obtained, an alternate calculation was performed to check major assumptions, analyses methods, and input (materials properties and geometry). This alternate calculation (see Appendix A-4) estimated the inflow into the shaft and MPZ of one shaft, i.e., ES-1. The volume of water used in the calculation is the 500-year flood volume at the ES-1 location (see Table 4-1). This volume is 22,000 m^3. Considering the general nature of the alternate calculation, the results from both calculations are in very good agreement. Appendix A-4 presents a comparison of the results as well as the details of the alternate calculation.

4.2 WATER FLOW INTO THE UNDERGROUND FACILITY

From consideration of infiltration rates and saturation levels of the stratigraphic units in the unsaturated zone at Yucca Mountain, it is believed that the moisture flux beneath the repository horizon is less than 0.5 mm/yr (DOE, 1986, p. 6-129). Because the flux is low, the ground-water flow is expected to be dominantly through the matrix, and not through fractures. Nevertheless, for purposes of conservatism, we assume that inflows could occur from the rock matrix and from discrete faults following heavy rainfall.

4.2.1 Ground-Water Inflow From the Rock Matrix

Fernandez and Freshley (1984, p. 32-34) report analyses performed with the computer code TRUST for ground-water inflow to a drift under pre-waste-emplacement conditions. Excepting discrete faults, Fernandez and Freshley predicted that the inflow into a drift should be small, on the order of 10^{-11} m^3/s for a unit length of drift backfilled with clay, to essentially zero for a drift backfilled with sand. If the entire repository is backfilled with a clay-like material and the drift inflow rate is assumed to be 10^{-11} m^3/s for a unit length of drift, the total amount of water entering the emplacement drifts for vertical emplacement would be 82 m^3/yr. To estimate more accurately flow into the emplacement drifts, the input parameters were changed to more closely represent the anticipated conditions within the Topopah Spring Member (Freshley et al., 1985). The hydrologic properties of the host rock were changed as well as the flux through the modeled area. The saturated permeability of the host rock matrix was modeled as 1.9×10^{-18} m^2 (approximately 1.9×10^{-9} cm/s) and the flux was 0.1 mm/yr. Inflows into drifts backfilled with clay and sand were computed as 1.3×10^{-12} m^3/s and 9.7×10^{-15} m^3/s, respectively. Using these inflow rates and assuming the emplacement drifts are backfilled, the total inflow into all the emplacement drifts would be about 5 m^3/yr and less than 0.1 m^3/yr for the clay and sand backfilled cases,

respectively. These additional analyses predict low matrix flow into backfilled emplacement drifts within the repository boundary.

To confirm the results and conclusions of the analyses using TRUST, similar analyses were performed by Mondy et al. (1985). A two-dimensional finite-element code, SAGUARO, was used to model flow through a backfilled drift and adjacent to a vertically emplaced waste package. The drift was backfilled with sand or clay. The model considered only matrix flow with no flow through fractures. The results confirm those obtained by Fernandez and Freshley in that sand backfill acts as a capillary barrier that inhibits flow into the drift. The results predict that downward ground-water flux will be diverted around the sides of the emplacement drift and will flow back toward the waste package below the drift. Flow past the waste package was 91% of the influx rate for the case of clay backfill and 81% of the influx rate for sand backfill.

4.2.2 Ground-Water Inflow From Discrete Faults

4.2.2.1 Occurrence of Faults

Figure 2-2 shows faults mapped at the surface in the vicinity of the proposed repository location. While a relatively large number of faults are shown, it is possible that many are minor features that may not penetrate to the repository horizon. It also is possible to suppose that fracture zones will occur at depths that have not been recognized at the surface. For purposes of evaluating potential inflow, two faults are evaluated: the unnamed fault or series of faults in Drill Hole Wash, and the Ghost Dance Fault which runs north-south across the repository to the west of the ES. In the absence of more definitive site information, the evaluation of these two faults is, at present, a reasonable estimate of fault occurrences at depth.

The faults in Drill Hole Wash have a major topographic expression in the form of the wash. This presumably reflects a relatively erodible fracture zone that may also possess high vertical permeability. Scott et al. (1984) discussed the nature of the faults in Drill Hole Wash and other northwest-southeast trending washes to the north of the repository and evaluated the possibility that the faults may act as conduits for ground-water flow. A preliminary conclusion was that the ground-water flow characteristics were not significantly different beneath the washes than in the adjacent unfaulted blocks. This preliminary conclusion does not disprove the possibility of recharge below the major washes. The faults in Drill Hole Wash may be significant with regard to sealing because they intersect the two ramps and because they underlie an area inundated periodically by flooding (Figure 2-3).

The Ghost Dance Fault intersects the repository upgradient from the ES and is a potential source for inflow that could drain through the drifts to the north and northeast. The fault dips 84° to 89° to the west and is identified at the surface by small stratigraphic separations and by breccia zones (Scott and

Bonk, 1984, Sheet 1). The fault cuts across the east-west-trending washes which drain off Yucca Crest (Figure 2-2). There is no information available regarding the subsurface character of the fault or its hydrologic properties. For purposes of the present discussion, the significance of the Ghost Dance Fault derives from its potential continuity and its location. For the vertical emplacement configuration, Figure E-1 (Appendix E) shows the fault intersecting seven panels in the conceptual repository layout, including 24 emplacement rooms. Nine small washes (Figure E-3) that cross the fault on the surface (within the proposed repository block) may contain flowing surface water during storm events. If recharge* occurs only directly below the intersection of the fault and a wash, possibly only nine rooms would experience inflow after rainfall. In the horizontal emplacement configuration, the Ghost Dance Fault would intercept approximately 100 emplacement holes (Figure E-2). Depending on the observations of water flows into the underground facility, repository layout modification in the vicinity of the Ghost Dance Fault may be desirable. Because the fault crosses the washes almost perpendicularly, the amount of recharge directly from the surface would probably be less than for a fault that underlies a wash along its length.

4.2.2.2 Inflow to Drifts and Ramps

The scenario postulated for fault inflow presupposes there will be rainfall of sufficient intensity and duration to saturate the alluvium and to provide water to a fault/fracture system intersecting the wash. Two cases are considered, one in which the fault zone becomes saturated down to the water table producing fully convergent flow to the drifts intersected by the fault and one in which full saturation does not develop, leading to partially convergent flow (Figure 4-8).

For the first case, with the fracture zone completely saturated from the surface to the water table, the discharge from the fracture zone can be estimated by a method in Freeze and Cherry (1979, p. 490) for inflow to a tunnel located in the saturated zone:

$$Q = \frac{2\pi KHW}{2.3 \log(2H/r)} \qquad (4\text{-}3)$$

where

- Q = ground-water inflow from fracture zone (m^3/s),
- K = effective hydraulic conductivity of the fracture zone (m/s),
- H = hydraulic head (m),
- r = effective radius of the drift (m), and
- W = width of fault zone (m).

*It is assumed that there exists a permeability contrast between the fault zone and the surrounding media, and that flow is confined within this zone.

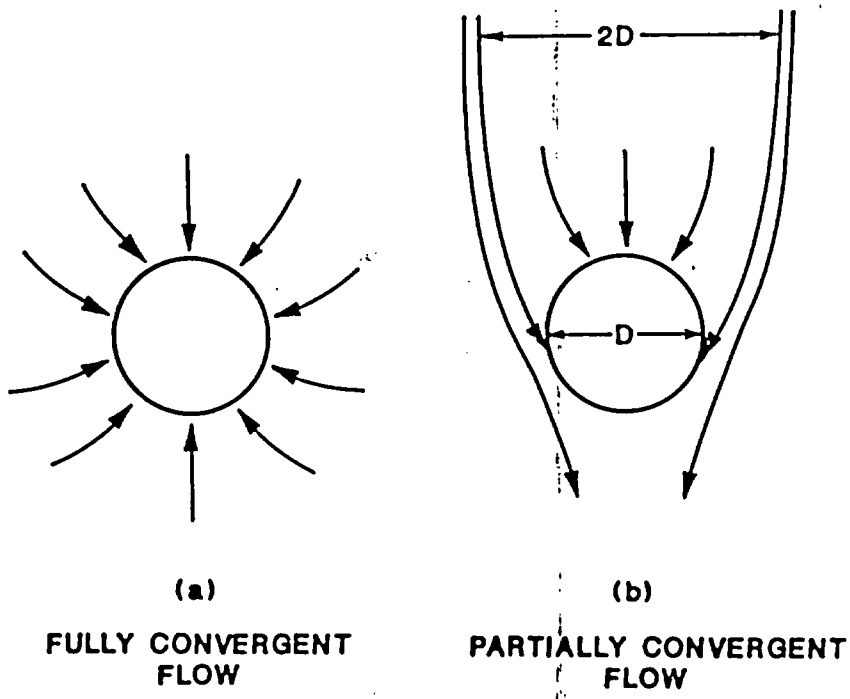

Figure 4-8. Flow Toward a Tunnel in Fully Convergent or Partially Convergent Conditions

This solution also assumes that the drift is infinitely long, that the equivalent porous medium within the fault zone is homogeneous and isotropic, that the pressure heads on the drift wall are atmospheric, and that the water table is maintained at a constant elevation. In reality, it is not likely that the head in the fracture zone will be maintained over significant periods of time, as is implied by a constant water table elevation. It is expected that drift inflows will be transient in nature, not steady, so that the solution may provide the peak rather than the steady-state flow. Another simplification is that the solution implies that flow converges to the tunnel from all directions (Figure 4-8), which is unlikely for a tunnel above the water table. Considering all of the assumptions involved, Equation (4-3) is believed to provide a conservative upper limit for inflow.

There are no reported values for the hydraulic conductivities of fault zones in welded tuff. Intuitively, the conductivity might be expected to be relatively low perpendicular to the fault (because of the presence of clay gouge), but higher within the plane of the fault. The present analyses use a range of 10^{-5} to 10^{-2} cm/s, as given by Scott et al. (1983, p. 299), for the effective, rock mass, hydraulic conductivity for welded tuff. For this range of hydraulic conductivity, the inflow to a 3-m-radius tunnel for a 0.3-m-wide, fracture zone under a head of 366 m ranges from 10^{-5} to 10^{-2} m^3/s. Values for wider fracture zones or other conductivities can be scaled accordingly.

As noted above, it is questionable that flow to a tunnel above the water table would be fully convergent, as shown by Figure 4-8. An alternative solution may be derived for partially convergent flow. In this case, the flow is essentially vertically downward but allowance is made for some convergence, perhaps along fractures inclined towards the tunnel. In the present analysis the area of influence of the tunnel is assumed to be twice the tunnel diameter. Inflow is estimated by

$$Q = KA \tag{4-4}$$

where

Q = flow rate through the fault (m^3/s),
K = equivalent saturated hydraulic conductivity of the fault (m/s),
A = (2D • W) area of influence (m^2),
D = diameter (m), and
W = tunnel width of fault zone (m).

Note that the effective width of flow is assumed to be twice the diameter of the excavation, and accounts for partial convergence of flow through fractures. For the same conditions analyzed above, the calculated range of inflow is 4×10^{-7} to 4×10^{-4} m^3/s. These values are about four percent of the values calculated using Equation 4-3.

For convergent flow to occur from a fault above the water table, the fault must become saturated from a point above the drift down to the water table. Some indication of whether convergent flow might occur is obtained by considering the time required for surface water to penetrate from the ground surface to the water table.

The Green and Ampt transient, one-dimensional, analytical solution (Appendix A-3) is used to calculate the infiltration rate and cumulative infiltration into a fault zone. It is assumed that the fault plane surfaces are impermeable, and that water flowing down the fault system is not absorbed into the matrix. In these calculations, the hydraulic conductivity of the fault zone is varied from 10^{-5} to 10^{-2} cm/s (as used above) and the porosity from 0.004 to 0.4. The lower porosity value is intended to correspond to the effective fracture porosity of a densely fractured zone, whereas the higher value corresponds to a vuggy breccia zone. Initial saturation is varied from 0 to 85%. The times required to achieve saturation to the water table (Figure 4-9) may be evaluated relative to the length of time over which the fault could be recharged at the surface.

To illustrate the variability in the time to saturation to the water table, several scenarios that conceivably bound the potential hydraulic properties of a fault system are postulated. Table 4-4 gives the input parameters for each scenario together with the saturation time and cumulative infiltration for both the repository horizon and the water table. Figure 4-9 shows the time to

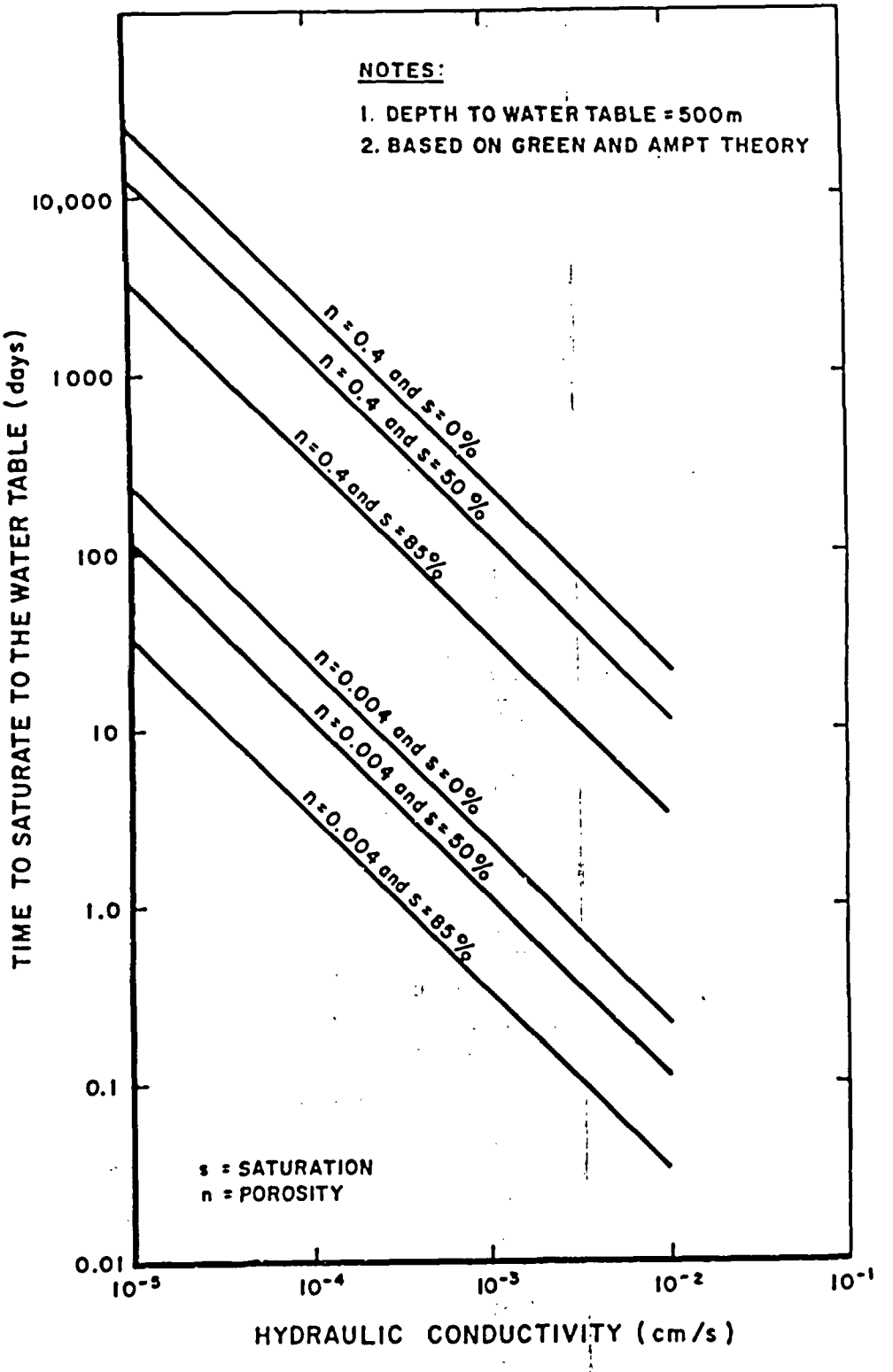

Figure 4-9. Time to Saturate a Fault From the Ground Surface to the Water Table

Table 4-4. Selected Scenarios for Saturating a Fault Zone

Scenario	Initial Saturation, %	Porosity	Saturated Hydraulic Conductivity, cm/s	Saturation Time		Cumulative Infiltration, m	
				Repository Horizon[a]	Water Table[b]	Repository Horizon	Water Table
1	0	0.004	10^{-2}	0.14 day	0.23 day	1.2	2.0
2	0	0.4	10^{-2}	14 days	23 days	120	200
3	50	0.004	10^{-2}	0.07 day	0.12 day	0.60	1.0
4	50	0.4	10^{-2}	7.0 days	11.5 days	60	100
5	85	0.004	10^{-2}	0.02 days	0.03 days	0.18	0.30
6	85	0.4	10^{-2}	2.0 days	3.4 days	18	30
7	0	0.004	10^{-5}	140 days	230 days	1.2	2.0
8	0	0.4	10^{-5}	40 yrs	60 yrs	120	200
9	50	0.004	10^{-5}	70 days	115 days	0.60	1.0
10	50	0.4	10^{-5}	20 yrs	30 yrs	60	100
11	85	0.004	10^{-5}	20 days	34 days	0.18	0.30
12	85	0.4	10^{-5}	5.6 yrs	9.4 yrs	18	30

(a) 300-m depth
(b) 500-m depth

saturate to the water table as a function of hydraulic conductivity. For a relatively conductive fracture system (10^{-2} cm/s) with a low porosity (0.004), the saturation times to the repository horizon and the water table are a matter of hours. When the conductivity is the same but the porosity is high (0.4), saturation time extends to almost a month. When the hydraulic conductivity is decreased to 10^{-5} cm/s, the saturation times are long (≥100 days) regardless of the porosity. As would be expected, when the initial saturation of the fault zone increases, the time required to saturate decreases. Using the Green and Ampt approach, the time to saturate an initially dry system is about twice as long as one starting at a 50% saturation level. The porosity of the fault zone also influences the volume of water required to saturate to the repository level or to the water table. For a high-porosity (0.4) system, the volume required to saturate to the water table is 200 m^3 per unit area (1 m^2) of the fault exposed at the surface. For a low-porosity (0.004) system, the equivalent volume is only 2.0 m^3. These calculations confirm that saturation of a fault and consequent development of convergent flow are very sensitive to two parameters, conductivity and porosity, neither of which is well known at present.

4.2.2.3 Inflow to Horizontal Emplacement Holes

In the horizontal waste emplacement configuration, the emplacement boreholes may intersect water-producing faults such as the Ghost Dance Fault. Potential inflows can be calculated using the same methods described in the previous section, adjusting for the smaller diameter of the opening. For a 1-m-diameter hole, the estimated ranges of inflows corresponding to a range of hydraulic conductivity of 10^{-5} to 10^{-2} cm/s are 1×10^{-5} to 1×10^{-2} m^3/s for convergent flow and 6×10^{-8} to 6×10^{-5} m^3/s for nonconvergent flow.

4.2.2.4 Comparison With Previous Observations

Ground-water inflow from faults in nonwelded tuff above the water table has been observed in tunnels in Rainier Mesa at the Nevada Test Site (Thordarson, 1965, p. 35-46). The intent in presenting this information is to illustrate the magnitude of water inflows into tunnels under perched water conditions.

Typically, the maximum inflow occurred immediately after the water-bearing fracture was penetrated by the tunnel. The inflow then declined rapidly to become a small seep or drip after a few days. Most of the fractures drained completely within a few weeks or months, but water dripped from some fractures for greater than 2 years. In one tunnel system (U12e), the estimated total discharge over a 5-year period was 30 to 50 million gallons (Thordarson, 1965, p. 46). The total length of drifts in the system was about 5.8 km (19,000 ft) and about 110 faults were mapped. Most of the fracture water was produced by 50% to 60% of the faults.

Most of the faults described by Thordarson from the U12e tunnel system were minor anticipated faults with stratigraphic displacements measured in inches.

Few faults were traced between parallel drifts, indicating lateral extents of less than 30 m to 90 m (100 ft to 300 ft). Water occurred irregularly and some highly fractured zones were dry even though open fractures were noted. Thordarson noted that water was perched, probably as a result of the faults pinching out at depths or along a strike.

Thordarson reports the occurrence of fracture water into flow categories. These include <1, 1-5, 5-20, and >20 gal/min. The number of occurrences per category were: 117 for <1 gal/min, 41 for 1-5 gal/min, 14 for 5-20 gal/min, and 5 for >20 gal/min, (Thordarson, 1965, Plate 5). For comparison, the flows computed in Section 4.2.2.2 range from 10^{-5} to 10^{-2} m^3/s (0.2 to 200 gal/min) for convergent flow to $4x10^{-7}$ to $4x10^{-4}$ m^3/s (0.006 to 6 gal/min) for partially convergent flow. At one location in G-tunnel at the Nevada Test Site, fracture water drains at a rate of about $6x10^{-7}$ m^3/s (0.01 gal/min) (Fernandez and Freshley, 1984, p. 24).

In making the above comparisons, it is important to note differences in conditions at Rainier Mesa and Yucca Mountain. The tunnels at Rainier Mesa are in an area of greater surface recharge and greater moisture flux in the unsaturated zone than in the proposed repository location at Yucca Mountain (DOE, 1986, pp. 6-333). Tunnels that yield the most perched water were driven into zeolitic-bedded tuff. The fractures that contained water were generally drained within days and were poorly interconnected (Thordarson, 1965, pp. 35-42). By contrast, the repository at Yucca Mountain is proposed to be located in densely welded, highly fractured, Topopah Spring Member tuff. Because of the pervasive and abundant fractures within the Topopah Spring welded unit, it can be inferred that the fractures are interconnected; thus, this unit is free draining (DOE, 1986, pp. 6-138), and buildup of water in fractures, similar to that encountered at Rainier Mesa, is not likely.

4.2.3 Assumptions Selected for Water Flow Into the Underground Facility

Based on the discussions in Sections 4.1.1 and 4.1.2, additional assumptions for anticipated and unanticipated water flow conditions are given in Table 4-5. Water can potentially enter the drifts, ramps, and horizontal emplacement holes from water-producing faults or the rock matrix.

For anticipated conditions, the water-producing zones include the Ghost Dance Fault and the fault underlying Drill Hole Wash. It is assumed that inflow into the zones identified above is controlled by the duration and frequency of precipitation events greater than 1.3 cm/yr (0.5 in./yr) (identical conditions defined in Section 4.1.4). It is further assumed that only 50% of all drifts intercepted by the Ghost Dance Fault are interconnected to surface water supplies. For vertical emplacement, only 9 of the 24 drifts are below surface water supplies. For horizontal emplacement, only 25 of the 97 horizontal emplacement holes are below surface water supplies (see Appendix E). Therefore, 50% is considered reasonable and conservative. For the ramps, it is assumed that two water-producing zones are encountered for each ramp.

Table 4-5. Assumptions Made for Anticipated and Unanticipated Water Flow Into the Underground Facility

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Anticipated Water Flow

- Water flow in water-producing zones (i.e., Ghost Dance Fault and Drill Hole Wash) is periodic and controlled by duration of precipitaton event.
- Water supply is defined by four thunderstorms >1.3 cm (>0.5 in.) and runoff lasts 1 hour per storm.
- Duration for each thunderstorm is 1 hour.
- 50% of Ghost Dance Fault and repository drift intersections produce periodic inflow of water.
- No matrix flow contacts waste.

Unanticipated Water Flow

- Matrix and fracture flow contacts waste
  - Maximum infiltration rate of 1.0 mm/year
  - For vertical and horizontal emplacement, the amount of water is that which passes the total projected floor area of the repository (including ramps)

Duration and frequency of recharge are the same as those proposed for the repository drifts.

An additional source of water that could enter waste emplacement areas is matrix water. Under ambient conditions, i.e., pre-waste emplacement, the water flow into waste disposal area as determined by Fernandez and Freshley (1984) for a broad range of conditions varied from about 100 to less than 0.1 m^3/yr (Section 4.2.1). But, using conditions that more closely represent the hydrologic conditions at Yucca Mountain, the flow into drifts is extremely low, i.e., 5 m^3/year to 0.1 m^3/year.

No analyses have been performed to determine water flow into the waste disposal areas under elevated temperature conditions. Various analyses have been performed to investigate very-near-field phenomena of vapor and water transport near waste containers (Evans and Huang, 1983; and Pruess et al., 1986). While the analyses are definitive in indicating near-field, thermal phenomena that could potentially occur, they are not definitive in indicating how the near-field, temperature environment will influence the net flux rate over the life of the repository. It is anticipated that as the temperature in the rock drops below 100°C, resaturation of the rock will occur. These phenomena may vary temporally as temperature changes in the repository. For purposes of the present analyses, it is assumed that such phenomena would balance out over the life of the repository.

For unanticipated conditions, it is assumed that the infiltration rate is 1.0 mm/yr for faults and the rock matrix. This value is twice the upper bound of recharge to the ground-water table and potentially sufficient to initiate fracture flow. For vertical and horizontal emplacement, all of the volume of water that passes the plan area of drifts is conservatively assumed to contact the waste packages. The assumptions discussed above are summarized in Table 4-5.

The following sections apply these assumptions and derive estimates of water volumes that can enter waste disposal areas.

4.2.4 Estimates of Water Flow Into the Underground Facility Under Anticipated Conditions

As assumed in the previous section, periodic water flow can enter the repository drifts and the ramps through structural features such as the Ghost Dance Fault and faults underlying Drill Hole Wash. Further, it is assumed that this flow is controlled by the duration and frequency of precipitation events greater than 1.3 cm/yr. Using these assumptions and the Green and Ampt solution, water flow through faults is computed below.

Because the hydrologic nature of faults is generally unknown, variation in several parameters that could affect flow through faults is considered, as discussed in Section 4.2.2.2. The saturated hydraulic conductivity of the

fault zone is assumed to vary from 10^{-2} to 10^{-5} cm/s, similar to the bulk rock conductivity of the Topopah Spring Member. The fault zone is assumed to 0.3-m wide. Porosity of the fault zone ranges from 0.004 to 0.4, while the initial saturation ranged from 0% to 85%. Additionally, either partially convergent or fully convergent flow is considered possible.

The type of flow (partially or fully convergent) is dependent on the time to saturate the fault zone from the surface to the water table (Section 4.2.2.2). If the time to saturate the faults zone is greater than the 1-hour, water flow at the surface, then partially convergent flow is assumed. If the time to saturate the fault zone to the water table is less than 1 hour, then partially convergent flow occurs for the remainder of time up to 1 hour. A determination as to whether partially or fully convergent flow occurs is made from the results presented in Figure 4-9 obtained using the Green and Ampt solution. If partially convergent flow occurs, the extent of saturation into the fault zone can be approximated.*

The results of the calculations for vertical and horizontal emplacement are presented in Table 4-6. As indicated in this table, fully convergent flow does not occur within the 1-hour storm. The upper values postulated to enter waste disposal areas are 62 m^3/yr for vertical emplacement and about 44 m^3/yr for horizontal emplacement. Water flows into ramps can be computed using this same approach. The most likely area that inflow will occur appears to be where the ramps penetrate through Drill Hole Wash. If it is assumed that each ramp penetrates two, water-producing, fault zones, then the total inflow from these sources is about one-third of the drift inflow computed in Table 4-6 or about 21 m^3/yr entering the ramps with the current repository design. This volume would enter the perimeter drift and migrate to the lowest point of the repository near the EES.

4.2.5 Estimates of Water Flow Into the Underground Facility Under Unanticipated Conditions

Because of the broad range of conditions that can be assumed for flow through faults and matrix, the two types of flows were combined. For vertical and horizontal emplacement it is assumed that all waters passing the cross-sectional area of the repository could potentially contact the waste

*The Green and Ampt solution assumes saturation of a material having uniform properties. In applying this approach, it is important to understand that fractures and presumably fault zones comprised of fractures would desaturate more easily than pores in the rock matrix in the unsaturated zone. Further, a relatively continuous air phase within a fracture can provide a practically infinite resistance to liquid flow within the fracture (Wang and Narasimhan, 1985, p. ii). Therefore, the application of the Green and Ampt approach may overestimate inflow through partially saturated, fault zones comprised of fractures.

Table 4-6. Yearly Inflow Through Ghost Dance Fault Under Anticipated Conditions

Saturated Hydraulic Conductivity of Fault Zone, cm/s	Porosity of Fault Zone	Saturation of Fault Zone, %	Time to Saturate Fault Zone to Ground-Water Table[a], hr	Total Flow Into Drifts or Horizontal Emplacement Holes[b], m^3
			Vertical Emplacement	
10^{-2}	0.004	0	5.5	62
	0.004	50	2.8	62
	0.004	85	0.8	62
	0.4	0	550	62
	0.4	50	280	62
	0.4	85	82	62
10^{-5}	0.004	0	5,500	6.2×10^{-2}
	0.004	50	2,800	6.2×10^{-2}
	0.004	85	820	6.2×10^{-2}
	0.4	0	550,000	6.2×10^{-2}
	0.4	50	280,000	6.2×10^{-2}
	0.4	85	82,000	6.2×10^{-2}
			Horizontal Emplacement	
10^{-2}	0.004	0	5.5	44
	0.004	50	2.8	44
	0.004	85	0.8	44
	0.4	0	550	44
	0.4	50	280	44
	0.4	85	82	44
10^{-5}	0.004	0	5,500	4.4×10^{-2}
	0.004	50	2,800	4.4×10^{-2}
	0.004	85	820	4.4×10^{-2}
	0.4	0	550,000	4.4×10^{-2}
	0.4	50	280,000	4.4×10^{-2}
	0.4	85	82,000	4.4×10^{-2}

[a] From Figure 4-9.

[b] For vertical emplacement, partially convergent flow is 1.3 m^3/hr per drift (for 10^{-2} cm/s) and 1.3×10^{-3} m^3/hr per drift (for 10^{-5} cm/s); water enters 24 emplacement drifts; four events having 1-hour duration each; and diameter of drift = 6m. For horizontal emplacement, partially convergent flow is 2.2×10^{-1} m^3/hr (for 10^{-2} cm/s) and 2.2×10^{-4} m^3/hr (for 10^{-5} cm/s); water enters 100 emplacement holes; four events having 1-hour duration each; and diameter of emplacement hole = 1 m.

packages. If the total floor area of the repository, including the projected floor area of the ramps, is 5,588,850 m^2 (Table 2-6), and the upper-bound infiltration for unanticipated conditions is assumed to be 1.0 mm/yr, the amount of water could be 5,600 m^3/yr. We recognize that, for unanticipated conditions, infiltration in selected areas could be higher than 1.0 mm/yr. The value of 1.0 mm/yr, therefore, represents an average, upper-bound infiltration value. As results from hydrologic testing from the Exploratory Shaft Facility are obtained, estimates of water flow under unanticipated and anticipated conditions will be modified as appropriate.

5.0 ASSESSMENT OF THE NEED FOR SEALING

The determination of the need for sealing is based on a comparison of estimated, allowable amounts of water that can enter the waste disposal area and the actual amount of water that enters the waste disposal area. This assessment (Section 5.1) addresses the quantitative necessity to reduce water or air flow or to divert water flow so that radionuclide releases from the waste will not exceed the NRC limits. In the overall determination of the need for seals, additional consideration is given to other nonquantitative considerations, including the requirements from 10 CFR 60.134 that shafts and boreholes should not act as preferential pathways for ground water (Section 5.2).

5.1 ASSESSMENT OF THE NEED FOR SEALING TO MEET THE NRC RELEASE CRITERIA

Calculations presented in Chapter 3 were used to develop the maximum-allowable-hydrologic-performance goals for the sealing system. These goals can be compared with estimates developed in Chapter 4 and summarized in the following table of the amounts of water that can enter various parts of the sealing subsystem for anticipated and unanticipated conditions.

Table 5-1. Summary of Estimated Inflows Into Shafts, Ramps and the Underground Facility

	Anticipated Conditions, m^3/yr	Unanticipated Scenarios or Events, m^3/yr
Shafts[a]	270	4,950[b] - 83,700[c, d]
Ramps	21	(e)
Underground Facility		
Vertical Emplacement Drifts	62	5,600
Horizontal Emplacement Boreholes	44	5,600

(a) Assumes shaft fill, hydraulic conductivity = 10^{-2} cm/s. These values represent the amount of water entering the upper portion of the shaft.
(b) 500-year flood scenario
(c) Probable maximum flood scenario (PMF)
(d) In the above estimate, boreholes are not included. The inflows through boreholes are expected to be small because the boreholes are few in number and cross-sectional areas are small (Section 6.7). Sealing of the borehole, as required by state laws, will further reduce the potential for water flow into the underground facility.
(e) Estimated ramp inflow included in underground facility estimates.

These values (Table 5-1) refer to the amounts of water that can enter the repository and not necessarily the water that can potentially enter the waste emplacement areas and contact the waste. The amount of water that can enter the waste emplacement areas will be less than the amounts (as indicated in Table 5-1) entering the overall repository for the following reasons:

1. The Topopah Spring Member is highly fractured and some drainage will occur through the lower portion of each of the shafts. Table 5-2 shows a comparison between the volumes of water estimated to enter each shaft and the estimated sump storage capacities and drainage capacities (see Appendix A-5 for further details). Drainage rates are calculated assuming that the bulk, saturated hydraulic conductivity of the rock at the lower portion of each shaft is 5×10^{-6} cm/s, a reasonable lower bound for either the Topopah Spring Member or the vitric Calico Hills Member. With the exception of the Emplacement Exhaust Shaft (EES), the drainage through the sumps can easily accommodate the infiltration into each shaft under assumed anticipated conditions. The EES connects to drifts in the lowest part of the repository that can accommodate a much larger volume (see Section 6.2.1) than the infiltration shown in Table 5-2.

2. All of the shafts are located near the lowest part of the repository. With the current repository layout for vertical emplacement, about 6,800 m^3 of water can accumulate in the lowest part of the repository before the water level would rise sufficiently to enter the waste emplacement area (see Section 6.2.1). Of the additional amount above 6,800 m^3 that could enter the waste disposal area, only a fraction would potentially contact the waste.

3. Within the underground facility, some drainage will occur through drift floors. If the lower value of bulk, saturated hydraulic conductivity of the Topopah Spring Member (10^{-5} cm/s from Table 2-2) is used to compute drainage through the drift floor, each 32 m^2 of drift floor will drain about 100 m^3 in 1 year (Figure A-5.3). The drainage capacity of the drifts in the lowest part of the repository (corresponding to the storage volumes discussed above) is about 32,000 m^3/year for the vertical emplacement configuration and 20,000 m^3/year for the horizontal emplacement configuration. For the highest value (10^{-2} cm/sec), the drainage capacity could be three orders of magnitude higher or

Table 5-2. Comparison of Inflow to Shafts With Storage and Drainage Capacities of Sumps

Shaft	Sump Depth,[a] m (ft)	Yearly Water Entry Into Shaft,[b] m^3 (ft^3)	Storage (Sump) Volume Below Repository Station,[c] m^3 (ft^3)	Yearly Drainage Through Unlined Shafts,[c] m^3 (ft^3)	
				Nasberg-Terletskata[d]	Glover[d]
Emplacement Exhaust Shaft (EES)	3.0 (10)	106.0 (3,750)	34 (1,190)	63 (2,230)	315 (11,100)
Men and Materials Shaft (MMS)	24 (80)	106.0 (3,750)	270 (9,540)	1,340 (47,300)	2,680 (9.47×10^4)
Exploratory Shaft-1 (ES-1)	140 (460)	44.2 (1,560)	659 (23,300)	31,100 (1.13×10^6)	50,000 (1.77×10^6)
Exploratory Shaft-2 (ES-2)	(e)	13.4 (473)	(e)	NA	NA

(a) Height of water in shaft is equal to sump depth (see Figure 2-10).
(b) From Table 4-3 for normal conditions.
(c) All storage and drainage capacities based on a shaft diameter of 3.7 m. The larger the shaft diameter, the larger will be the drainage capacity.
(d) See Appendix A-5 for formulas, assumes liner removed.
(e) The current design of ES-2 has no sump, but does have a connecting drift between ES-1 and ES-2 as indicated in Figure 2-10.

32,000,000 m^3/year for vertical emplacement, and 20,000,000 m^3/year for horizontal emplacement.

4. With the horizontal emplacement configuration each emplacement borehole is elevated about 1.4 m above the floor of the emplacement drift. This means that any water entering the drifts (either directly or from the shafts or ramps) cannot enter the emplacement boreholes until the storage volume in the lowest part of the repository is filled. As noted above, the volume is 3,300 m^3 for the horizontal configuration with a backfill porosity of 0.3. Moreover, since the annual drainage capacity of the lowest area exceeds the storage volume, the volume entering the drifts would have to exceed 3,300 m^3 each year before water would back up sufficiently to enter emplacement boreholes. Higher volumes could be accommodated by backfill with a higher porosity, by not backfilling some drifts, or by modifying the repository design.

From the preceding discussion, it can be concluded that for anticipated conditions water entering the shafts and ramps will not enter the waste emplacement areas. Only the water directly entering the emplacement drifts (for vertical emplacement) or the emplacement boreholes (for horizontal emplacement) could contact waste. This suggests that the amount of water that can potentially contact the waste under anticipated conditions can be no greater than approximately 62 m^3/year for vertical emplacement or 44 m^3/year for horizontal emplacement. These values assume that all water entering the underground facility will contact the waste which is unlikely to occur. These amounts fall well below the maximum-hydrologic-performance goals computed for select radionuclides in Chapter 3 (Table 3-2).

Considering now unanticipated scenarios or events, it is seen that the unanticipated water inflows (Table 5-1) do not exceed the maximum-hydrologic-performance goals computed for selected radionuclides in Figure 3-2. Referring first to inflow from the shafts, the inflow into shafts resulting from a 500-year flood in each of the drainage basins is computed to be 4,950 m^3. This amount could be contained in the lowest part of the repository for vertical emplacement but not for horizontal emplacement.* The potential inflow from the probable maximum flood would exceed the storage capacity for

*The flood volume could be accommodated for the horizontal emplacement configuration if some of the potential water storage area is not backfilled or if the repository design is modified to enlarge the water storage volume. For example, a volume of 10,000 m^3 of water could be accommodated with the present repository design if no wastes are stored below 2,945 feet elevation. This would mean relocating about 15 emplacement boreholes, representing about 0.6% of the total waste inventory.

both vertical and horizontal emplacement. However, such an event would theoretically occur infrequently during the lifetime of the repository. From Table 5-1, the direct inflow to the emplacement drifts (vertical and horizontal emplacement) under unanticipated conditions is 5,600 m^3/year.

In reaching a determination on the need for sealing, it is important to understand that many conservative assumptions are incorporated in both the computed goals and computed inflows. At the same time, there are acknowledged uncertainties in the analysis. The conservative assumptions include

- failure of waste packages begins at 300 years following closure,
- all water entering the emplacement drifts or boreholes contacts the waste, and
- the assumed solubility limit is 5 μg/ml (5×10^{-3} kg/m^3). This value coincides with the uranium concentration (unfiltered solution) of J-13 water in which bare fuel was emplaced.

More realistic assumptions could include

- substantially complete containment is assumed for 300 to 1,000 years,
- if uniform corrosion is the only mechanism to breach a waste disposal container, the expected container lifetimes would be on the order of 10,000 years (DOE, 1986, pp. 6-371, 6-372),
- only a portion of the water entering the emplacement drifts contacts the waste, and
- the uranium concentration of J-13 water, in which spent fuel having a split defect was emplaced was 0.10 μg/mℓ (10^{-4} kg/m^3) (Wilson and Oversby, 1985, p. 9).

If failure did occur similar to that described in Section 3.1.1.2; if only a portion of the water contacted the waste; and the uranium solubility was 0.10 μg/ml; the recomputed hydrologic goals could be as much as three orders of magnitude greater than the maximum-allowable-hydrologic-performance goals computed for the radionuclide given in Figure 3-2. At the same time, there are uncertainties in the derivation of the goal (e.g., the radionuclide inventory in the cladding gap and the release rate for these radionuclides) and in the analysis of potential inflows (e.g., the number and properties of faults).

Figure 5-1 compares the results of the maximum-allowable-performance goals for the sealing subsystem (from Table 3-2) with the estimates of the anticipated and unanticipated water flows computed in Chapter 4. The annual, water volumes entering the waste disposal area are shown in Figure 5-1 to be 44 to 62 m^3/year for anticipated hydrologic conditions and about 5,600 m^3/year for unanticipated hydrologic conditions. For unanticipated hydrologic conditions, it is postulated that the estimated annual flow of 5,600 m^3/year can be exceeded by some amount associated with flooding events. As computed in Chapter 4, this amount can be up to 4,950 m^3/year for a 500-year flood or up to 83,700 m^3/year for a PMF flood. The estimates for the maximum-allowable-performance goals are always greater than 136,000 m^3/year. Because

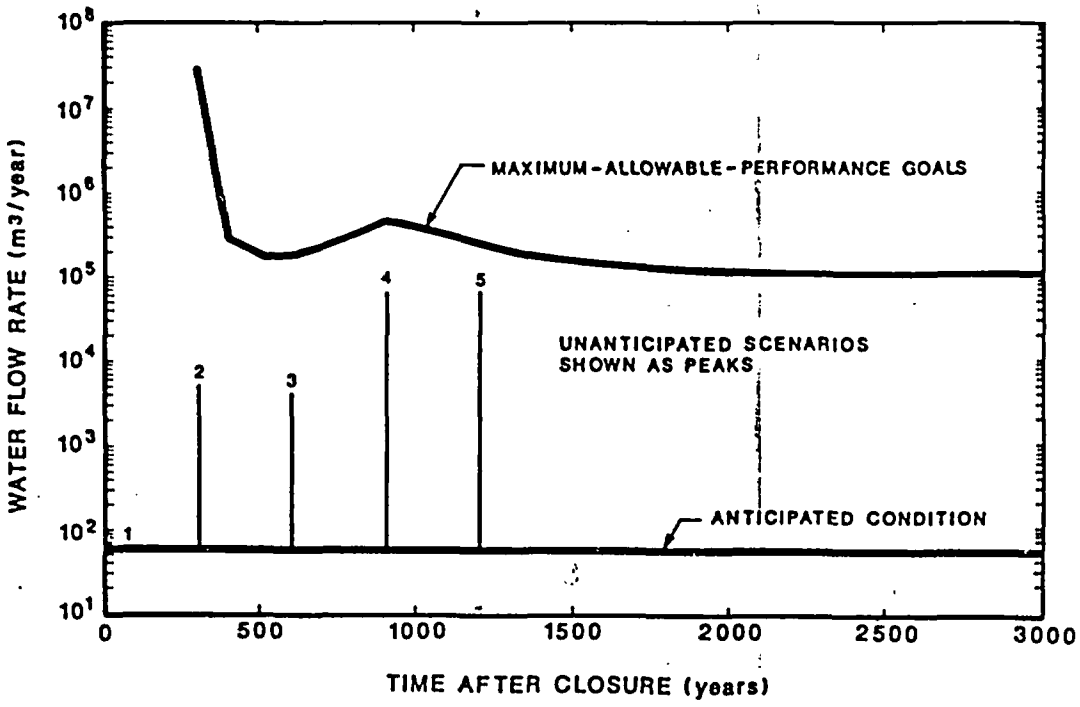
- the estimates for water volumes entering the waste disposal area are less than the maximum-allowable-performance goals,
- the lines representing the anticipated and unanticipated flow volumes have not been reduced to account for drainage through drift floors or the sump areas,
- only a portion of the water entering the repository contacts the waste, and
- the maximum-allowable-performance goals are computed using the conservative assumptions identified above and in Chapter 3 rather than the more realistic assumptions identified above,

we feel, given the current understanding of the site, that sealing is not necessary.

5.2 SHAFTS AND BOREHOLES AS PREFERENTIAL PATHWAYS

In addition to the release limit for radionuclides for the engineered barrier system, 10 CFR 60 provides general guidance for the design of seals and selection of seal materials. Section 60.134.a states that "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." Section 60.134.b states that "materials and placement methods for seals shall be selected to reduce, to the extent practicable: (1) the potential for creating a preferential pathway for ground water; or (2) radioactive waste migration through existing pathways."

In evaluating the potential for shafts and boreholes to act as pathways for radionuclide migration, it is important to note that, at present, the proposed repository at Yucca Mountain lies in an unsaturated zone above the water table, and ground-water movement in the vicinity of the repository would be predominantly, vertically downward if the position of the ground-water table



CONDITION	DESCRIPTION
1	Anticipated Condition - Vertical Emplacement • Episodic infiltration into shafts and Ghost Dance Fault from surface - four flooding events per year - condition of flow is sheet flow (1.25 ft deep) over shaft and fault locatons for one hour per event
2	Unanticipated Scenario • Climatic change inducing fracture flow - infiltration of 1 mm/year - all of infiltration passing through repository plane enters waste disposal area
3	Unanticipated Scenario • 500-year floods occur simultaneously at all shaft locations - all waters retained in alluvium at all shaft locations for potential transport into shaft - no seals in shafts - nominal shaft fill only - no sheet flow out of drainage basin and no evapo-transporation - maximum value given
4	Unanticipated Scenario • PMF floods occur simultaneously at all shaft locations - all water retained in alluvium at all shaft locations for potential transport into shaft - no seals in shafts - nominal shaft fill only - no sheet flow out of drainage basin and no evapo-transporation - maximum value given
5	Unanticipated Scenario • Combine climatic change scenario 2 with PMF scenario 4

Figure 5-1. Comparison of Maximum-Allowable-Performance Goals to Anticipated and Unanticipated Flows

remains below the repository, and is not subject to climatic change. Because the shafts do not penetrate to the water table, and because there is no known mechanism for waste transport upwards in the shafts, it currently appears that it is not credible that the shafts could act as pathways for radionuclide migration. The shafts do clearly provide preferential pathways for water movement into the repository although, as described above, the amount of water will be small except possibly for extreme events.

It is noted that where existing boreholes penetrate through the repository horizon to the ground-water table, such boreholes could represent preferential pathways if they intersected emplacement zones. In these few instances, it would be prudent to insure through accurate survey control that the emplacement rooms were located a minimum distance of 15 m away from such preferential pathways as discussed further in Section 6.7.

5.3 CONCLUSIONS

From the evaluation in Section 5.1, we conclude that sealing is not necessary. Nevertheless, sealing measures are proposed in Chapter 6 to provide additional assurance that repository performance objectives will be met when extreme conditions occur. For these extreme conditions, sealing is proposed for a period of less than 1,000 years following closure.

The time period for sealing is established by considering design-basis-performance goals (Chapter 3) in relation to the unanticipated volumes of water entering waste disposal areas for selected, unanticipated scenarios (Chapter 4). The design-basis-performance goals are reduced from the maximum-allowable-performance goals. In this sense, the use of the design-basis-performance goals provides a degree of conservatism. The assumption of constant, unanticipated flow provides additional conservatism and indicates only an excess for events of low probability. This excess can only occur up to 1,000 years after closure, which establishes the maximum life for sealing components.

The inclusion of some sealing to address extreme conditions will also address the 10 CFR 60 guidelines that some measures should be taken so that the penetrations will not act as preferential pathways for ground-water movement. These measures are required to reduce, but not prevent ground-water flow so that they should not be regarded as seals in the strict sense of the word. Moreover, Chapter 6 presents design options for reducing the potential inflow through shafts, controlling water flow through the repository drifts, and sealing boreholes. Quantitative design requirements associated with these design options are also defined.

6.0 DESIGN REQUIREMENTS FOR SEAL COMPONENTS

Previous chapters of this report have developed hydrologic performance goals and evaluated the need for sealing. While it is concluded in Chapter 5 that sealing is not needed, design options and associated design requirements are proposed to provide a greater degree of assurance that the performance goals can be met. In this chapter, design options are presented and associated design goals and design requirements are identified.

The logic used to arrive at preferred design options (or sealing components), goals, and design requirements is presented in Figure 6-1. First, performance goals* are allocated to the shafts and ramps and the underground facility (Section 6.1). Design options are identified, and design goals are then established. These design goals can be related to the overall performance goal of the sealing system or the general functional requirement of diverting water away from the waste packages. For example, because water might (1) enter the underground repository, (2) flow through seal components, (3) enter emplacement rooms, and (4) potentially affect overall performance, the sealing component design for flow through such components would be subject to performance allocation. Alternatively, in remote areas of the repository or in areas where waste is not emplaced, the inflows may pose little potential to contact waste, provided that the drainage capacity is adequate. In these latter cases, design goals are developed independently of the performance allocation. After the design goals are selected, design requirements are computed. The design options are then rank ordered by considering the ease with which the design requirements can be achieved.

The design requirements are generally based on simple hydrologic models that assume fully saturated conditions and Darcy flow. In this way, it is anticipated that conservative design requirements are developed. Because the performance goals for the sealing system relate mainly to reducing or diverting water flow, the bulk of the design requirements relate to hydrologic properties such as hydraulic conductivity. Additional design requirements considered in Chapters 6 and 7 concern strength, density, emplacement considerations, longevity, and cost.

The design requirements are developed at this stage in the design development process as guidelines for design analyses and future materials testing. Revisions and additional design requirements are expected to emerge from the Advanced Conceptual Design (ACD) and during the License Application Design (LAD).

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*A performance goal is a restriction placed on the sealing subsystem to achieve a desired level of performance. Here, our primary intent is to present sealing design options that will limit the amount of water entering the waste disposal areas or will control water in waste disposal areas.

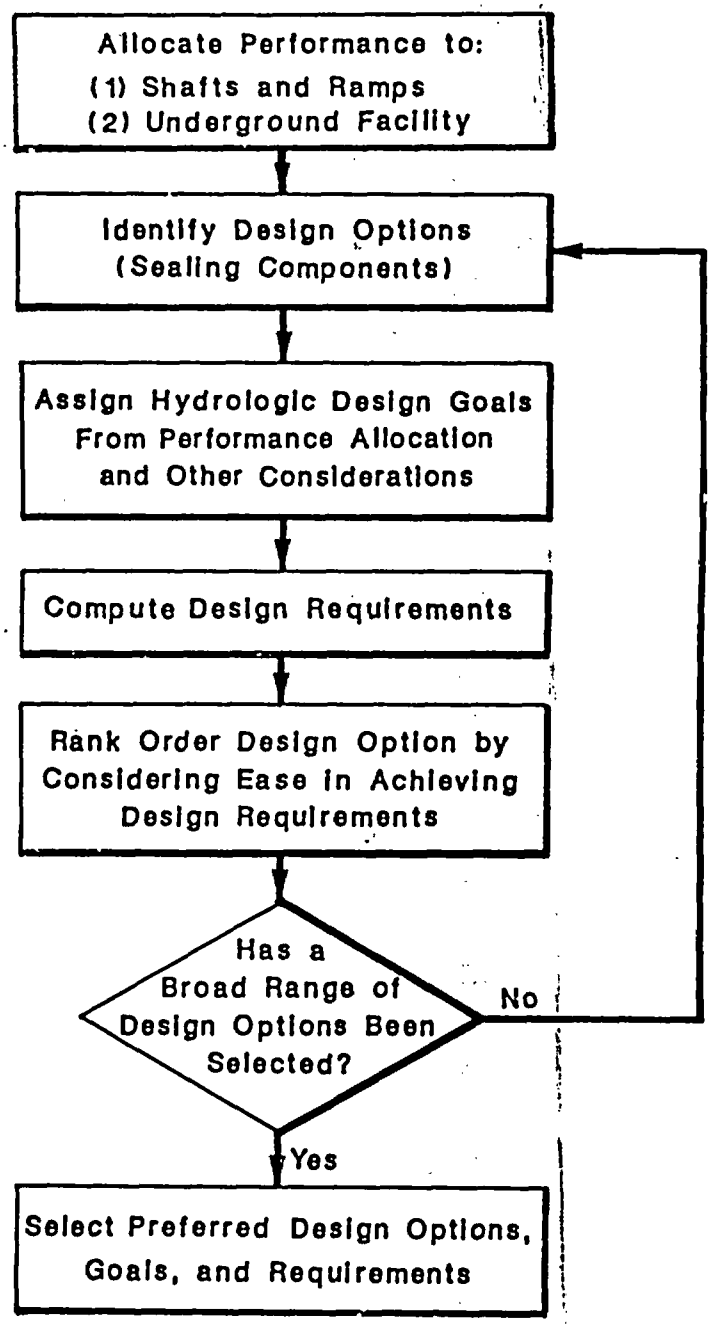


Figure 6-1. Logic Used to Select Preferred Design Options, Goals, and Requirements

6.1 PERFORMANCE ALLOCATION

In Chapter 3, hydrologic performance goals for the sealing subsystem are developed. In this section, these performance goals (for specific times following closure) are allocated between two major portions of the sealing subsystem: sealing components within the shafts and ramps and sealing components within the underground facility. This allocation is made such that the allocation to the shafts and ramps plus the allocation to the underground facility is equal to the performance goal for each specified year. Mathematically, the relationship can be expressed as

$$T(t) = S(t) + U(t)$$

where

$T(t)$ = performance goals for the entire sealing subsystem,
$S(t)$ = performance goals allocated to seals in the shaft/ramp subsystem,
$U(t)$ = performance goals allocated to the underground facility subsystem, and
t = time.

A comparison of the functions $T(t)$, $S(t)$, and $U(t)$ are shown schematically in Figure 1-4. When allocating performance, the designer considers the potential difficulty in achieving the performance of one of the sealing subsystems (i.e., underground facility) in relation to the other subsystem. In determining the potential difficulty, the following were considered:

- the difference in uncertainty in the number of sealing components required to achieve the performance goal for the underground facility and the performance goal for the shafts and ramps. In the underground facility, the Ghost Dance Fault intersects approximately 24 waste emplacement rooms for the vertical emplacement configuration and approximately 100 horizontal emplacement boreholes (see Section 4.2.2 and Appendix E) for the horizontal emplacement configuration. The conditions at any or all of these locations, plus possibly additional locations, may require sealing. Contrary to this uncertainty, the numbers and locations of the shafts and ramps are currently defined together with their relationship to the underground facility. The number and types of sealing components can thus be defined with more certainty for the shafts and ramps than for the underground facility.

- the site conditions that must be evaluated to achieve some desired performance. In the underground facility

the sealing components, will be subjected to higher temperatures and stresses than those components in the shafts and ramps.

These considerations suggest a comparatively greater difficulty in achieving a desired performance for sealing components in the underground facility, as compared to sealing components in the shafts and ramps. Because of the greater uncertainty in the number and locations of seals in the underground facility, and the higher temperature environment, we have elected to allocate all of the total performance goal [T(t)] initially to the underground facility [U(t)]. By initially allocating to the underground facility, there is greater flexibility in the underground facility seal design in achieving the higher performance goal.

The maximum amount of the performance goal that is allocated to the underground facility is the maximum, ground-water inflow computed to enter the underground facility from the matrix and fractures. For vertical and horizontal emplacement, the amount is 5,600 m^3/year. Allocating more of the performance goal to the underground facility is not required if these volumes of water are considered to be upper-bound values.

The designer may specify, within a limited degree, the time period and degree of performance bearing in mind that a relaxation in the design goal for one seal subsystem results in a more stringent design goal for the other. It is noted that the expected performance for each seal subsystem may change as site characterization information on unanticipated flow scenarios becomes available, and that an alternative performance allocation may then be selected. It is further noted that the two, major, seal subsystem components might be further subdivided to reflect differences among individual shaft/ramp or underground facility locations. For example, one shaft may be placed in a more unfavorable location than another, and have an unfavorable performance (high unanticipated flow). This might be compensated by specifying a larger performance allocation for this individual shaft.

Table 6-1 presents the current performance allocation to the underground facility and the shafts/ramps. All of the total flow is allocated to the underground facility until the total flow equals the maximum unanticipated flow and specific performance from the seal components is not necessary. For vertical emplacement and horizontal emplacement, this match occurs at about 500 years following closure. Following this time

- the required performance allocated to the underground facility remains fixed for future years,

- specific performance from seal components within the underground facility is not necessary to reduce the maximum computed inflows beyond 500 years after closure, and

- the difference between the total performance goals [T(t)] beyond 500 years and the allocation of 5,600 m^3/year to the underground facility [U(t)] represents the performance that can be allocated to the shafts and ramps [S(t)].

Table 6-1. Performance Allocation for the Sealing Subsystem

Time After Closure, years	Performance Allocation, m^3/year				
	Underground Facility	+	Shafts and Ramps	=	Total Performance Goals (Table 3-2)
0 - 300	1,180	+	0	=	1,180
400	2,600	+	0	=	2,600
500	5,600[a]	+	90	=	5,690
600	5,600	+	6,700	=	12,300
700	5,600	+	20,900	=	26,500
800	5,600	+	50,300	=	55,900
900	5,600	+	110,400	=	116,000
1,000	5,600	+	130,400[a]	=	136,000
1,000 - 10,000	5,600	+	130,400	=	136,000

[a]No specific performance required of seal components beyond this point in time.

Using the same logic as applied to the underground facility, performance can be allocated to the shafts and ramps until the performance allocated to the shafts and ramps is equal to the maximum computed inflow (83,700 m^3/year) into all the shafts and ramps. This match occurs before 1,000 years following closure. After this point in time, specific performance of seals within the shafts and ramps is not needed to reduce the maximum computed inflows. Therefore, the specific sealing measures that are emplaced within the shafts and ramps to control flow between zero and 1,000 years following closure are not needed beyond 1,000 years. The seals that are emplaced at closure would, of course, remain in place over the long term together with the shaft and ramp fill.

In general, the time period in which sealing is required is proposed on (1) the maximum computed inflows into the underground facility and the shafts and ramps, and (2) the performance goals. If these values change as part of the design process, the time period over which sealing is proposed will also change.

6.2 HYDROLOGICAL DESIGN GOALS FOR SEAL COMPONENTS

The primary emphasis in this chapter is placed on the selection of hydrological design goals and requirements because the flow of water relates directly to seal subsystem performance. This section presents these design goals on a component basis, and develops the philosophy towards their selection.

Water that enters the underground facility might either flow towards the waste emplacement rooms and potentially contact the waste or alternatively be drained through the floor of the repository before contacting the waste. For example, seals in emplacement rooms might isolate flow from a fault. Any flow occurring through the seals enters directly into the waste emplacement room and might contact the waste. Alternatively, water might enter the perimeter drift downgrade from the waste emplacement areas and not contact the waste provided there is adequate floor drainage.

For planning purposes in selecting design goals for the seal system, it is assumed that water flowing through shafts and ramps to the underground and water flowing through emplacement drifts above specified minimum volumes could potentially contact the waste. These inflows are subject to performance allocation. It has been further assumed that water flowing from faults into non-emplacement drifts is sufficiently isolated, and would not contact the waste if adequate floor drainage is provided. In the former case, the design goals are established through the performance allocation process to arrive at design requirements for specific seal options (see Section 6.2.1). In the latter case, leakage through the seal system is restricted to a small percentage of the floor drainage capacity (see Section 6.2.2).

6.2.1 Design Goals Based on Performance Allocation

For performance allocation between the shaft/ramp and underground facility seal subsystems, the performance goals are modified to account for storage at the repository horizon:

$$S_m(t) = S(t) + C_s(t)$$

$$U_m(t) = U(t) + C_u(t)$$

where

$S_m(t)$ = modified performance goal for the shaft and ramp subsystem,
$U_m(t)$ = modified performance goal for the underground facility subsystem,
$S(t)$ = performance goals allocated to seals in the shaft and ramp subsystem,
$U(t)$ = performance goals allocated to the underground facility subsystem,
$C_s(t)$ = storage capacity for the shaft and ramp seal subsystem, and
$C_u(t)$ = storage capacity for the underground facility subsystem.

In the above relations, the performance goals are modified by the potential storage volume that could be drained as discussed subsequently. For planning purposes, it is assumed that 10,000 m^3 of water could be stored at the lowest point of the repository for shaft/ramp inflows. For the underground facility, the storage capacity is assumed to be zero.

The design goals for the two seal subsystems are evaluated at repository closure, and at the end of the sealing period (the point in time when specific seal performance is not needed). The design goals after repository closure may be stated as

$$DG_a(t_o) = \frac{S_m(t_o)}{X_a}$$

or

$$DG_a(t_o) = \frac{U_m(t_o)}{X_a}$$

where

$DG_a(t_o)$ = design goal of a specific seal component at time t_o,
X_a = number of proposed components, and
t_o = time at repository closure.

At the end of the sealing period, the design goals may be stated as

$$DG_a(t_f) = \frac{S_m(t_f)}{X_a}$$

or

$$DG_a(t_f) = \frac{U_m(t_f)}{X_a}$$

where

t_f = time at the end of the sealing period.

The shaft/ramp storage volume $C_s(t)$ is established at the lowest point of the repository near the Emplacement Exhaust Shaft (EES). Water that infiltrates through a shaft must first fill the sump before it overflows into the shaft station area. Any overflow would enter the access drifts and could possibly enter emplacement drifts, depending on the grades of the drifts. With relatively simple measures such as drainage channels or earthen dams, however, the water could be diverted away from emplacement drifts (Section 6.5). Ultimately, any water that enters the repository is expected to drain through the floor or will collect in the lowest part of the repository near the EES.

The following analysis estimates the amount of water accumulating in the lowest point of the repository before contacting the waste. As the water level in this area rises, water first contacts the waste (approximately) at elevation 897.6 m (2,945 ft) for horizontal emplacement (Figure 6-2) or 900.6 m (2,955 ft) for vertical emplacement (Figure 6-3). Assuming a backfill porosity of 30%, the water volumes stored in this area are 3,300 m^3 and 6,800 m^3, respectively. Considering a water volume of 10,000 m^3 the repository would flood up to an elevation of 903 m (2,963 ft). At this elevation, the number of emplacement holes flooded would be about 42 for vertical emplacement or 15 for horizontal emplacement. This represents only 0.1% and 0.6%, respectively, of the total waste inventory.

This analysis indicates that large volumes can potentially enter the repository through shafts/ramps without contacting the waste. And, depending on where the water enters the underground facility, a storage capacity $C_u(t)$ could also be established for the underground facility.

6.2.2 Design Goals Based on Storage Capacity and Drainage

For seals not affected by hydrologic performance goals in Section 6.1, the design goals are established to restrict flow through the seal to 10% of the drainage capacity of the floor area behind the seal. This design will assure that flow in nonwaste emplacement areas is dominantly vertical infiltration through the floor, and not lateral flow to waste emplacement areas or the lowest point of the repository.

6.2.3 Summary of Hydrologic Design Goals

A summary of the design goals for various components or design options of the sealing subsystem are summarized in Tables 6-2 and 6-3. The proposed, design goals are used for the selection of hydrologic design requirements for specific design options contained in the following sections of this chapter. To facilitate the selection of design requirements, Appendix F provides a series

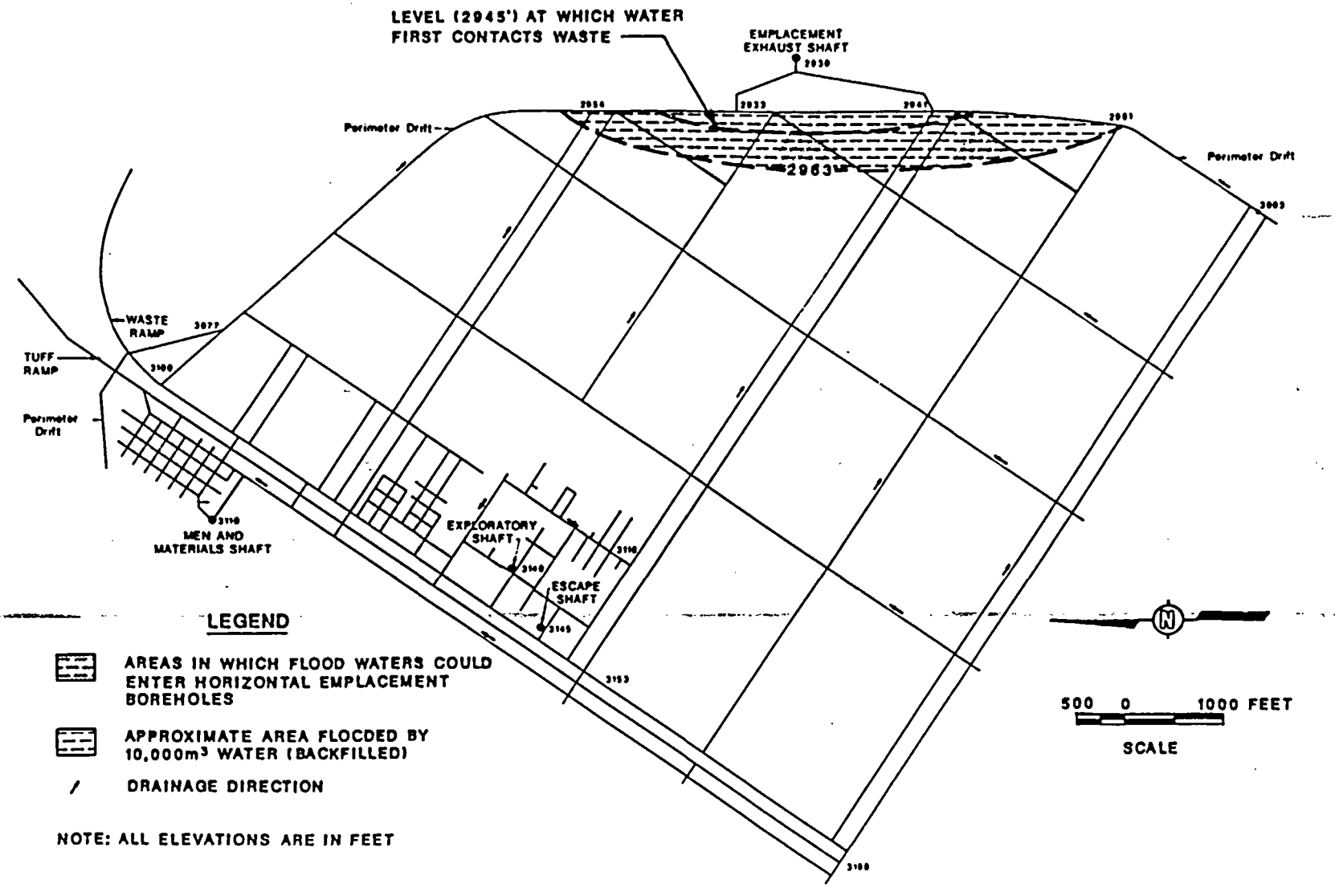


Figure 6-2. Area of the Repository Flooded by 10,000 m^3 Water - Horizontal Emplacement

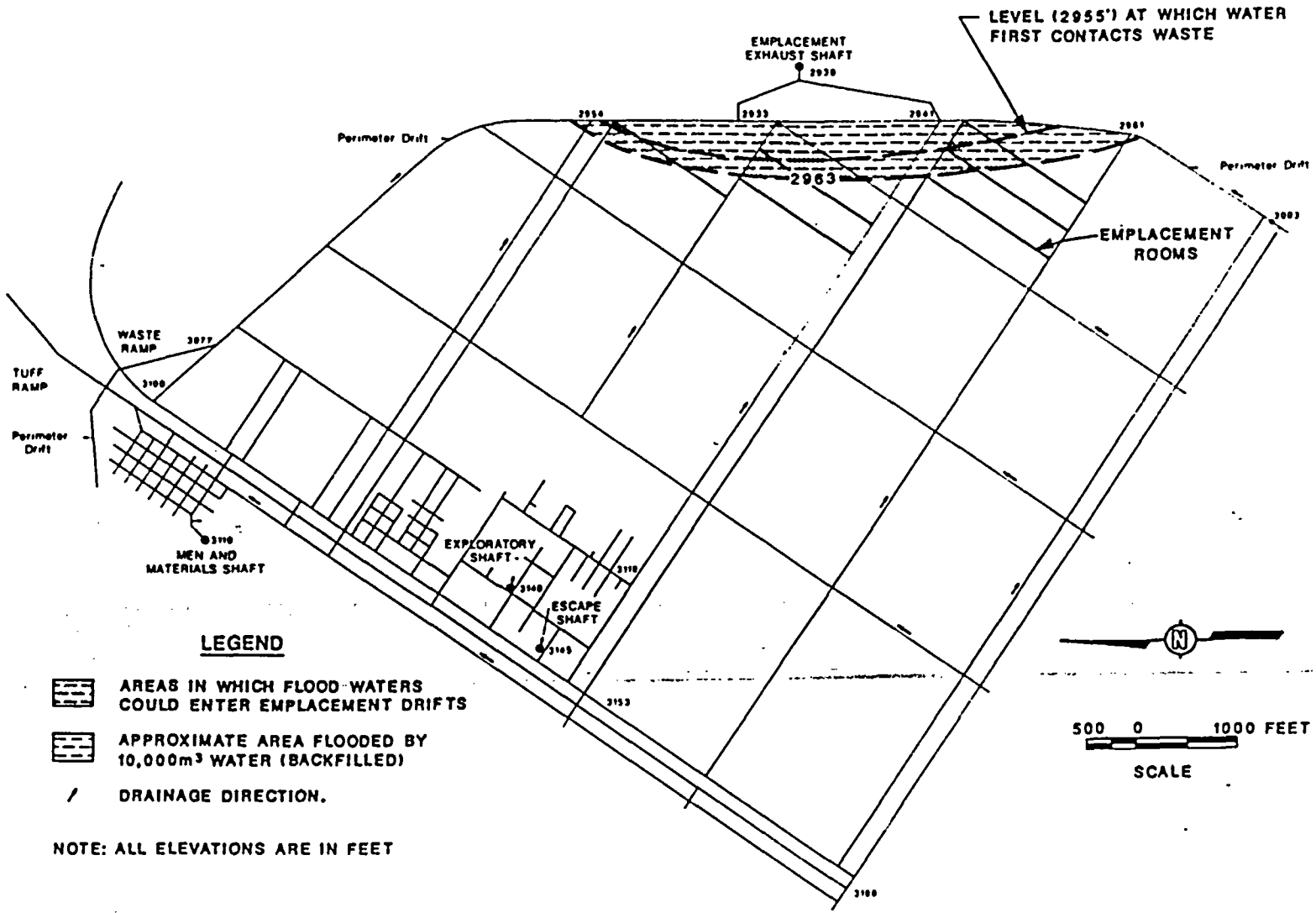


Figure 6-3. Area of the Repository Flooded by 10,000 m^3 Water - Vertical Emplacement

Table 6-2.[a] Summary of Design Goals - Shaft/Ramp Seal Subsystems

Component	Performance Goals $S(t_o)$	Performance Goals $S(t_f)$	Storage Capacity $C_s(t)$	Modified Performance Goals $S_m(t_o)$	Modified Performance Goals $S_m(t_f)$	Number of Components X_s	Design Goals: After Closure $DG_s(t_o)$	Design Goals: End of Sealing Period $DG_s(t_f)$	Reference Section
Anchor-to-Bedrock Plug	0	130,000	10,000	10,000	140,000	6	1,700	23,000	6.3, 6.4
Shaft/Ramp Station Plug	0	130,000	10,000	10,000	140,000	10	1,000	14,000	6.3, 6.4
Shaft Fill Ramp Fill	0	130,000	10,000	10,000	140,000	6	1,700	23,000	6.3

[a]Goals and storage capcities are in units of m^3/year.

Table 6-3.[a] Summary of Design Goals - Underground Facility Subsystem

Component	Performance Goals $U(t_o)$	Performance Goals $U(t_f)$	Storage Capacity $C_u(t)$	Modified Performance Goals $U_m(t_o)$	Modified Performance Goals $U_m(t_f)$	Number[b] of Components X_u	Design Goals: After Closure $DG_u(t_o)$	Design Goals: End of Sealing Period $DG_u(t_f)$	Reference Section
Single Dam or Single Bulkhead in Emplacement Drifts	1,180	5,600	0	1,180	5,600	25	47	220	6.5
Perimeter Drifts	-	-	350	-	-	-	Restrict flow to 10% of drainage behind the structure		6.5
Main Drifts	-	-	620	-	-	-	Restrict Flow to 10% of drainage behind the structure		6.5
Double Bulkheads in Emplacement Drifts (no settlement)	1,180	5,600	0	1,180	5,600	50	24	110	6.5
Double Bulkheads in Emplacement Drifts (settlement)	1,180	5,600	0	1,180	5,600	50	24	110	6.5
Plug in an Emplacement Hole	1,180	5,600	0	1,180	5,600	100	12	56	6.6

[a]Goals and storage capacities are in units of m^3/year.
[b]Number of components based on current designs or assumptions made for the number of water-producing zones encountered in emplacement drifts.

of design charts with design goals at the time of repository closure and at the end of the sealing period.

6.3 DESIGN REQUIREMENTS FOR SHAFT SEALS

6.3.1 Functional Requirements

The primary functions for shaft seals are as follows:

- reduce the potential for surface water or ground water to enter the waste emplacement areas via the shafts, and

- deter human entry to the repository via the shafts.

For the unanticipated scenario postulated (Section 4.1.5), as much as 83,700 m^3 of water could enter each of the shafts if no shaft seals (other than backfill) are emplaced. Small amounts of ground water could also enter the shafts through fractures (Section 4.1.2).

The potential for casual or inadvertent human entry to the repository can be reduced by filling or sealing the shafts and by marking the location of the repository and its purpose. Methods of warning future populations of the hazards associated with a repository have been evaluated by a DOE Task Force (ONWI, 1984). This group recommended installation of monuments around the repository perimeter as well as a central monument that would house written and pictorial messages. The task force did not address whether messages should be left at specific shaft locations. To our knowledge, there is no regulatory guidance addressing this topic.

It may be noted that marking a shaft location with a prominent, durable monument could be inconsistent with the goal of deterring human entry because the monument would draw attention to the location of access points. The current position adopted in the NNWSI Repository Sealing Program is that there is no necessity to mark the surface location of the shaft. Rather, it is preferable to restore the surface location to its original condition. Human entry will be discouraged by fill and seals placed below the ground surface. Further, no special high-strength or abrasive materials will be used to deter entry on the grounds that any group intent on mining into the repository could easily mine through such materials.

6.3.2 Design Options

The functional requirements for shaft seals may be satisfied by one or more seal components used in combination. These components, as identified by Fernandez and Freshley (1984) include the following:

- surface cover (incorporating the collar fill),

- anchor-to-bedrock plug,
- shaft fill,
- shaft plug (at intermediate depth in the shaft or at the base above the shaft station), and
- station plug (in drifts adjacent to the shaft station).

Any of these components could reduce the amount of water that could enter the repository. A surface cover would directly prevent surface water from entering the shaft, but a disadvantage of this component is that it would be located in an erosional environment where durability over hundreds to thousands of years would be questionable. As noted previously, it is not considered necessary to provide a durable surface cover such as a monument. Flow through the shaft can be reduced by fill placed along the length of the shaft or by one or more seals (plugs) placed at intervals. Fill alone may not be a satisfactory option if there is the potential for flow through a modified permeability zone (MPZ) adjacent to the shaft wall. In such a case, it would be necessary to form a key way through a portion of the MPZ possibly by keying a plug into the walls (Lundstrom et al., 1986). In this concept, the most severely damaged zone is removed at the plug location and the plug is "keyed" into the rock to replace a highly fractured material with a low permeability material.

6.3.3 Design Goals and Design Requirements for Shaft Seal Components

The hydrologic design goals may be achieved by a shaft plug or shaft fill. The design requirements are derived from the hydrologic design goals (Section 6.2) and the design charts for flow through a plug placed at the base of the shaft or near the top of the shaft (Appendix F, Figures F-3 and F-4).

A design requirement is obtained by dividing 10,000 m^3 (modified performance goal, Table 6-2) by the number of seals required to seal all of the shafts and ramps. If the seals are placed in the shafts, the number of seals required is six (four shafts plus two ramps). In this case, if the allocation for each shaft and ramp is 1,700 m^3/year (i.e., 10,000 m^3/year ÷ 6), the design requirement for the effective hydraulic conductivity of an anchor-to-bedrock plug in a shaft is about 10^{-5} cm/s (Figure F-4). At the end of the sealing period the design goal is 23,000 m^3/yr and the design requirement is 10^{-4} cm/s. If the seals are placed in drifts adjacent to the shafts, the number of seals required is 8 for shafts plus 2 for ramps, a total of 10. In this latter case, the design goal for each seal is 1,000 m^3/year and the design requirement for hydraulic conductivity is about 10^{-6} cm/s (Figure F-3) from zero to 500 years. At the end of the sealing period the design goal is 14,000 m^3/year and the design requirement is 10^{-5} cm/s.

For shaft fill, a general design goal is the reduction of air and water flow. As discussed in Section 3.2 and illustrated in Figures C-7 through C-10 the air flow rate through the shafts can be reduced to less than 2.5% and 5.0% of the total airflow through the shafts and the rock mass (for vertical and horizontal emplacement respectively) if the air conductivity is 3×10^{-4} m/min (or 10^{-2} cm/s effective hydraulic conductivity). These values are less than the performance goal of 25% given in Section 3.2. To achieve this airborne performance goal and to provide an additional level of conservatism, a design requirement of 10^{-2} cm/s (saturated, effective hydraulic conductivity) is selected for the shaft fill.

To assess the effectiveness of shaft fill to reduce water flow through the shaft, a design chart was prepared (Figure F-10). This chart can be used to obtain design requirements for selected design goals and to illustrate the potential influence of an MPZ. For example, to achieve a hydrologic design goal of 1,700 m^3/yr (as developed above), shaft fill with a hydraulic conductivity of about 10^{-4} cm/s can be specified provided flow through the MPZ is not significant. At the end of the sealing period the design goal is 23,000 m^3/year and the design requirement is 10^{-3} cm/sec. Conversely, if an MPZ did exist as defined by cases 1 through 4 in Figure F-10, emplacement of shaft fill would not be effective in achieving the design goal of 1,700 m^3/yr from 0 to 500 years.

6.3.4 Preferred Options for Shaft Seals

The design requirements for the various optional shaft seal components are summarized in Figure 6-4, and the design analyses and associated input data required for each component are listed in Table 6-4. The preferred option for reducing water flow and deterring human entry is the anchor-to-bedrock plug for the following reasons:

- The anchor-to-bedrock plug is located in a relatively benign environment protected from surface temperature extremes, surficial geologic processes and from the heat generated by the waste. Conversely, any surface cover or monument placed over a shaft would have to withstand the maximum expected flood and debris flow. Additionally, the surface climatic and geochemical environment, including extreme diurnal temperature variations and the possibility of sulphate-rich water deleterious to anticipated concretes, would also have to be considered. Station plugs are isolated from the waste emplacement areas by barrier pillars, but the maximum temperatures could potentially reach or even exceed 100°C. The in situ stress would be greater than that associated with a plug closer to the surface.

- The design requirement for the anchor-to-bedrock plug is less stringent than that for a seal at the base of the shaft because of the lower, maximum hydraulic head.

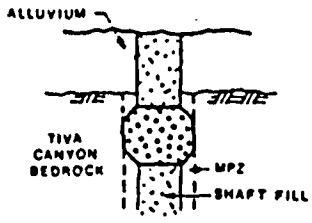
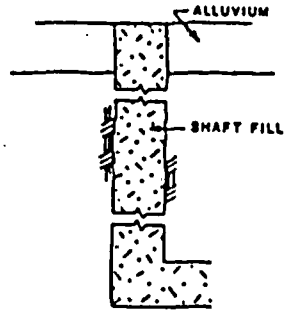
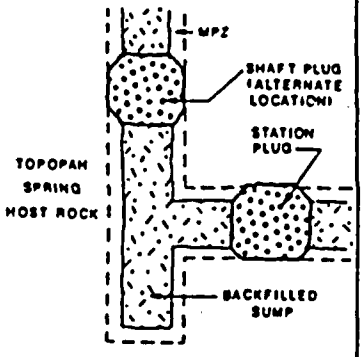
DESIGN OPTIONS	DESIGN GOALS	DESIGN REQUIREMENTS
 ANCHOR-TO-BEDROCK PLUG	• Limit surface waters entering shaft to 1,700 m^3/yr from 0 to 500 yrs and 23,000 m^3/yr at the end of the sealing period (1,000 years). • Deter human entry.	• Construct an anchor-to-bedrock plug having an effective hydraulic conductivity (including interface zone and MPZ) 10^{-5} to 10^{-4} cm/s. • Structural requirements to be determined by ACD.
 GENERAL SHAFT BACKFILL	• Reduce potential for water and air flow.	• Emplace shaft fill having a saturated hydraulic conductivity $\leq 10^{-2}$ cm/s.
 STATION AND SHAFT PLUGS	• Provide structural support of shaft backfill (station and shaft plugs in shaft) if necessary. • Limit surface and subsurface waters from entering the underground facility to 1,000 m^3/yr (station plug) from 0 to 500 yrs and 14,000 m^3/yr at the end of the sealing period (1,000 years).	• Structural requirements to be determined by ACD. • Effective hydraulic conductivity (including interface zone and MPZ) 10^{-6} to 10^{-5} cm/s.

Figure 6-4. Summary of Design Requirements for Potential Design Options - Shafts

Table 6-4. Design Analyses and Properties Required for the Shaft Seals

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[b]
Surface Cover[a]		
• Foundation Stability/ Erosion of Alluvium Colluvium		Shaft Liner/Collar Design
- Erosion by Stream Flow	Maximum Stream Flow Rate	
- Bearing Capacity and Settlement of Alluvium	Mechanical Properties of the Alluvium Depth to Bedrock	
- Liquefaction of Alluvium	Hydrologic Properties of the Alluvium	
- Seismic	Horizontal and Vertical Acceleration	
• Structural Stability of Cover		Shaft Liner/Collar Design
- Seismic	Horizontal and Vertical Acceleration	
- Thermal	Diurnal Temperature Variation Extreme Temperature Variation Increased Temperature Due to Curing	
- Gravitational and Hydraulic	Weight of the Slab and Maximum Imposed Load Stream or Debris Flow	
• Chemical Durability	Chemistry of the Alluvium	

(a) The current position of the NNWSI Repository Sealing Program is that surface covers will not be emplaced.

(b) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

Table 6-4. Design Analyses and Properties Required for the Shaft Seals (Continued)

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[b]
Anchor-to-Bedrock Plug		
• Flow Through Plug and MPZ	Saturated Hydraulic Conductivity of the Tiva Canyon	Shaft Diameter
	MPZ Model for the Tiva Canyon	
• Foundation Stability/ Erosion of the Alluvium/Colluvium	Mechanical Properties of the Tiva Canyon	
• Structural Stability of the Plug		Shaft Liner Design
- Seismic	Horizontal and Vertical Acceleration	
- Thermal	Diurnal Temperature Variation	
	Increased Temperature Due to Curing	
	Extreme Temperature Variation	
- Gravitational	In Situ Stresses in the Tiva Canyon	Shaft Depth at Plug Location
• Chemical Durability	Geochemistry of the Tiva Canyon	
Shaft Fill		
• Earth Pressure Within Backfill	Angle of Internal Friction	Shaft Depth Shaft Diameter
• Settlement	Coefficient of Compressiblity	
• Particle Segregation	Quantity of Seepage Water	

(b) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

Table 6-4. Design Analyses and Properties Required for the Shaft Seals (Concluded)

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[b]
Station Plug		
• Flow Through Plug and MPZ and Sump Drainage Capacity	Saturated, Bulk Rock, Hydraulic Conductivity of Topopah Spring MPZ Model for the Topopah Spring	Depth of Shaft at Shaft Station Elevated Temperature due to Radioactive Heat Generation
• Silting Potential in Sump	Shaft Fill Characteristics	
• Structural Stability of the Plug	Fracture Characteristics Backfill Characteristics	Drift Dimensions at Station Plug and Shaft Diameter
- Seismic	Horizontal and Vertical Acceleration	
- Thermal	Geothermal Temperature	Elevated Temperatures at Base of Shaft Due to Radioactive Heat Generation
- Gravitational/ Hydrostatic	Earth Pressure Within the Shaft Fill In Situ Stress in the Topopah Spring Water Pressure Within the Shaft Fill	Shaft Depth at Plug Location

(b) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

- Only one seal is required for each shaft, making a total of four, whereas eight total seals are required if seals are placed in the shaft stations.

- Construction of a seal at shallow depth in a shaft (about 10 m) should be easier and cheaper than construction at the base of the shaft. If necessary, the alluvium can be stripped away to facilitate construction of the anchor-to-bedrock plug.

- The anchor-to-bedrock plug can be designed to reduce the potential for flow through the MPZ, whereas simple placement of shaft fill would have no influence on the MPZ. Moreover, development of the MPZ at the shallow depth of the anchor-to-bedrock plug should be less than that at the station plug location where inelastic deformations are more likely to occur (Case and Kelsall, 1987).

6.4 DESIGN REQUIREMENTS FOR RAMP SEALS

6.4.1 Functional Requirements

The primary functions for ramp seals are the same as those for shaft seals:

- reduce the potential for surface water or ground water to enter the waste emplacement areas via the ramps, and

- deter human entry to the repository via the ramps.

As with the shafts, there is a potential for surface flood waters to enter the ramps, although this potential is alleviated by locating the portals on hill slopes above the present-day water courses (Appendix A-1). Compared to the shafts, there is a greater potential for ground water to enter the ramps through fractures or faults such as the Bow Ridge Fault, which intersects the waste ramp, or faults below Drill Hole Wash, which intersects both ramps (Figure 2-2).

The requirements for sealing ramps to deter human entry are the same as for shafts. While the location of ramps could be marked by monuments, the current position of the sealing program is that the ground surface should be restored to its original condition. Human intrusion will be deterred by an anchor-to-bedrock plug placed in the bedrock.

6.4.2 Design Options

The seal components proposed for ramps (Fernandez and Freshley, 1984) are similar to those for shafts:

- surface cover,
- anchor-to-bedrock plug,
- ramp fill, and
- ramp plug or dam (placed at intermediate depths, probably downgrade from faults).

For ramps, the surface cover and anchor-to-bedrock plug will probably be combined into a single component because the unconsolidated materials above the bedrock are thin.

Water flow into the repository can be reduced either by fill placed along the length of the ramps or by a plug placed at the bottom of the ramp (equivalent to the shaft station plug) or at some point in the ramp down grade from possible inflow sources. Backfill alone may not be a satisfactory option if there is the potential for flow through an MPZ in the floor or walls. In such a case, it would be necessary to install a plug with keyways or cutoffs excavated into the adjacent rock.

6.4.3 Design Goals and Design Requirements for Ramp Seal Components

The hydrologic design goals may be achieved by ramp plugs. The design requirements are derived from the hydrologic design goals (Section 6.2) and the design charts for flow through a ramp plug (Appendix F, Figure F-5). The modified hydrologic design goal accounts for the storage volume of the lower part of the repository near the EES. From Section 6.2, the modified hydrologic design goal is equal to the volume which can be allocated equally to 10 seals, one for each ramp and eight shaft seals. In this case, the allocation for each plug is 1,000 m^3/yr, and the design requirement for hydraulic conductivity is 10^{-6} cm/s over the period from 0 to 500 years. At the end of the sealing period (1,000 years), the design goal is 14,000 m^3/yr, and the design requirement is 10^{-5} cm/s.

6.4.4 Preferred Options for Ramp Seals

As noted above, the option of a surface cover separate from an anchor-to-bedrock plug does not pertain to ramps because of the thin sedimentary cover above the bedrock. Another difference regarding sealing shafts and ramps is that the more likely water sources in ramps are subsurface faults, whereas the more likely source for shafts is surface flooding.* For ramps the main function of the anchor-to-bedrock plug will be to deter human entry, while the

*For the proposed (new) locations of ES-1 and ES-2, the potential for water entering the surface is substantially reduced from the original locations.

function of reducing water will be provided by plugs located at or near the base of the ramps.

The main concern will be the need to reduce the potential for flow through the interface and MPZs. Design requirements for the optional ramp seal components are summarized in Figure 6-5 and the required design analyses are listed in Table 6-5. For ramp seals, the preferred option is the ramp seal placed just below the points intersected by vertical or nearly vertical faults if these represent potential water conducting zones. In such locations, the seal could intercept fault flow, and perhaps retain some of the advantages of near-surface emplacement cited previously.

6.5 DESIGN REQUIREMENTS FOR SEALS IN THE UNDERGROUND FACILITY - VERTICAL EMPLACEMENT

6.5.1 Functional Requirements

The function for seals located in the underground facility (vertical emplacement) is to divert the flow of any water that may enter the facility away from the waste. It was noted in Section 4.2.4 that only a small volume of water (62 m^3/year) is expected to enter the repository under anticipated conditions. Under unanticipated conditions, larger volumes of water (5,600 m^3) could enter from faults and matrix sources (Section 4.2.5).

Regardless of any seal design options that are placed in the underground facility, the current planning basis is that the drifts will be backfilled with crushed tuff. The backfill will prevent large-scale collapse of drifts that could lead to changes in the hydraulic conductivity of the rock mass overlying the repository and possibly increased water inflow.

6.5.2 Design Options

The present design considers several design options for underground seal components that would satisfy the design goals for seals emplaced in the underground facility. Seal components could be placed in nonemplacement areas to enhance vertical drainage prior to water reaching low points of the repository. If water is encountered in emplacement drifts, our current strategy is to abandon the drift. We, nevertheless, present design options that could be used in emplacement drifts if maximization of waste storage areas is considered desirable. The design options for the vertical emplacement configuration include:

- sumps and drains excavated below or adjacent to faults,
- grouting of fault zones,
- fault seals (plugs or bulkheads, used to seal off a potential inflow zone),

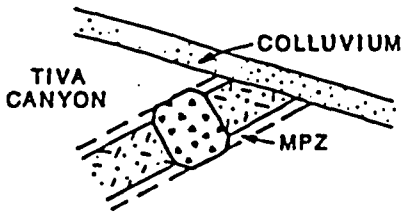
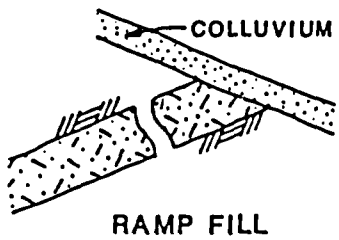
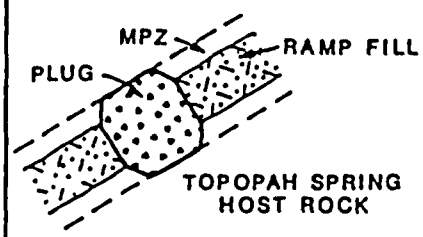
DESIGN OPTIONS	DESIGN GOALS	DESIGN REQUIREMENTS
 ANCHOR-TO-BEDROCK PLUG	• Limit surface waters entering ramp to 1,700 m^3/yr from 0 to 500 years and 23,000 m^3/yr at the end of the sealing period (1,000 years). • Deter human entry.	• Construct an anchor-to-bedrock plug having an effective hydraulic conductivity (including interface zone and MPZ) 10^{-5} to 10^{-4} cm/s. • Structural requirements to be determined by ACD.
 RAMP FILL	• Reduce potential for water and air flow.	• Emplace ramp fill having a saturated hydraulic conductivity $\leq 10^{-2}$ cm/s.
 RAMP PLUG	• Limit inflow from faults draining into waste emplacement areas to 1,000 m^3/yr from 0 to 500 years and 14,000 m^3/yr at the end of the sealing period (1,000 years).	• Construct a ramp plug having an effective hydraulic conductivity (including interface zone and MPZ) 10^{-6} to 10^{-5} cm/s.

Figure 6-5. Summary of Design Requirements for Potential Design Options - Ramps

Table 6-5. Design Analyses and Properties Required for Ramp Seals

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[a]
Anchor-to-Bedrock Plug		
• Flow Through Plug and MPZ	Saturated Hydraulic Conductivity of the Tiva Canyon	Ramp Diameter
	MPZ model of the Tiva Canyon	
• Foundation Stability	Mechanical Properties of the Tiva Canyon	
• Structural Stability of the Plug		
- Seismic	Horizontal and Vertical Acceleration	
- Thermal	Diurnal Temperature Variation	
	Extreme Temperature Variation	
	Increased Temperature During Curing	
- Gravitational	In Situ Stresses in the Tiva Canyon	Ramp Depth at Plug Location
• Chemical Durability	Geochemistry of the Tiva Canyon and the Colluvium	
Ramp Fill		
Earth Pressure Within Backfill	Angle of Internal Friction	Ramp Depth Ramp Diameter

(a) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

Table 6-5. Design Analyses and Properties Required for Ramp Seals (Concluded)

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[a]
• Settlement	Coefficient of Compressibility	
Ramp Plug		
• Flow Through Plug and MPZ, and Sump Drainage Capacity	Saturated, Bulk Rock Conductivity of the Topopah Spring	Length of Ramp, Depth at Base of Ramp
	MPZ Model for the Topopah Spring	Elevated Temperature due to Radioactive Heat Generation
• Silting Potential	Fracture Characteristics Backfill Characteristics	
• Structural Stability of the Plug	Fracture Characteristics Backfill Characteristics	Drift Dimensions at Ramp Seal and Ramp Diameter
- Seismic	Horizontal and Vertical Acceleration	
- Thermal	Geothermal Temperature	Elevated Temperatures at Base of Ramp Due to Radioactive Heat Generation
	Increased Temperature Due to Curing	
- Gravitational/ Hydrostatic	Earth Pressure Within the Ramp Fill	Ramp Depth at Plug Location
	In Situ Stress in the Topopah Spring	
	Water Pressure Within the Ramp Fill	

(a) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

- drainage channels in nonemplacement drifts, and
- abandon the hole for disposal.

Design features of the repository, although not seal design options, could be used to facilitate flow direction and drainage such that some of these seal design options would not be necessary. For example, drift grades could be specified to facilitate diversion of flow away from waste emplacement drifts. As water flows downgrade through the emplacement drift, it is intercepted by steeper panel entries that would divert flow away from the adjacent emplacement drift downgrade. Similarly, the main entries could be designed to have steeper grades than adjacent panel entries to divert flow away from emplacement waste areas.

The diversion of flow away from waste emplacement areas might be facilitated by open channel construction. Such channels placed at points where entries intersect would provide a more positive means of subsurface-water diversion around corners. Such channels might also be placed in areas where entry grades are unfavorable to subsurface-water diversion.

6.5.3 Design Goals and Design Requirements for Seals in the Underground Facility

A consideration in assigning design goals and requirements for seals in the underground facility is that the seals should be able to accommodate a wide range of inflows. Several types of components may be used at different points in the repository, with each type designed for a specific rate and total volume of inflow. Selection of the type of component for a specific fault intersection would be based on the geological conditions and the observed inflow at the time of construction and during operation. If it is considered desirable to maximize the waste storage capacity, a second consideration in the design of seals could be to minimize the length of sumps or bulkheads. However, our current strategy is to abandon drifts in which the inflow occurs. Observations of water inflows during the construction and operation of the Exploratory Shaft Facility will be used to refine this strategy.

6.5.3.1 Sumps

The simplest design option for retaining water inflowing from a fault is a sump excavated below the inflow zone (or downgrade in the drift). The volume of the sump, will be designed to accommodate flow from a single event. This inflow will be based in part on observations made during construction. For present planning purposes, we assume that the depth of the sump should be limited to 1 m (for ease of construction), in which case for a drift grade of 7.5% the volume is about 5 m^3 (Figure 6-6). The length of the sump is about 7 m and the drainage capacity in 1 year is 100 m^3, based on a floor hydraulic conductivity of 10^{-5} cm/s. The sump can thus handle as much as 5 m^3 in one event or 100 m^3 over an entire year (for perspective, 5 m^3 represents 22

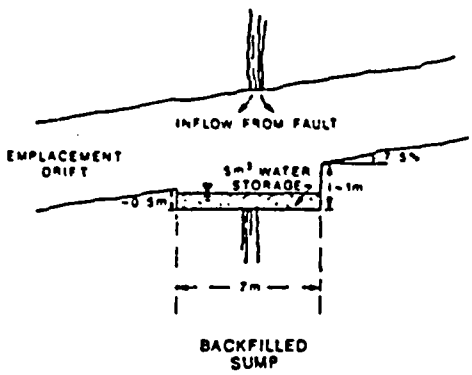
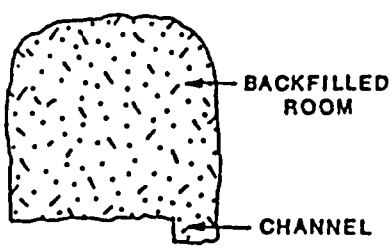
DESIGN OPTIONS	DESIGN GOALS	DESIGN REQUIREMENTS
	• Retain ground-water inflow near source.	• Provide temporary storage capacity ≥5 m^3 and a drainage capacity ≥100 m^3/yr.
	• Channel ground-water inflow away from waste packaging at the following flow rates: <2,000 m^3/year (<1 gal/min) 2,000 - 20,000 m^3/year (1 - 10 gal/min) 20,000 - $2x10^5$ m^3/year (10 - 100 gal/min) $2.0x10^5$ - $2.0x10^6$ m^3/year (100 - 1,000 gal/min)	<u>Emplacement Drifts</u> • Area in cross section ≥0.25 m^2. Hydraulic conductivity of backfill = 1 cm/s (<1 gpm flow) 10 cm/s (1 - 10 gpm) 100 cm/s (10 - 100 gpm) <u>Access Drifts</u> • Area in cross section >2 m^2. Hydraulic conductivities of backfill ≥100 cm/s.

Figure 6-6. Summary of Design Requirements for Design Options in the Underground Facility - Sumps and Channels

gal/min over 1 hour or 0.9 gal/min. over 1 day; 100 m^3/year represents 0.05 gal/min continuously over 1 year). It is noted that the effects of elevated temperature could alter the state of stress in the fracture system below the sump and possibly change the drainage capacity of the welded tuff floor. The size of the sump might be increased to provide additional protection. The extent to which sump capacity is increased is dependent on site specific factors.

6.5.3.2. Channels

Channels can handle higher flows than those accommodated by sumps, depending on the grade of the drift (and channel), cross-sectional area and backfill hydraulic conductivity (Figure 6-7). Assuming a minimum slope of 2.5% and a cross-section of 0.25 m^2, a flow of 2,000 m^3/year (1 gal/min) requires a backfill hydraulic conductivity of 1 cm/s. Flows of up to $4x10^5$ m^3/year (200 gal/min) corresponding to the maximum calculated for individual faults (Section 4.2.2), can be accommodated by two channels, each having a cross-sectional area of 0.25 m^2, with a backfill hydraulic conductivity of 100 cm/s. According to Freeze and Cherry (1979, p. 29), this is the upper limit of conductivity for a gravel. Larger channels may be considered for the access drifts or mains where the width could be increased. Drainage capacities of 1,000 gal/min (collecting water from several faults) could be achieved with a trench 2 m wide by 1 m deep backfilled with 100 cm/s gravel. The drainage capacity of the channels discussed above and given in Figure 6-6 in no way suggests that the large volumes of water are expected on a continuous basis. As indicated in Section 4.2.5 the total amount estimated for unanticipated conditions in a year is 5,600 m^3 or 2.8 gal/min on a continuous inflow basis.

6.5.3.3 Bulkheads or Dams in Emplacement Drifts

Inflows may be retained by bulkheads or dams constructed across the drift downgrade from the source (Figure 6-8). As described below, various types of bulkheads or dams can be considered:

- single dam, and
- single or double bulkhead or dam combination.

In this context, a dam is a structure comparable to an earth dam that need not be in contact with the roof of the drift. A bulkhead is a more rigid structure designed to form a tight contact with the roof.

For present purposes, we assume that the length of drift associated with this design option is 46 m. In this case, the storage capacity against the seal ranges from 17 m^3 for a 2.5% grade to 51 m^3 for a 7.5% grade. The drainage capacity is 470 m^3/yr for each case and the depth of water against the seal varies from 0.75 to 2.25 m. Because the water does not flood up to the full

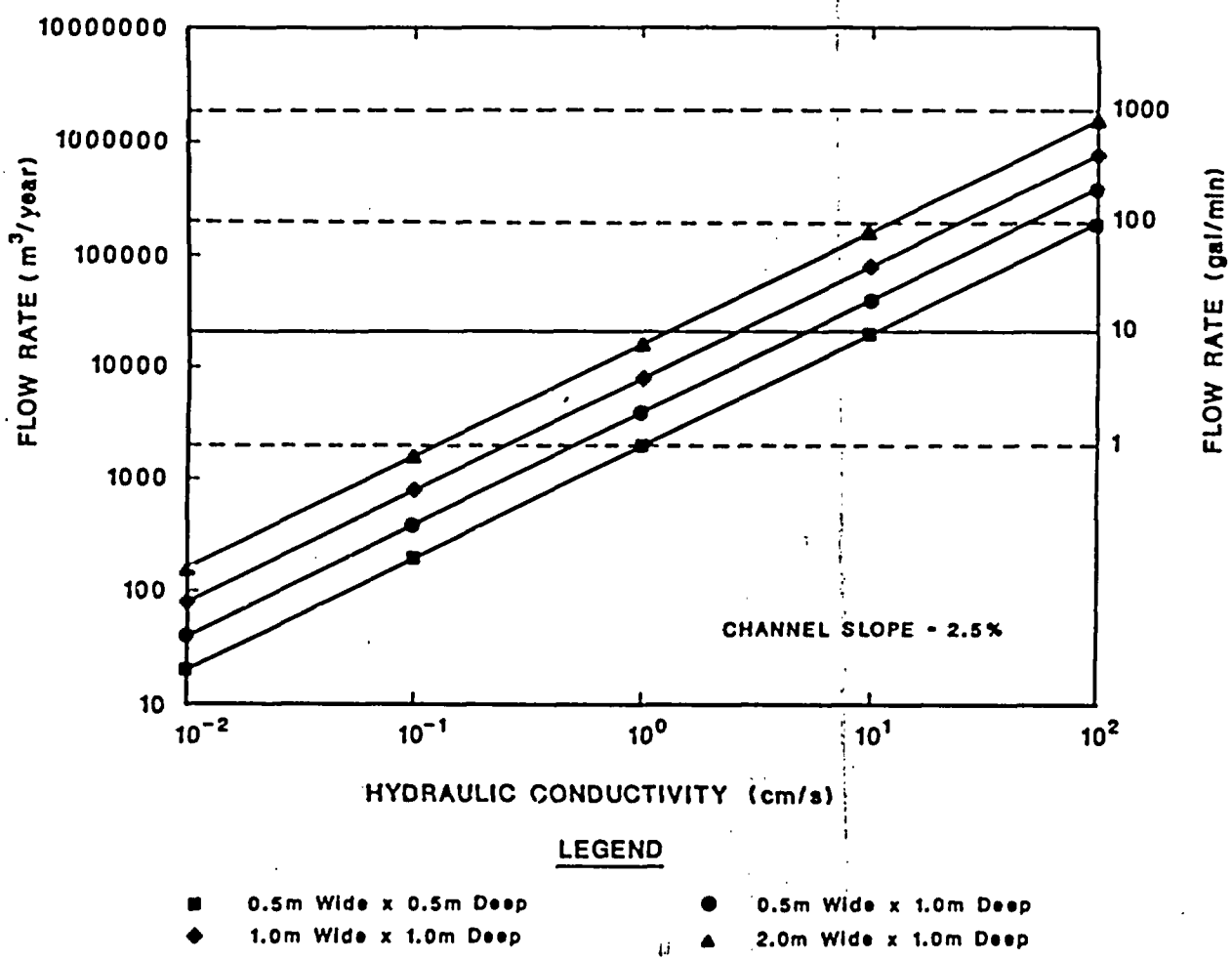


Figure 6-7. Design Chart for Flow Through an Inclined, Backfilled Drainage Channel

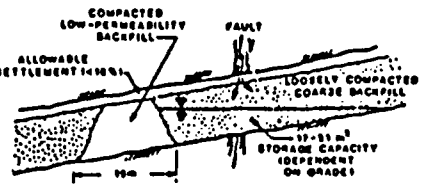
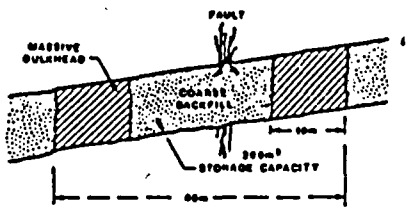
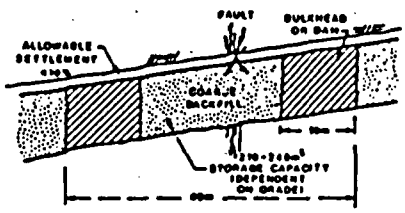
DESIGN OPTIONS	DESIGN GOALS	DESIGN REQUIREMENTS
 (A) SINGLE DAM OR SINGLE BULKHEAD WITH SETTLEMENT	EMPLACEMENT DRIFTS ● Retain a portion of the groundwater inflow near source by providing adequate storage volume and drainage capacity. ● Limit flow through dam to 47 m^3/yr from 0 to 300 yrs and 220 m^3/yr to the end of the sealing period (500 years). PERIMETER DRIFTS OR MAINS ● Promote drainage through drift floor upgradient from dam by limiting leakage through the dam or bulkhead to 10% of the drainage capacity of the drift floor upgradient from the dam, i.e., <200 m^3/yr.	● Provide the following minimum storage volume: 17 m^3 ● Design a drainage capacity of the drift floor upgradient from dam to 470 m^3/yr. ● Construct a dam with an effective hydraulic conductivity of 10^{-5} to 10^{-4} cm/s and <10% settlement. ● Provide a minimum storage volume of: 350 m^3 (perimeter drift) 620 m^3 (tuff and service mains). ● Assure a drainage capacity of ≥2,000 m^3/yr through the drift floor upgradient from the dam. ● Construct a dam with an effective hydraulic conductivity of <10^{-4} cm/s and <10% settlement.
 (B) DOUBLE BULKHEAD (NO SETTLEMENT)	EMPLACEMENT DRIFTS ● Retain inflow between two bulkheads by providing adequate storage volume and drainage capacity. ● Limit flow through bulkhead to 24 m^3/yr/bulkhead from 0 to 300 yrs and 110 m^3/yr/bulkhead to the end of the sealing period (500 years).	● Provide the following storage volume: ≥300 m^3. ● Design a drainage capacity of the drift to be >470 m^3/yr. ● Construct bulkheads with an effective hydraulic conductivity of 10^{-6} to 10^{-7} cm/s and no settlement. ● No storage above upper dam.
 (C) DOUBLE DAM OR DOUBLE BULKHEAD WITH SETTLEMENT	● Retain a portion of the inflow near source by providing adequate storage volume and drainage capacity. ● Limit flow through bulkhead to 24 m^3/yr from 0-300 years and 110 m^3/yr at the end of the sealing period (500 years).	● Provide the following minimum storage volume: 210 m^3. ● Design a drainage capacity of the drift floor between bulkheads to be 470 m^3/yr. ● Construct bulkheads or dams with an effective hydraulic conductivity of 10^{-5} to 10^{-4} cm/s. ● No storage above upper dam.

Figure 6-8. Summary of Design Requirements for Design Options in the Underground Facility Dams and Bulkheads - Vertical Emplacement

height of the room, an effective seal can be achieved by a dam that is not in contact with the roof.

A design requirement for the hydraulic conductivity of a single dam in an emplacement drift may be derived on the basis that the total flow through all such dams should be less than the total allowable inflow from faults that can contact the waste. (In effect, this assumes that any water leaking through a seal contacts the waste.) From Section 6.2, the modified design goal for the total ground-water inflow that can potentially contact the waste is 1,180 m^3/year (0 to 300 years following closure). From Appendix E, the number of intersections of the Ghost Dance Fault with emplacement drifts is about 25. A modified design goal of 47 m^3/yr can be achieved by each dam with a hydraulic conductivity of approximately 10^{-5} cm/s for a period of 0 to 300 years, provided that no flow occurs through the adjacent rock, i.e., provided that the hydraulic conductivity of the adjacent rock is approximately 10^{-5} cm/s or less (Figure F-8, Appendix F). At the end of the sealing period, the design goal is 220 m^3/yr per dam and the design requirement is 10^{-4} cm/s. These calculations assume that the maximum depth of water against the seal is about 6.1 m.

A more complete isolation of a water inflow in an emplacement drift would be achieved by two bulkheads, one on either side of the source. Again assuming that the length of the drift lost to disposal is about 46 m, the storage capacity between the bulkheads would be about 300 m^3 if the length of each bulkhead is about twice the width. If the volume between the bulkheads became filled, the hydrostatic pressure would rise, increasing leakage into the host rock and through the bulkhead. For a total leakage of 1,180 m^3 through 50 bulkheads (i.e., 25 pairs), and assuming a full column of water to the ground surface (300 m), the required hydraulic conductivity would be 10^{-8} cm/s for 0 to 300 years (Figure F-6, Appendix F). At the end of the sealing period the design goal is 110 m^3/yr per bulk head and the design requirement is 10^{-7} cm/s. As with other bulkheads, the host rock conductivity adjacent to the bulkhead must be reduced to the same value as the seal material. Also, as with the single bulkhead described above, the requirement for the downslope bulkhead could be less stringent with the provision of drainage channels to divert leakage away from the waste. This would not apply to the up-slope bulkhead, however, because leakage would build up against the seal and would not be confined to the drainage channels.

For two bulkheads to be completely effective as described above, there should be no settlement or separation from the drift roof. If settlement occurs (or is allowed to occur), the storage capacity will depend on the amount of settlement (i.e., the maximum depth of water against the lower bulkhead) and the drift grade. For a grade of five percent and a settlement not to exceed 10% the storage would be about 210 to 240 m^3 (again with a total 46 m lost disposal). The hydraulic conductivity required to limit flow through 50 bulkheads to 1,180 m^3, or 24 m^3/yr per bulkhead, would be slightly less than 10^{-5} cm/s for 0 to 300 years (Figure F-8, Appendix F). At the end of the sealing period, the design goal is 110 m^3/year per bulkhead and the design requirement

is slightly less than 10^{-4} cm/s. These goals are less stringent than the case for no bulkhead settlement because the maximum head that can develop is less.

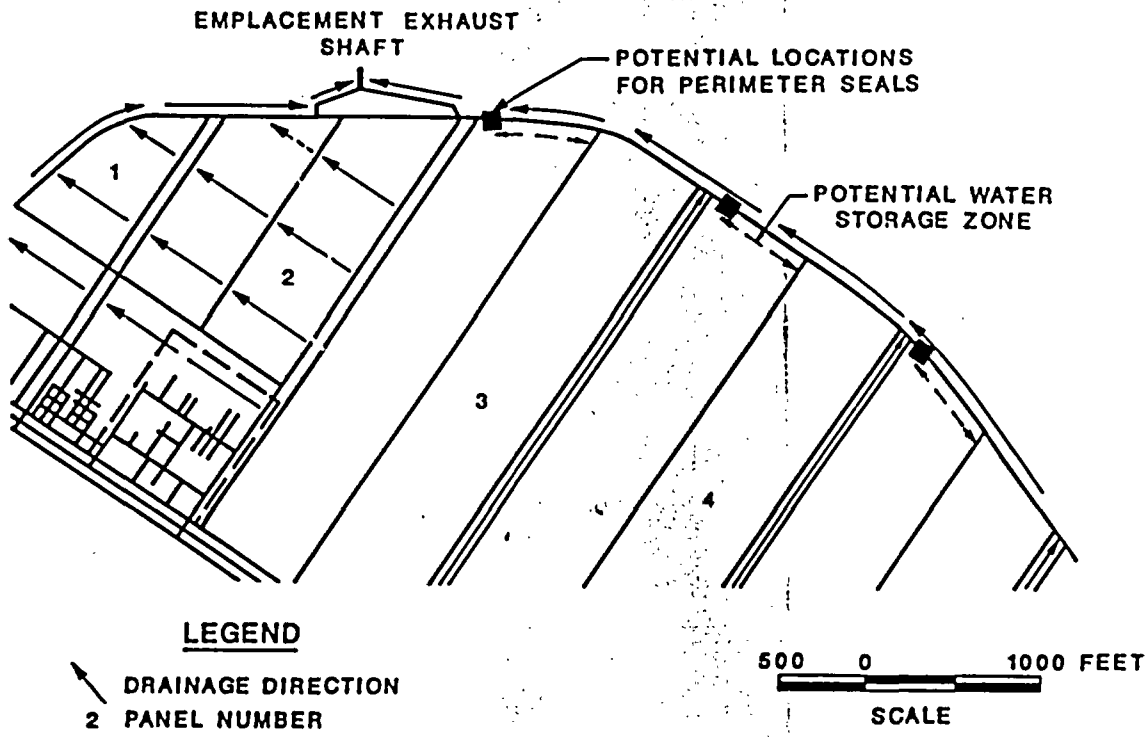
6.5.3.4 Perimeter Drift Seals

The repository floor grades to the north and east to the lowest point at the EES. If flow were to occur through the repository drifts after closure, much of the flow would eventually follow the perimeter drift on the northeast side of the repository. This design option calls for seals in the perimeter drift installed so that the drift will not form a preferred pathway for rapid flow toward the lowest point. These seals could be placed up gradient from each access drift that connects to the perimeter drift (Figure 6-9). The storage behind each bulkhead would then be determined by the length of perimeter drift between access drifts and by whether the water level could rise to or above the top of the seal; i.e., by whether settlement of the bulkhead was allowable. For the condition of no settlement, the storage capacity behind each seal would be equal to or exceed 1,000 m^3, whereas for a case of up to 10% settlement (with no overflow), the storage would range from 350 m^3 for a 10% grade to 700 m^3 for a 2.5% grade. For all cases, the drainage capacity through the floor would be at least 2,000 m^3/year for a floor hydraulic conductivity of 10^{-5} cm/s or greater. The total, cumulative inflow that could be handled by each seal (for each half panel on the eastern side of the repository) would thus be 2,000 m^3/yr.

The perimeter seals need not be completely impermeable since their function would be to retard flow, not prevent it. A design requirement for hydraulic conductivity is established by assuming that the flow through the seal in 1 year is selected to be less than 10% of the drainage capacity behind the seal (i.e., so that the bulk of the water drains through the floor). This leads to a requirement for hydraulic conductivity of the seal to be about 10^{-4} cm/s (Figure F-9, Appendix F) for a period of 500 years.

6.5.3.5 Seals in the Mains

Seals in the mains would have a similar function to those in the perimeter drift. Whereas the seal in the perimeter drift would collect and store water entering in the eastern half of the repository, the seals in the mains would collect and store water entering in the western half. As with the perimeter drift, the storage capacity is determined by the length of the mains between seals placed at cross-cuts (Figure 6-9), by the grade, and by whether water can rise to or above the top of the seal. Considering two of the mains (tuff main and service main), the storage capacity is 1,900 m^3 (2.5% grade) for the case of no settlement and 1,900 m^3 (2.5% grade) to 620 m^3 (8% grade) for up to 10% settlement. (The volume in the waste main is not considered because this main is not connected to the panels on the west side by cross-cuts). The drainage capacity, i.e., the cumulative yearly inflow that can be accommodated, is 9,000 m^3/year (2.5% grade) to 2,800 m^3/year (8% grade) for 10% settlement. Using the same design goal for leakage through the seals as for



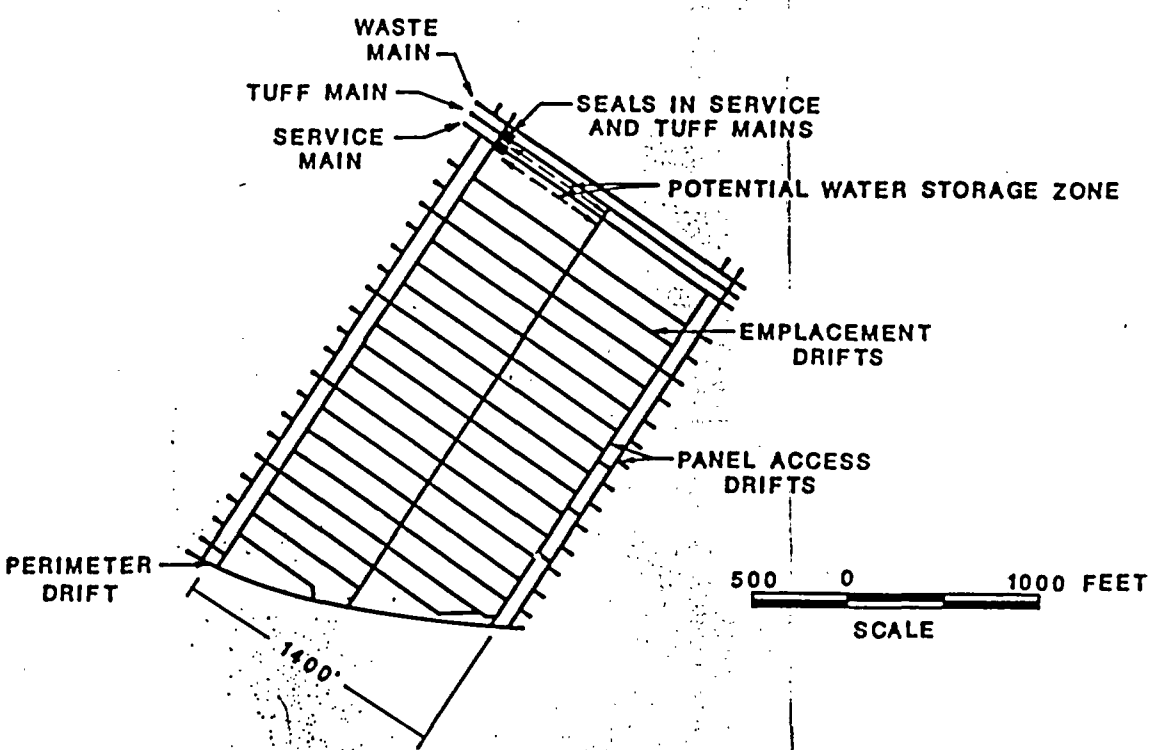


Figure 6-9. Proposed Locations for Seals in the Underground Facility - Vertical Emplacement

the perimeter drift (i.e., 10% of the floor drainage per year), the required hydraulic conductivity is 10^{-4} cm/s with or without settlement.

6.5.4 Preferred Options for Seals in the Underground Facility - Vertical Emplacement

As noted at the beginning of Section 6.5.3, the selection of seal components to be used in the underground facility depends on the hydrologic conditions encountered. If no significant water-bearing zones* are encountered during construction, and if there are no fractured zones that are expected to conduct water, it will not be necessary to place any seals. If there are zones that are expected to conduct significant amounts of water, the available options range from sumps and channels to bulkheads or dams.

Sumps and drainage channels offer a simple means of channeling water away from the waste. A major advantage of sumps and channels is that they do not require the use of materials that could be degraded by ground-water interaction at the high temperatures that will exist in the repository after closure. For sumps and drainage channels, the main process of concern is the reduction of storage volume or drainage capacity by silting. Fines could be derived from the inflow (i.e., silt or clay washed out of a fault) or from the backfill placed in the sump or channel. Silting from fault inflow could be prevented by use of a double sump, the first of which, located below the inflow, would catch the fines. Silting of the backfill can probably be prevented by appropriate gradation.

Bulkheads are routinely used in mining to control water flow, but in situations where leakage can often be tolerated and where the design life is short. The most stringent case evaluated above was for two bulkheads intended to isolate a fault zone and limit flow to 24 m^3/yr through each bulkhead. This would require an effective hydraulic conductivity range of from 10^{-8} to 10^{-7} cm/s for the seal and the MPZ. This is considered to be a very stringent design requirement for a bulkhead built in a rock mass with a hydraulic conductivity of 10^{-2} to 10^{-5} cm/s.

In contrast, the requirements of 10^{-5} cm/s for an embankment-type dam placed in an emplacement drift or 10^{-4} cm/s for a dam in the mains or perimeter drift are considered to be attainable. In utilizing emplacement dams, it will be important to assure that the storage capacity behind each dam is sufficient to prevent the water height from exceeding the dam height during the time period of 500 years. It is emphasized that silting may be an important issue that could require provision for a large storage volume.

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*Significant amounts of water are defined as volumes of water entering the underground facility in excess of the performance allocation established for the underground facility.

For all bulkheads or dams, the main concerns relate to structural stability and the need to reduce flow through the interface and damaged zones. Seals located in the emplacement rooms would be subject to temperatures exceeding 100°C. Seismic loading may be a concern if it is necessary to maintain a tight interface between the seal and the host rock or if it is necessary to prevent cracking. To reduce flow through the damaged zone, it may be necessary to cut out a keyway or to grout the fractures in the host rock adjacent to the seal. Concerns regarding structural cracking within the seal or at the interface may be alleviated by the use of cementitious or earthen materials or a combination of both. Consideration would then be given to limiting settlement and cracking due to desiccation.

Grouting of fault zones is not favored as a means for long-term water control because of concerns with the longevity of the materials employed and because it may be difficult to demonstrate the effectiveness of the technique.

Given the concerns regarding the practicability of constructing low permeability seals in fractured rock, the preferred option for sealing in the underground facility is a combination of sumps and earthen dams that may, in certain, selected, nonemplacement areas, be supplemented with channels. Sumps can be used alone to accommodate inflows of up to 5 m^3 [illegible] a single event or 100 m^3/year. Earthen dams could effectively promote vertical drainage of up to 2,000 m^3/year in selected areas. Channels within nonemplacement drifts can accommodate large flows as high as 2,000,000 m^3/year (1,000 gal/min), provided they are backfilled with coarse crushed rock and steps are taken to prevent silting. If silting were to occur reducing channel capacity, the reduced flow capacity would likely still be large, and exceed flow rates for unanticipated conditions. Because the underground facility is located in the unsaturated zone and the anticipated inflows are estimated to be 62 m^3/year (Table 4-6), the full drainage capacities of the channels and dams given above will not be reached.

Design requirements for the optional seal components are summarized in Figures 6-6 and 6-8, and the required design analyses are listed in Table 6-6.

6.6 DESIGN REQUIREMENTS FOR SEALS IN THE UNDERGROUND FACILITY - HORIZONTAL EMPLACEMENT

6.6.1 Functional Requirements

The function for seals located in the underground facility (horizontal emplacement) is to reduce the amount of water that can contact the waste. Under anticipated conditions, little or no water is expected to enter the emplacement boreholes from matrix flow. Because this water will enter the emplacement boreholes directly, it is probably not practical to attempt to reduce the volume. For extreme conditions, seals may be required to isolate inflows from faults.

Table 6-6. Design Analyses and Properties Required for Underground Facility Seals - Vertical Emplacement

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[a]
Fault Sump		
• Storage and Drainage Capacity Calculations	Saturated, Bulk Rock, Hydraulic Conductivity of the Topopah Spring	Grade of Emplacement Drifts Drift Dimensions Elevated Temperatures Due to Radioactive Waste Generation
• Silting Potential	Condition of Fault Flow Zones Gradation of Fill	
Drainage Channels		
• Flow Capacity Calculation	Saturated Hydraulic Conductivity of Backfill	Grade of Emplacement Drifts Drift Dimensions
• Silting Potential	Condition of Fault Flow Zones Gradation of Fill	
Drift Bulkhead		
• Flow Through Plug and MPZ	Saturated, Bulk Rock, Hydraulic Conductivity of the Topopah Spring MPZ Model for the Topopah Spring	

(a) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

Table 6-6. Design Analyses and Properties Required for Underground Facility Seals - Vertical Emplacement (Concluded)

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[a]
• Structural Stability of the Bulkhead		Drift Dimensions Depth of Repository
- Seismic	Horizontal and Vertical Acceleration	
- Thermal	Repository Temperature Temperature Rise Due to Curing	Temperature Rise Due to Radioactive Heat Generation
- Gravitational/ Hydrostatic	Depth of Water in the Fault System In Situ Stress in the Topopah Spring	
• Settlement of the Bulkhead	Coefficient of Compressibility of the Backfill	
• Chemical Durability	Geochemistry of the Topopah Spring	

(a) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

6.6.2 Design Options

The sealing approach for horizontal emplacement is different than for vertical emplacement where emphasis is placed on diverting water flow away from the waste. In the case of horizontal emplacement, if an emplacement borehole intersects a fault inflow, diversion of water is not possible. The options that exist for horizontal emplacement holes (Figure 6-10) are

- place a seal in the borehole between the source and the waste,
- grout the permeable zone in the borehole, and
- abandon the hole for disposal (not shown in figure).

If a hole is abandoned, any water inflow will flow freely into the access drifts, which will act as drains (analogous to the drainage channels proposed for vertical emplacement) because the emplacement boreholes are drilled at the mid-height of the walls of the rooms. Any water that enters a drift will not be able to enter another emplacement borehole until extreme flooding occurs in the low part of the repository.

6.6.3 Design Goals and Design Requirements for Seals in the Underground Facility

The total design goal for the underground facility is 1,180 m^3/year from 0 to 300 years. At the end of the sealing period (500 years), the design goal is 5,600 m^3/yr. It is recognized that uncertainty exists in the amount of flow that can enter the horizontal emplacement boreholes under unanticipated conditions. An underestimation of this amount of water might suggest the selection of a design goal to reduce the amount of water entering the waste disposal area. Reduction of water could be achieved by emplacing fault seals (plugs) within the emplacement boreholes or by abandonment of the emplacement hole. For the purposes of calculating a design requirement for a plug, we assume that the total leakage through 100 fault seals* is to be limited to 1,180 m^3/year. A design goal of 12 m^3/year per seal can be achieved by a seal with a hydraulic conductivity of 10^{-7} cm/s provided no flow occurs through the adjacent rock (Figure F-7, Appendix F). At the end of the sealing period, the design goal is 56 m^3/yr and the design requirement is slightly less than 10^{-6} cm/s. In practice, it will be necessary to grout the host rock adjacent to the plugs so as to reduce the potential leakage.

In the remainder of the underground facility, design concepts similar to those proposed for the vertical emplacement configuration can be implemented.

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*As described in Appendix E, the Ghost Dance Fault will intersect about 97 emplacement boreholes.

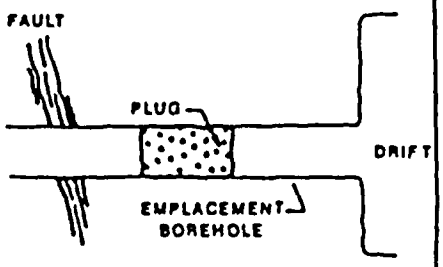
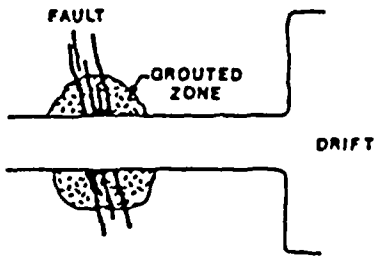
DESIGN OPTIONS	DESIGN GOALS	DESIGN REQUIREMENTS
 PLUG	• Limit ground-water inflow past plugs to 12 m^3/yr from 0 to 300 yrs and 56 m^3/yr at the end of the sealing period (500 years).	• Emplace a borehole plug having an effective hydraulic conductivity (including interface zone and MPZ) 10^{-7} to 10^{-6} cm/s.
 GROUTING	• Reduce inflow to emplacement borehole.	• Reduce hydraulic conductivity of fault zone to 10^{-5} cm/s (equivalent to adjacent host rock).

Figure 6-10. Summary of Design Requirements for Design Options in the Underground Facility - Horizontal Emplacement

Within the emplacement drifts, dams can be placed or sumps excavated to isolate small quantities of water. To prevent potential water entry into the horizontal emplacement holes, however, the height of water behind dams should be controlled so that water does not flow into the horizontal emplacement holes. The option to direct water to the access and perimeter drifts and the tuff and service mains is possible. The selection of dams to isolate water inflows at their source or the selection to direct waters to other locations in the repository (e.g., perimeter drifts or mains) depends on the volume of water and number of occurrences of water. For example, diversion of waters to the perimeter drift and mains may not be necessary if the flow into the underground facility is low and infrequent. If waters are diverted away from their source, excavation of the emplacement drift floor to create channels would not be necessary.

Figure 6-11 illustrates the potential locations for perimeter seals and seals in the service and tuff mains. Because no mid-panel access drifts exist for horizontal emplacement, the potential, water storage zone and drainage capacity associated with these seals is greater than zones associated with similar seals proposed for vertical emplacement (Figure 6-9). This is particularly true if the seals do not settle. The storage and drainage capacities are approximately twice the values calculated for vertical emplacement if settlement is not allowed to occur.

6.6.4 Preferred Options for Seals in the Underground Facility - Horizontal Emplacement

Similar concerns exist for seals placed in horizontal emplacement holes as for bulkheads placed in drifts, in that the hydraulic conductivity required for a tight seal (10^{-7} cm/s) is very stringent for sealing in fractured rock. These concerns may be more serious for horizontal emplacement holes given that the seals must be placed remotely and that the seals might be subjected to temperatures exceeding 200°C. The same concerns apply to grouting within the fracture zone as to placing a seal in the borehole.

Given the concerns expressed above, the preferred option for any borehole in which water is encountered would be to abandon the hole. Consider the extreme case of abandoning (or not drilling) any hole intersecting the Ghost Dance Fault. A total of 97 holes or 3.6% of all the holes would be lost from the repository. More realistically (but still an extreme case), considering only those boreholes that intersect the Ghost Dance Fault below washes (Section 4.2.3), the number of holes lost would be about 25, or 0.9% of the total.

Design requirements for seals in horizontal emplacement holes are summarized in Figure 6-10, and the required design analyses are listed in Table 6-7.

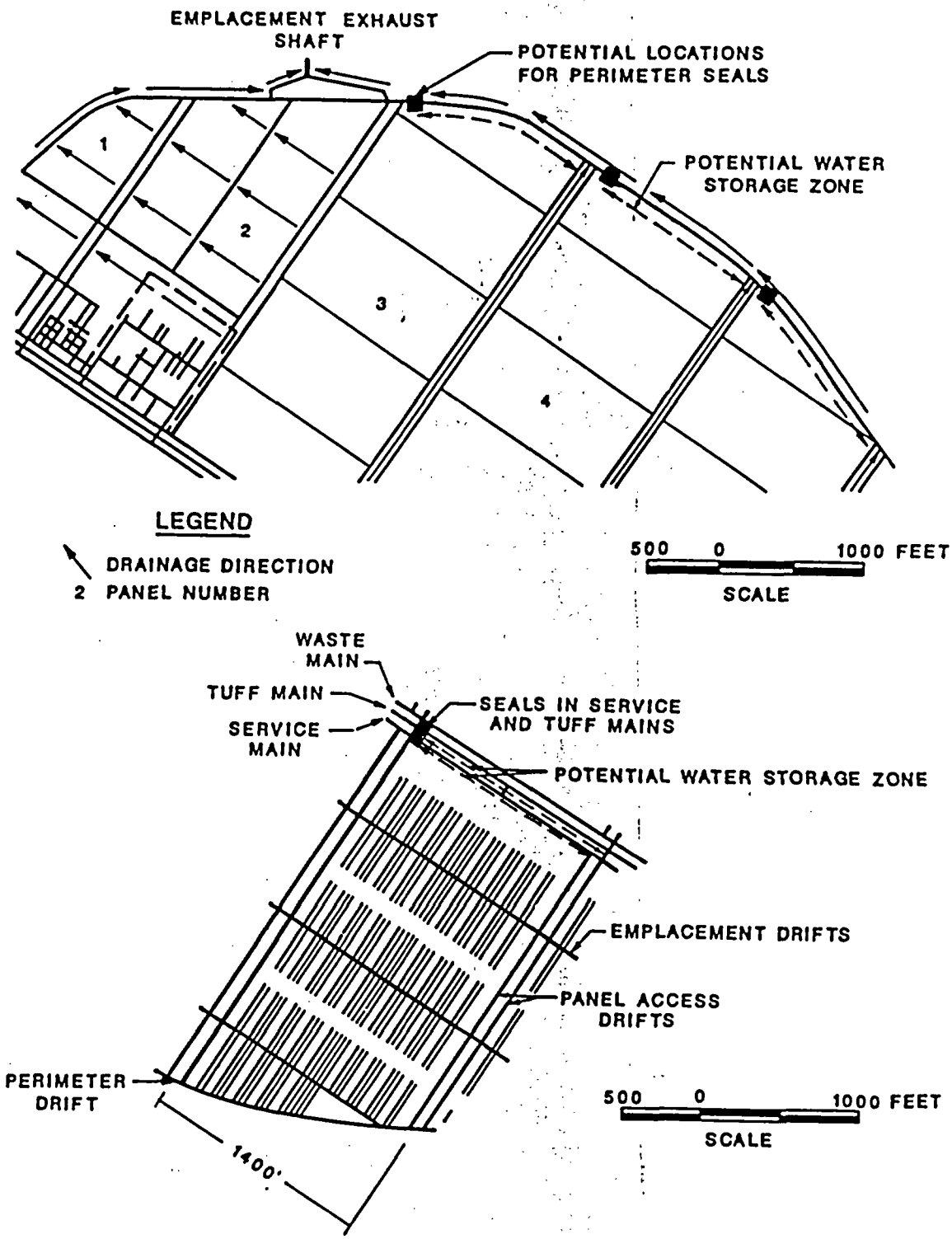


Figure 6-11. Proposed Locations for Seals in the Underground Facility - Horizontal Emplacement

Table 6-7. Design Analyses and Properties Required for Underground Facility Seals - Horizontal Emplacement

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[a]
Borehole Seal		
• Flow Through Plug and Damaged Zone	Saturated, Bulk Rock, Hydraulic Conductivity of the Topopah Spring	Borehole Diameter
	MPZ Model for the Topopah Spring	
• Structural Stability of Plug		Borehole Dimensions
		Depth of Repository
- Seismic	Horizontal and Vertical Acceleration	
- Thermal	Repository Temperature	Temperature Rise Due to Radioactive Heat Generation
	Temperature Rise Due to Curing	
- Gravitational/ Hydrostatic	Depth of Water in the Fault System	
	In Situ Stress in the Topopah Spring	
• Settlement of the Bulkhead	Coefficient of Compressibility of the Backfill	
• Chemical Durability	Geochemistry of the Topopah Spring	

(a) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

6.7 SEAL DESIGN REQUIREMENTS FOR EXPLORATORY HOLES

6.7.1 Functional Requirements

Exploratory boreholes drilled from the surface could potentially provide pathways for radionuclide migration from the repository to the water table. The primary function of seals for these holes is to reduce the potential for water-transported radionuclides to be preferentially transported through boreholes. The physical process of concern would be preferential, ground-water flow through the repository, Calico Hills Unit, and into the ground-water table (see Section 3.3). A secondary function for borehole seals may be to mark the location of the boreholes at the surface, if this is required by State regulations.

As of May 1986, there are four existing boreholes within the proposed repository perimeter that penetrate to the water table (Figure 6-12). These boreholes provide a simple vertical pathway from the repository to the water table. About 25 additional boreholes penetrate to the water table within 5 km of the repository perimeter. Among these holes, a potential pathway exists for those holes, which are downdip from the repository. Water that drains through the repository floor could become perched at a stratigraphic contact, flow downdip, and enter a borehole that penetrates through the contact to the water table.

Figure 6-13 shows structural contours on the various stratigraphic contacts that occur between the repository horizon and the water table.* All of the units shown in Figure 6-13 occur above the water table on the western side of the repository, whereas only the Topopah Spring and part of the Calico Hills units occur between the repository and the water table on the eastern side. The strikes on each of the contacts are generally north-south or northwest-southeast, so that if any of the contacts act as barriers to downward infiltration, flow would occur downdip to the east or northeast. Generally, boreholes that occur to the west of the repository, as well as probably those to the south and north, cannot act as pathways for radionuclide migration.

6.7.2 Design Options

Design options for borehole seals were presented by Fernandez and Freshley (1984). The major component is a seal placed in the Calico Hills section. This seal may extend up into the Topopah Spring, or this section of the hole could be backfilled with sand or gravel to be free draining. A surface cover is required to mark the location of the hole and deter reentry. Abandonment

*Structural contours are related to the reference stratigraphy given by Ortiz et al. (1985) which is more detailed than the hydrogeologic classification in Table 2-1. The reference stratigraphy by Ortiz et al. is based on thermal, mechanical and physical properties.

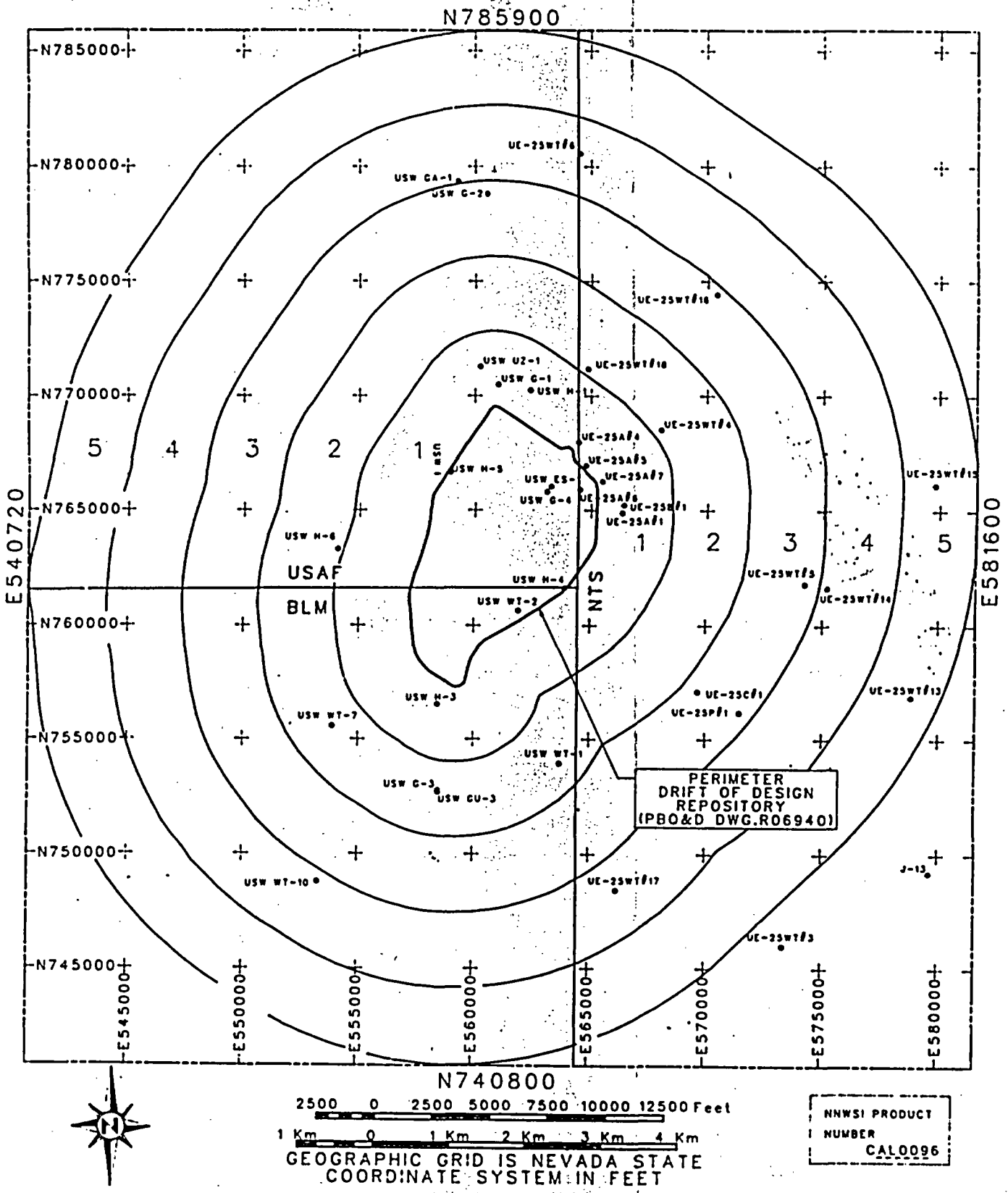


Figure 6-12. Repository Perimeter Drift, Borehole Locations, and 1-, 2-, 3-, 4-, and 5-km Distance Contours From Edge of Repository Perimeter Drift

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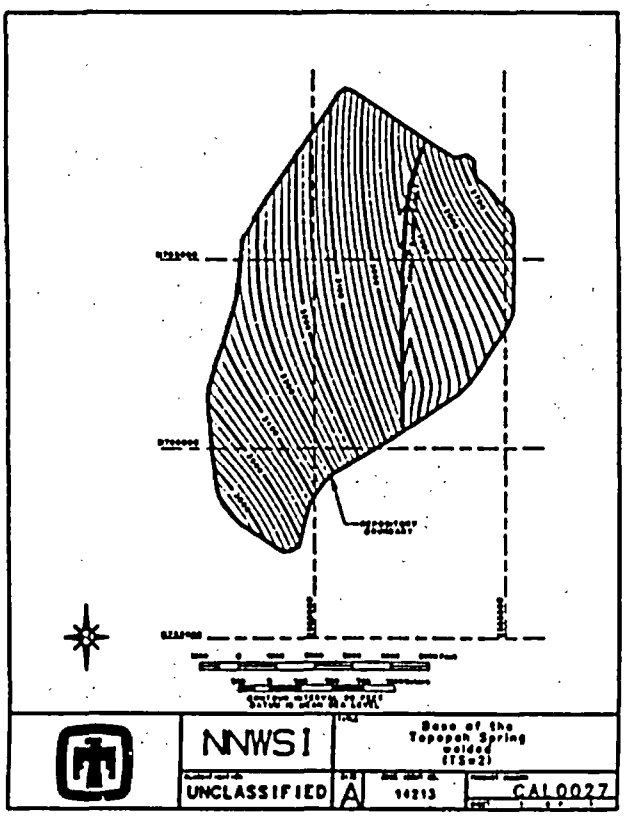
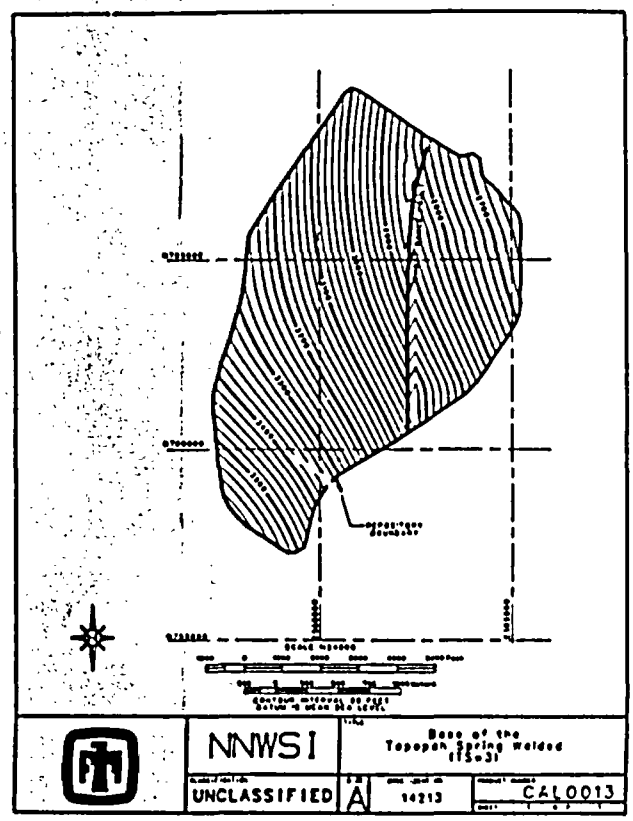
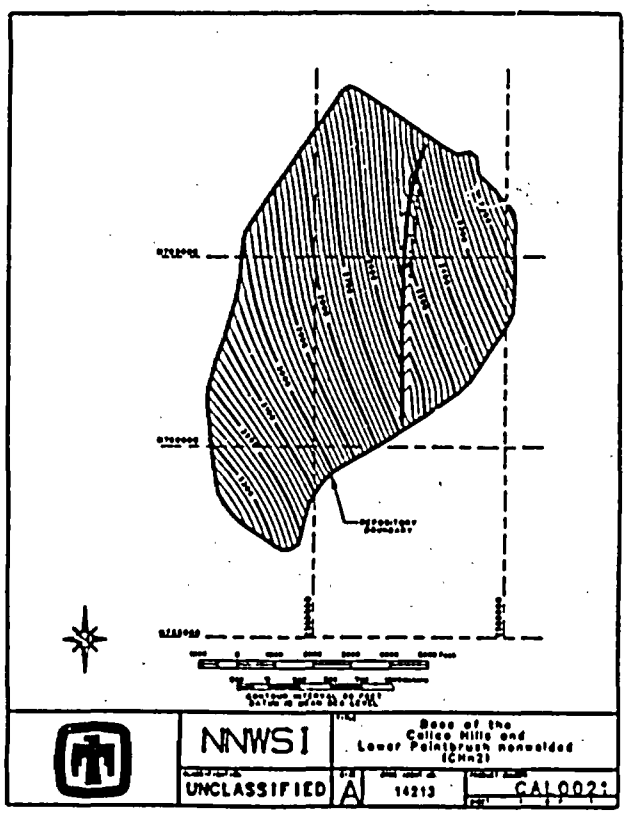






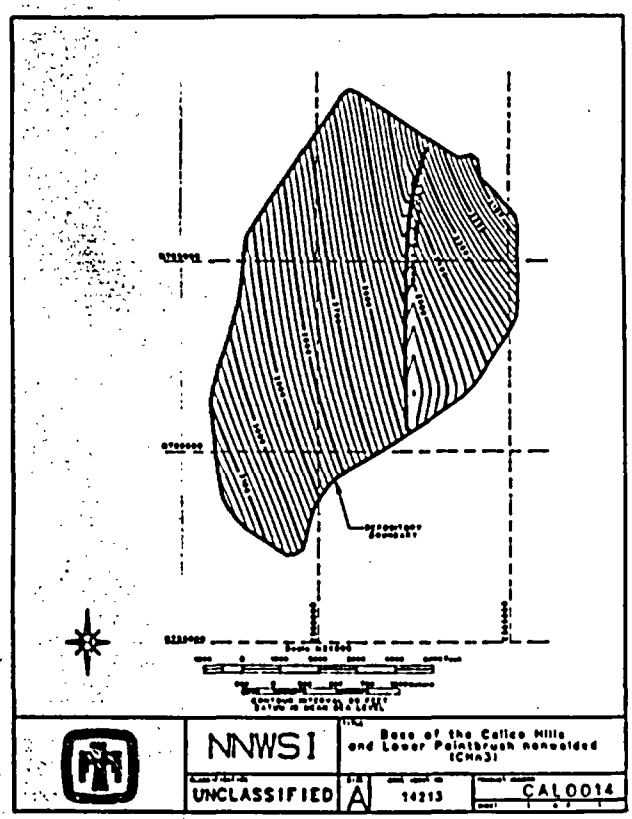
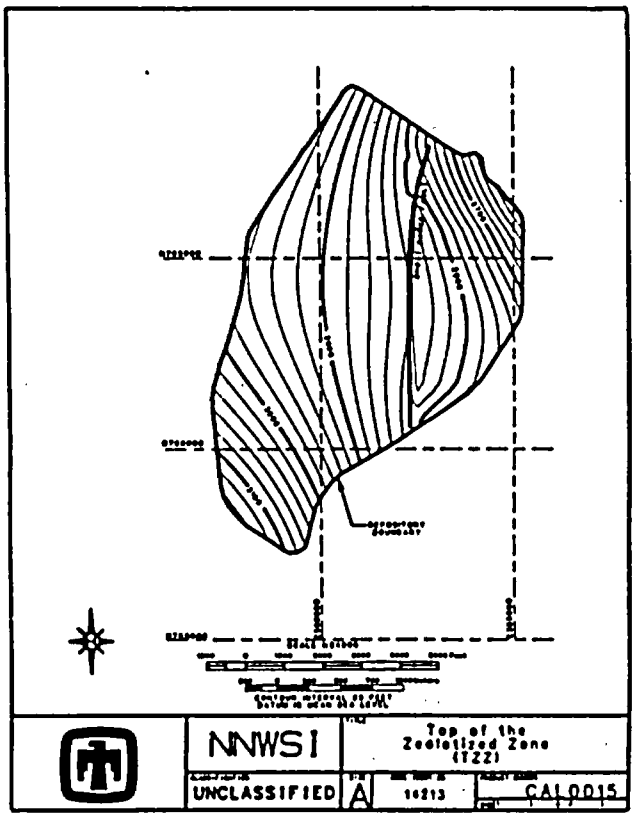
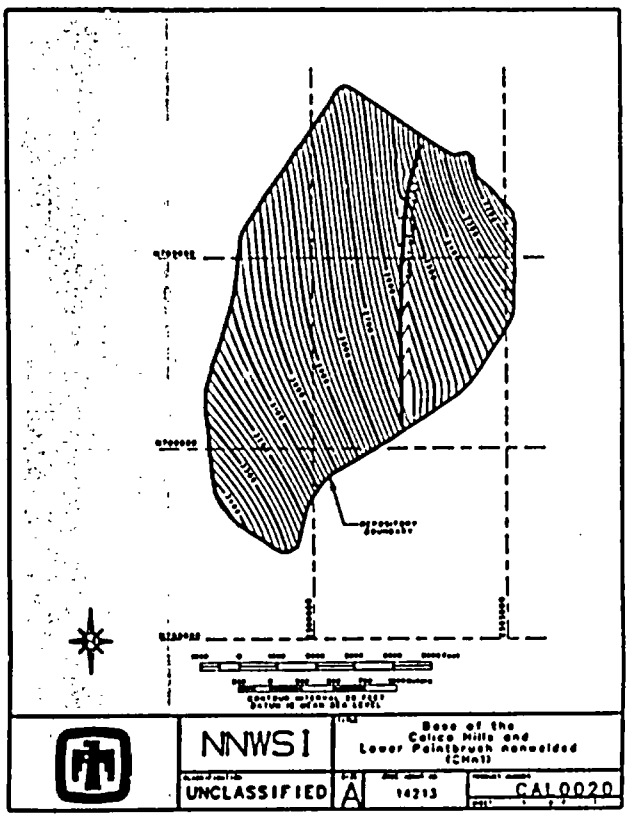
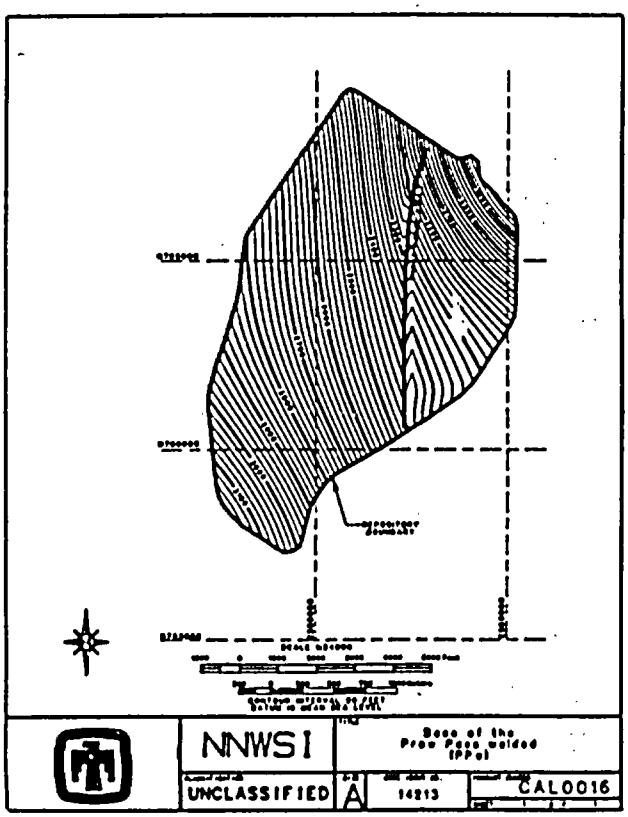
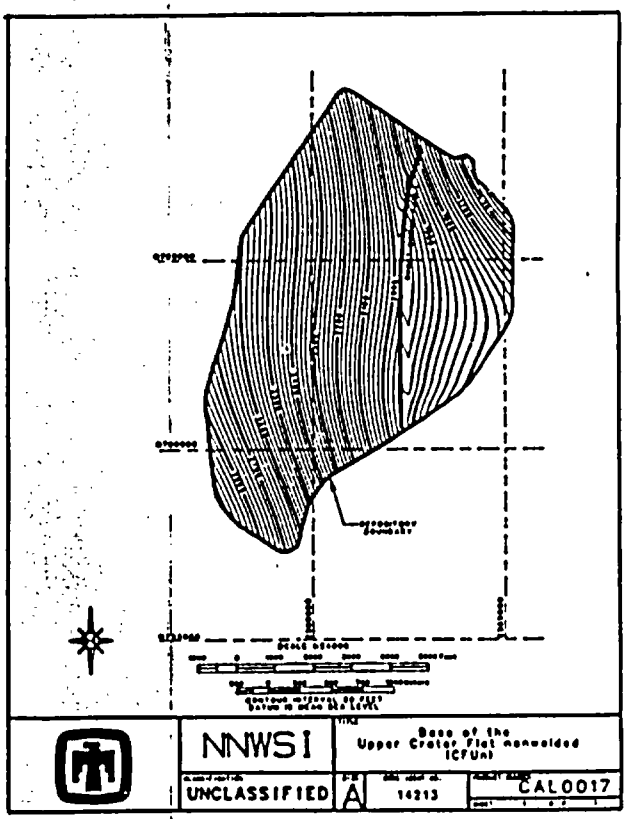


Figure 6-13. Contour Maps of Stratigraphic Cont









c Contacts of Geologic Units Below the Repository

requirements developed by the Nevada Department of Conservation and Natural Resources for oil, gas, and water wells may also apply.

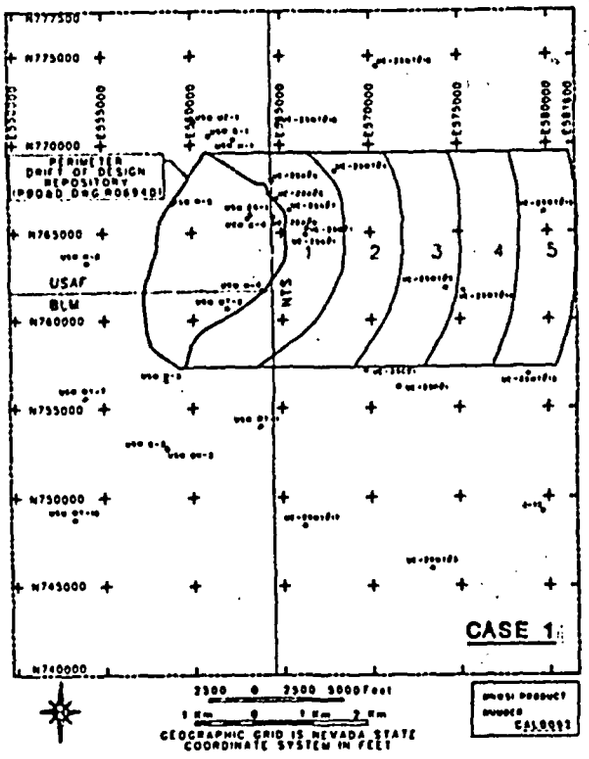
6.7.3 Performance Goals and Design Requirements for Borehole Seals

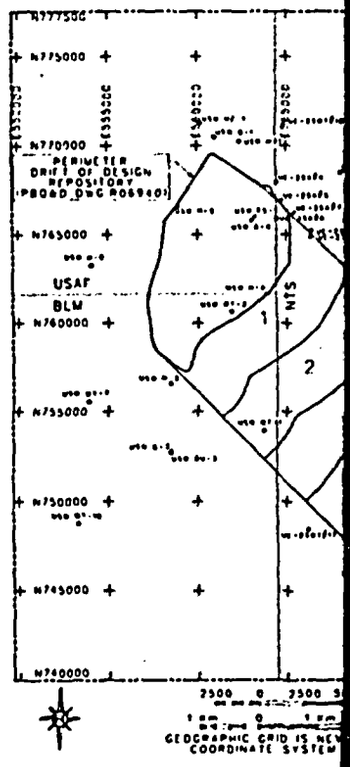
A performance requirement for borehole seals is developed from the NRC 10 CFR 60 requirement (Section 1.2) that "boreholes shall be designed so that following closure they do not become pathways that compromise the geologic repository's ability to meet the performance objectives." We adopt the position that this requirement is satisfied if the potential for vertical flow through boreholes is only one percent of the potential for vertical flow through the host rock mass. For both boreholes and the rock mass, the effective hydraulic gradient is unity for the condition of vertical infiltration under atmospheric pressure. Considering the area within the repository perimeter, the potential vertical flux through the rock mass is the effective hydraulic conductivity of the rock units between the repository and the water table times the floor area of the repository drifts or, more conservatively, the total area within the repository. The potential flow through boreholes is the total, cross-sectional area of the boreholes within the repository area times the effective hydraulic conductivity of the seal material (taking into account the effect of the seal host rock interface). The required hydraulic conductivity is obtained by

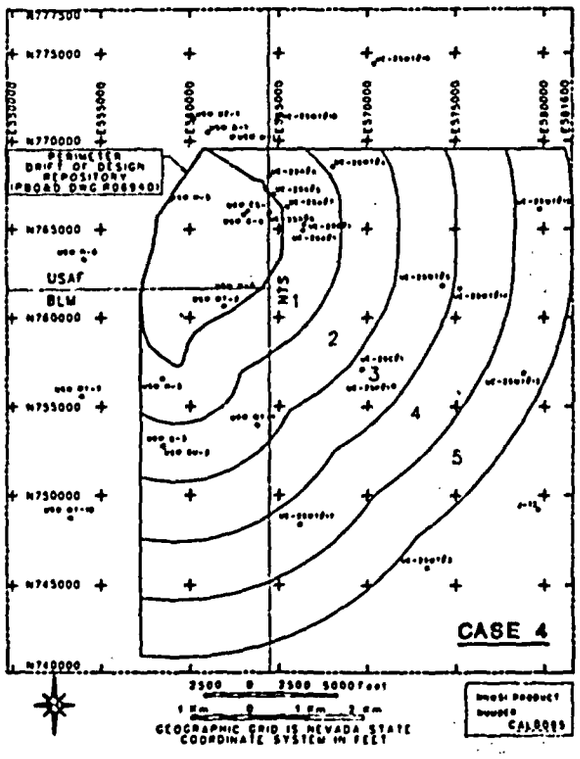
$$\text{hydraulic conductivity of seal material} = 0.01 \times \frac{\text{area within repository perimeter} \times \text{rock mass, hydraulic conductivity}}{\text{total area of boreholes}}$$

A range of rock mass, hydraulic conductivities may be considered; the lower the value selected, the lower will be the required hydraulic conductivity for the seal material.

Performance goals for borehole seal materials have been determined for various cases representing different areas to the northeast, east and southeast of the repository, as well as the repository area (Figure 6-14). A range of rock mass, hydraulic conductivities is also used, from 10^{-8} to 10^{-4} cm/s representing approximately the range for rock mass and matrix conductivity of the Calico Hills (Table 2-2). Applying the formula given above, the required hydraulic conductivity for the borehole seal material varies from about 10^{-3} to 10^{2} cm/s (Figure 6-15), the lowest value being obtained for the case of the repository area and the 10^{-8} cm/s rock mass, hydraulic conductivity. Because boreholes are distributed fairly uniformly around the repository, the required hydraulic conductivity is relatively insensitive to the area included in the calculation. Additional boreholes drilled from the exploratory shaft test facility during site characterization are expected to have similar hydraulic conductivity requirements.







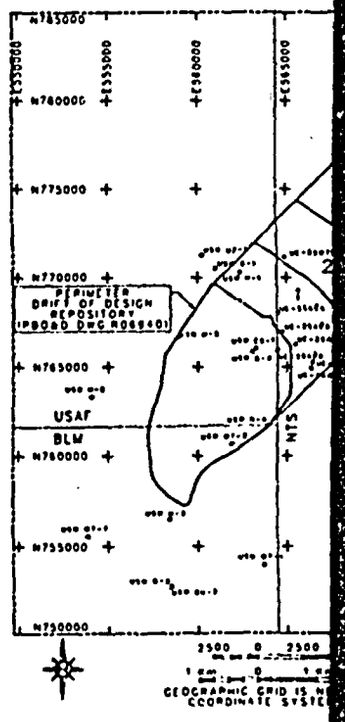
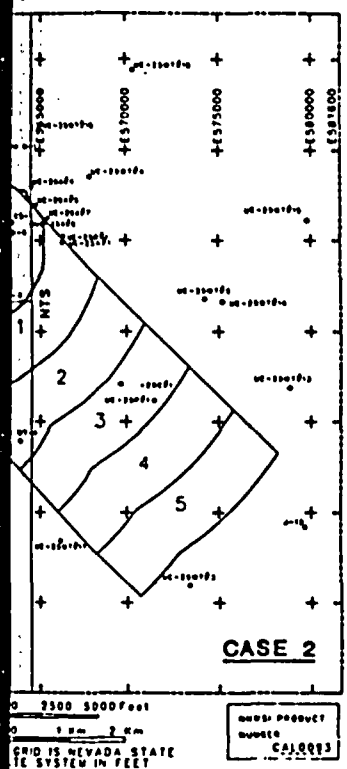
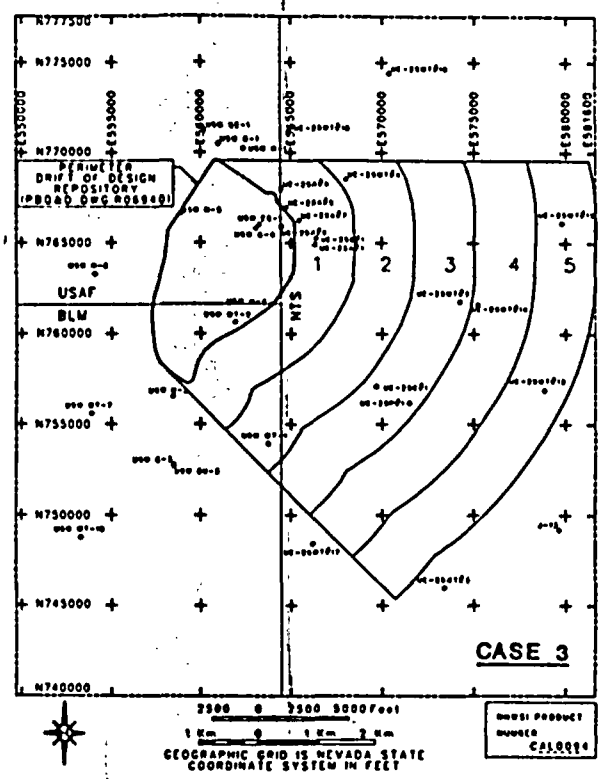
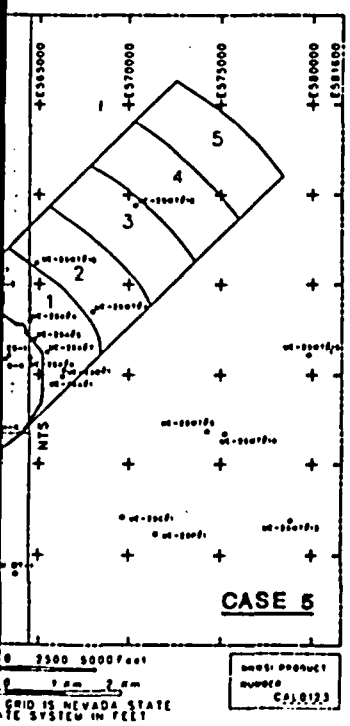
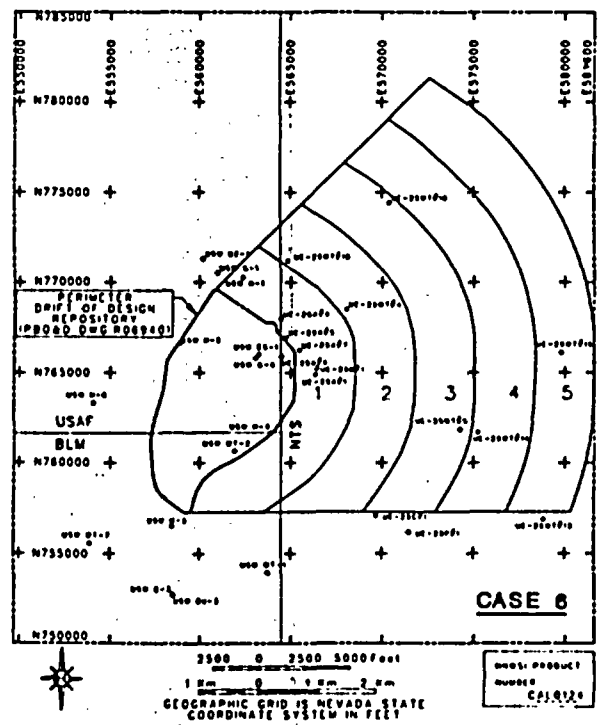


Figure 6-14. Schematic Presentation of Cases Analyzed to

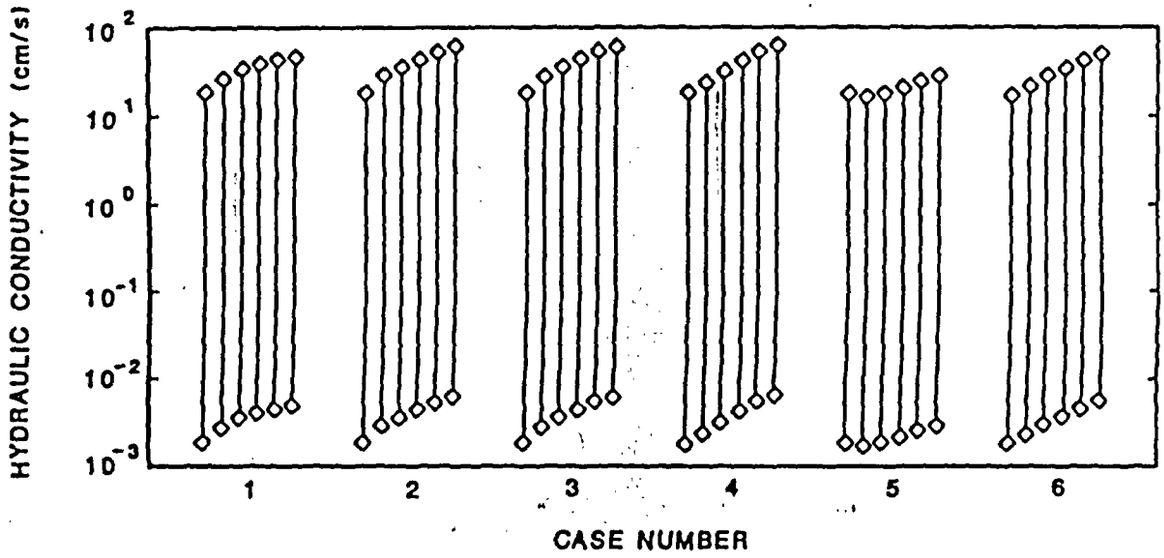








d to Develop Hydrologic Performance Goals for Borehole Seals



LEGEND

BULK ROCK HYDRAULIC CONDUCTIVITY
10^{-4} cm/s (UPPER POINTS) 10^{-8} cm/s (LOWER POINTS)

FIRST RANGE FOR CASE - REPOSITORY AREA ONLY
SECOND RANGE FOR CASE - AREA 1 KM OUT
THIRD RANGE FOR CASE - AREA 2 KM OUT
FOURTH RANGE FOR CASE - AREA 3 KM OUT
FIFTH RANGE FOR CASE - AREA 4 KM OUT
SIXTH RANGE FOR CASE - AREA 5 KM OUT

Figure 6-15. Performance Goals for Borehole Seals Considering Varying Bulk Rock Hydraulic Conductivity Values and Assuming Several Conditions of Lateral Flow Along Stratigraphic Contacts

6.7.4 Preferred Options for Borehole Seals

For current planning purposes, the approach adopted involves placing a moderately low permeability material in the Calico Hills section. It is noted, however, that the application of the performance goals for borehole seals depends on the location of the borehole relative to the repository. Therefore, only a portion of the boreholes are intended to achieve the performance goals. These boreholes are identified in Table 6-8. Most of the boreholes identified penetrate through the underground facility and into the ground-water table. Shallow boreholes that penetrate into the Topopah Spring Member or overlying members are not included in Table 6-8 because they do not represent pathways to the accessible environment. In Table 6-8, Category A boreholes are those that represent potential pathways to the accessible environment. The further the borehole is located from the repository boundary, the lower its potential to act as a pathway to the accessible environment. Those boreholes located generally to the northeast, southeast, and east of the underground facility fall into Category A. Those boreholes located to the west, 5 km or more from the edge of the repository boundary, and to the far north and far south are not included here. Category B boreholes are those boreholes that do not represent potential pathways to the accessible environment but because of their proximity to the repository boundary will be sealed. Those boreholes immediately to the south and north of the underground facility are included in Category B.

There appear to be fewer concerns related to sealing boreholes drilled from the surface than for sealing shafts, ramps, drifts, or emplacement boreholes. Borehole features advantageous to sealing include the low volume of seal materials, isolated locations away from a high temperature field, a less significant MPZ than for shaft or drift seals (Kelsall et al., 1982), and the fact that the seal can be placed over the entire length of the holes between the water table and the repository. In two other respects, borehole sealing may be more difficult because remote methods are required for emplacement and because direct inspection of the completed seal is not possible. Accordingly, the most significant consideration for borehole seals is the development of adequate emplacement methods. Design analyses required for borehole seals are listed in Table 6-9.

Two additional requirements imposed in the design of the underground facility are that (1) the underground facility does not intercept any borehole and (2) a minimum distance of 15 m occurs between a drift excavation and the borehole. This minimum distance of 15 m represents a point where elastic deformation occurs and the deviatoric stresses are small (Case and Kelsall, 1987, p. 27).

Further, interconnecting a borehole with the underground facility could provide a preferred pathway from the underground facility to the water table. For those boreholes within the repository outline, sealing of boreholes should be performed from the repository horizon to the top of the Calico Hills in addition to sealing within the Calico Hills unit. The proposed design

Table 6-8. Boreholes Proposed for Sealing to a Specified Performance Goal

Category A

- Within repository limits
  - USW H-5
  - USW G-4
  - UE-25 a#6
  - USW H-4
  - USW WT-2
- 1 km from edge of repository
  - UE-25a#4
  - UE-25a#5
  - UE-25a#7
  - UE-25b#1
  - UE-25a#1
- 2 km from edge of repository
  - UE-25WT#4
  - USW WT-1
- 3 km from edge of repository
  - UE-25WT#5
  - UE-25c#1
  - UE-25p#1
- 4 km from edge of repository
  - UE-25WT#14
- 5 km from edge of repository
  - UE-25WT#15
  - UE-25WT#13

Category B

- USW H-3
  - USW G-3/GU-3
  - USW UZ-1
  - USW H-1
  - USW G-1

Table 6-9. Design Analyses Required for Borehole Seals

Design Analyses	Site Properties/ Conditions Required for Design	Repository Design Features Required for Seal Design[a]
Borehole Seal		
• Flow Through Seal and MPZ	Saturated, Bulk Rock, Hydraulic Conductivity of the Calico Hills	
	MPZ Model for Borehole in the Calico Hills	
• Structural Stability of the Borehole Seal		
- Thermal	Geothermal Temperature	Elevated Temperatures at Seal Location Due to Radioactive Heat Generation
	Repository Temperature	
- Gravitational/ Hydrostatic	Earth Pressure Within the Backfill	
	In Situ Stresses in the Calico Hills	
	Water Pressure Within the Backfill	

(a) Appropriate repository design features will be taken from SNL (1987b) or subsequent updates of the repository design requirements report.

requirement for this seal should be equivalent to the bulk hydraulic conductivity of the fractured Topopah Spring unit.

7.0 SEAL MATERIAL SELECTION

This chapter describes the sequence of steps that lead to the selection of candidate materials for seals and backfills. Materials selection incorporates a series of screens in which materials properties or performance characteristics are evaluated with respect to the design requirements (i.e., desirable materials properties) developed in the previous chapter.

7.1 OVERVIEW OF THE MATERIALS SELECTION METHODOLOGY

The materials selection process includes a screening procedure that is devised to determine candidate seal materials that will satisfy the functional requirements of repository seals and the specific design requirements determined for individual seal components. The materials screening procedure provides the rationale for the initial selection and subsequent refinements in the selection of materials. The candidate materials will be included in continued materials testing activities and incorporated into the Advanced Conceptual Design.

The materials screening logic and activities are summarized in Figure 7-1. Each materials selection screen corresponds to a step in the development of seal designs. Each screen examines whether materials possess several physical and chemical properties that must be met by the candidate seal materials, as determined from seal functional and design requirements. Thus, each screen is the impetus for materials performance testing and analysis.

The initial screen (completed before developing the design requirements) was a qualitative comparison between the functional requirements of seals described in Fernandez and Freshley (1984) and the expected performance of broad materials groups that might be expected to be applied to sealing a tuff repository. The second materials screen identifies more specific materials types by means of a matrix of design requirements, emplacement considerations, and environmental conditions. The third materials screen to be conducted as part of the Advanced Conceptual Designs will be based on specific design requirements determined by design analysis.

The materials selection criteria become increasingly quantitative and objective in more advanced screens. The first screen identifies broad materials groups such as cementitious and earthen materials and explains why other materials such as metals or ceramics are not being evaluated in current testing. The second screen identifies specific material types such as "high-temperature concrete" or "low-permeability crushed rock/clay" for each of the seal components that may be included in the design. The third screen (not yet conducted) will identify detailed materials formulations.

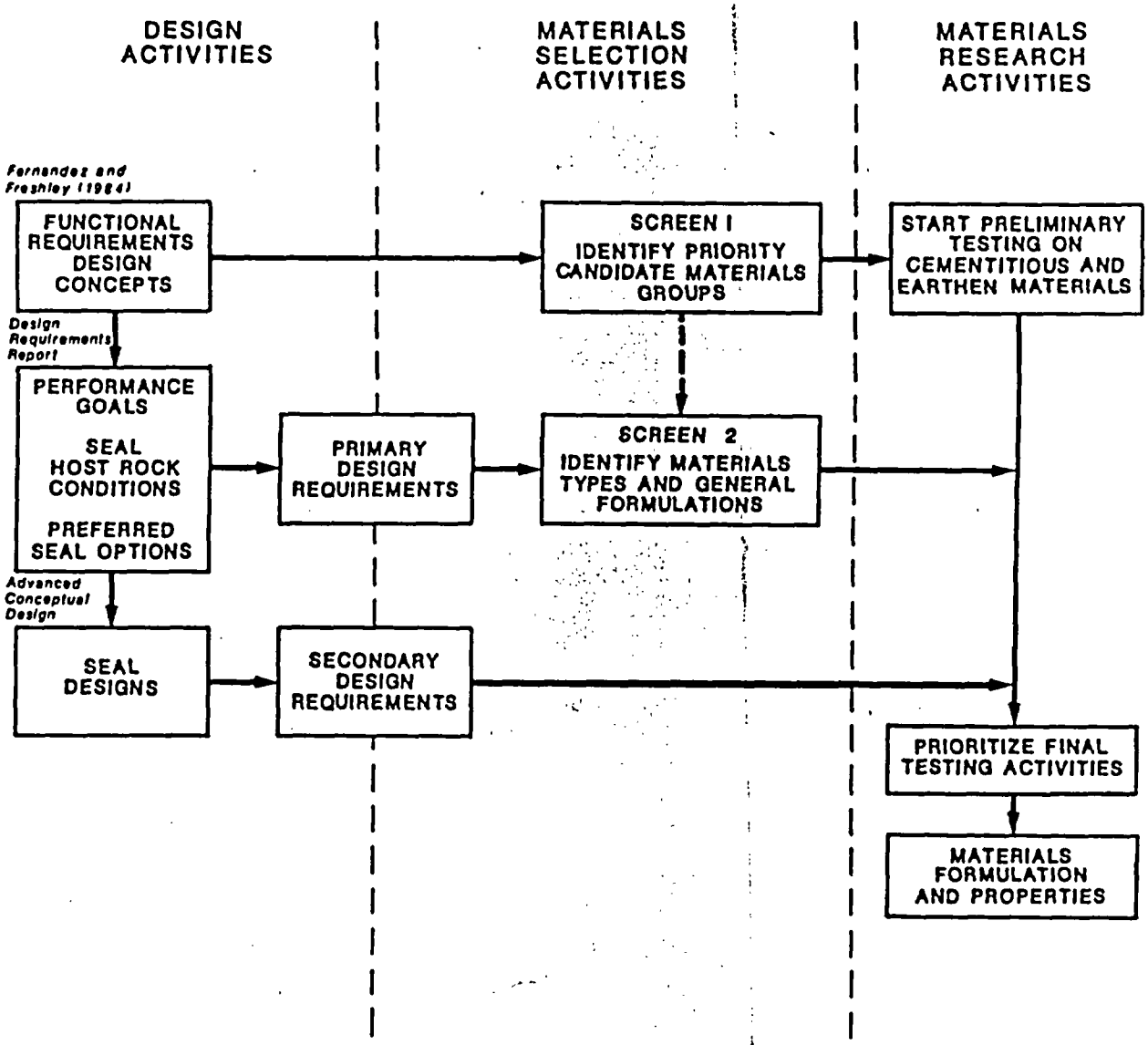


Figure 7-1. Summary of Materials Selection Activities Through Advanced Conceptual Design

7.2 INITIAL SCREEN

The initial screen was a qualitative evaluation of generic materials groups, with respect to general functional requirements. This screen was based on engineering judgments and was conducted after the initial design concepts for seals were developed by Fernandez and Freshley (1984).

The initial bases for screening and selecting materials were the functional requirements for seals described by Fernandez and Freshley (1984):

- containment and isolation, i.e., reducing the potential for radionuclide migration;
- human intrusion, i.e., deterring entry to the repository;
- longevity, i.e., providing materials that will perform as required over 10,000 years; and
- cost, i.e., other factors being equal, providing materials that are readily available and least expensive.

Materials properties and requirements that may relate to meeting functional requirements include the following:

- low permeability, to reduce radionuclide migration potential by limiting ground-water flow;
- moderate to high strength, to resist host rock deformations, provide structural support, or maintain low permeability by limiting the formation of fractures;
- chemical stability under in situ temperatures, to ensure adequate performance of seal for up to 10,000 years;
- chemical stability and inertness, to limit the potential for seal materials or their byproducts to affect waste isolation by, for example, enhancing the mobility of radionuclides;
- experience in materials applications in large engineering projects, which may help reduce development costs associated with research, materials handling, long-term performance, and emplacement technology for underground application; and

- cost and availability of raw materials used in constructing seals, as a practical guide to overall sealing costs.

This properties and requirements list is not exhaustive, but it is the basis for a qualitative assessment of candidate materials groups. The initial screen was designed to determine candidate materials groups that, in general terms, were likely to be capable of meeting seal functional requirements.

Figure 7-2 shows our subjective judgment as to how well each of several potential types of seal materials meet the desirable properties. Whereas all materials groups possess at least some desirable characteristics for sealing, differences among materials appear in the uncertainty associated with long-term stability, experience with engineering applications and emplacement in mined geologic structures, and cost and availability. Those materials groups that appear less desirable as seal materials include:

- Polymer cements, polymer concretes, and polymer impregnated concretes (PIC) can be formulated to have superior strength and permeability characteristics, compared with typical portland cement-based materials. Also, polymer cements have been developed for cementing geothermal wells and have proven stable (Coons et al., 1982). The organic content of these cements may be geochemically incompatible in a high temperature environment. These materials are extremely expensive, however, and should not be considered unless portland cement-based materials are found to be unsatisfactory.

- Inorganic chemical grouts have proven to be satisfactory in conventional engineering operations for sealing fine-aperture fractures, but their long-term performance is unknown. Many chemical grout formulations also include organic compounds that may affect radionuclide solubilities (e.g., by complexing) and would require some further investigation. Other potential problems with chemical grouts are shrinkage and strength deterioration (Karol, 1982).

- Organic chemical grouts may also appear desirable in sealing, particularly because low-viscosity grouts can penetrate deeply into fractured rock. The use of these materials will, however, introduce organic chemicals into the repository environment, many of which are toxic and difficult to handle. Considerable development with respect to long-term stability, effects on isolation, and materials handling may be required with these materials (Karol, 1982).

MATERIALS GROUP	LOW PERMEABILITY	HIGH TO MODERATE STRENGTH	CHEMICAL STABILITY AT TEMPERATURE (0 - 250°)	VOLUMETRIC/THERMAL STABILITY AND/OR EXPANSIVITY	EXPERIENCE WITH MATERIAL ENGINEERING APPLICATION	CHEMICAL EFFECTS ON ISOLATION	COST AND AVAILABILITY
Portland Cement-Based Concrete	●	●	O	O	●	O	●
Portland Cement-Based Grout	●	●	O	O	●	O	●
Clay-bearing Materials	●	●	O	●	●	●	●
Crushed Rock	O	●	●	●	●	●	●
Polymer Cement, Poly. Concr, PIC	●	●	O	O	X	O	X
Inorganic Chemical Grout	●	●	O	O	●	O	●
Organic Chemical Grout	●	●	O	O	X	O	X
Bitumen	●	X	O	X	O	O	●
Thermo Plastics	●	●	O	O	X	O	O
Thermoset Resins	●	●	O	O	X	O	O
Metals	●	●	O	●	X	X	O
Ceramics	●	●	●	●	X	O	O

● Expect acceptable performance can be achieved.
O Materials performance is uncertain.
X Materials performance may be unacceptable.

Figure 7-2. Initial Materials Screen for Candidate Seal Materials

- Bitumen (asphalt, rock asphalt, etc.) is frequently used for waterproofing and may be used like a grout to seal permeable zones, interfaces between other seals and host rocks, or boreholes or larger openings (e.g., sandwiched between stiff bulkheads or plugs). These materials have little thermal stability; and their physical properties change at elevated temperatures. This might limit their applicability in some sealing applications or seal locations. Although bituminous materials are very insoluble in water, they may introduce complexants that enhance radionuclide solubility, particularly over the long-term performance of the seal system.

- Thermoplastic and thermoset resins may have value for low permeability and reasonably high strength. Applications in large-scale engineering projects, materials handling, and emplacement technology would need significant development.

- Metals may be considered in the seal system for bulkheads, supportive structures, or reinforcement. A potential problem in a tuff repository may be oxidation of metal components. Metal seal components would probably have to be used in combination with other materials such as concrete or clay.

- Ceramics (complex mixtures of metal oxides to which an external heat source is applied to partially or completely fuse the mix) share many characteristics of metals (low permeability, heat resistance, and strength), but they may be difficult to emplace (particularly if external heat is required in situ) without developmental studies.

The outcome of the initial screen shows that the materials groups possessing desirable properties that are also familiar in engineering applications are portland cement-based concrete and grout, clay-bearing materials, and crushed rock. In addition to generally satisfactory engineering properties, the reasons for selecting these candidate materials include the following:

- The candidate materials are widely used in construction, and they have familiar properties and engineering characteristics. The need for basic research in materials development and characterization can be limited by the use of these familiar and well-studied materials.

- The technology to emplace these materials in shafts, boreholes, tunnels, or drifts is developed and, perhaps with some modification, can be used in repository sealing. Thus, research in emplacement technology may be minimized.

- The materials are widely available and relatively inexpensive.

7.3 SECOND MATERIALS SCREEN

Development of design requirements for seal materials (as presented in Chapter 6) allows a more specific identification of the materials types that will probably be used for each seal component. At this stage, the selection of materials types is based on primary design requirements (especially hydraulic conductivity), emplacement considerations, and environmental conditions. The next materials screening will be based on additional design requirements that emerge from the Advanced Conceptual Design, and on the results of ongoing materials testing that will confirm the assumed properties of the selected materials or form the basis for revision of assumed materials properties.

The second materials screen is based on the following requirements or conditions:

- hydraulic conductivity,
- strength or density,
- emplacement considerations,
- potential influence of seals on ground-water chemistry,
- emplacement conditions
  - -host rock lithology/chemistry
  - -temperature range at seal location.

Table 7-1 summarizes these requirements and conditions for each of the seal components that may be included in the overall system. For several seal components, low to moderate hydraulic conductivity is the primary design requirement. Required values range from 10^{-3} cm/s for sealing boreholes drilled from the surface to 10^{-8} cm/s for some bulkheads. (Note that these bulkheads are not preferred sealing options because of the low hydraulic conductivity required.) In contrast, backfills placed in drainage channels or sumps require high hydraulic conductivity, in the range 1 to 100 cm/s.

Moderate to high strength or density is required for components in which settlement or cracking could lead to increased hydraulic conductivity. Seals such as the anchor-to-bedrock plugs, shaft or ramp station plugs, and fault seals in horizontal emplacement boreholes are intended to provide low effective hydraulic conductivity by forming a tight interface against the host rock. Seals placed in drifts are intended to be compacted sufficiently that settlement will be limited. In contrast, backfills placed in drainage

Table 7-1. Summary of Design Requirements and Environmental Conditions for Seal Components

Design Requirements/ Host Environment	Component: Anchor-to-Bedrock Plug (Shaft, Ramp)	Station Plug[b] (Shaft or Ramp)	General Backfill (Shaft, Ramp or Drift)	Surface Cover[a] (Shaft, Ramp)	Drainage Channel Backfill (Access) or Main Drift
Hydraulic Conductivity	10^{-5}-10^{-4} cm/s	10^{-6}-10^{-5} cm/s	10^{-2} cm/s	No requirement	>100 cm/s
Strength/Density	High to increase durability, stability and permanence if exposed	High to limit movement under shaft fill load	Moderate density to limit compaction/ compressibility	High to limit intrusion, increase durability stability and permanence	Graded to limit silting of pore volume
Emplacement Considerations	Bulk or pumpable	Bulk or pumpable	Bulk	Bulk pumpable, or preformed	Bulk
Emplacement Conditions					
Host Rock	Alluvium/ Tiva Canyon	Topopah Spring	Various[c]	Alluvium/ Tiva Canyon	Topopah Spring
Temperature Range ~~(°C)~~	Near surface ambient (-27 to 53)	Ambient to <100	Ambient to >100	Surface ambient (-27 to 53)	Ambient to >100
Influence on ground-water chemistry near waste packages[d]	None[e]	None[e]	None[e]	None[e]	Possibly significant

(a) May be combined with anchor-to-bedrock plug.
(b) May not be required in shafts, preferred option is anchor-to-bedrock plug.
(c) To the extent possible, crushed, welded tuff should be placed as a backfill where welded tuff occurs. Crushed, nonwelded tuff should be placed where nonwelded tuff occurs.
(d) The influence of sealing components is probably not significant for all components associated with horizontal emplacement. The only possible exception would be the horizontal emplacement hole.
(e) To be further evaluated in subsequent studies.

Table 7-1. Summary of Design Requirements and Environmental Conditions for Seal Components (Concluded)

Design Requirements/ Host Environment	Component						
	Drift Dam (Emplacement Drift)	Drift Dam (Perimeter or Mains	Drift Bulkhead[f]	Fault Seals (Emplacement Borehole)	Fracture Treatment (Grout)	Borehole Surface Cover	Borehole Plug
Hydraulic Conductivity	10^{-5}-10^{-4} cm/s	$<10^{-4}$	10^{-8}-10^{-7} cm/s	10^{-7}-10^{-6} cm/s	$<10^{-5}$ cm/s	No requirement	$<10^{-3}$ cm/s
Strength/ Density	High density to limit settlement	High density to limit settlement	High to resist rock loads and resist cracking	High to prevent settlement/ cracking	High to prevent reopening fractures	High to limit intrusion, increase durability, stability and permanence	High to prevent settlement and gap formation
Emplacement Considerations	Bulk or pumpable	Bulk or pumpable	Bulk or pumpable	Pumpable into fine aperture fractures	Pumpable into fine aperture fractures	Bulk, pumpable, or preformed	Pumpable
Emplacement Conditions							
Host Rock	Topopah Spring	Topopah Spring	Topopah Spring	Topopah Spring	Tiva Canyon, Topopah Spring	Alluvium/ Tiva Canyon	Various, including Calico Hills
Temperature Range (°C)	Ambient to >100 (estimated range 30 to 110)	Ambient to <100	Ambient to >100 (estimated range 30 to 110)	Ambient to >100 (estimated range 30 to 200)	Ambient to >100 (estimated range 30 to 200)	-27 to 53	Ambient to <100
Influence on ground-water chemistry near waste packages[d]	Possibly significant	Minor	Possibly significant	Possibly significant	Possibly significant	None	None

[f] Double bulkhead with water flow occurring under confined conditions.

channels should be loosely compacted to provide high hydraulic conductivity, but they should also be graded so that permeability is not reduced by silting.

Emplacement considerations are also significant with respect to materials selection. Whereas some materials are required for filling large openings (e.g., drift seals or backfills), others are required for relatively small openings (e.g., emplacement hole seals, grouts). Large openings afford easy access to machinery and workers (bulk emplacement), but small openings may have to be filled remotely (e.g., by pumping material into a seal zone). Some components such as grouts that are required to fill cracks must be emplaced by pumping under pressure, whereas others such as backfills can be placed mechanically with little or no compaction. Other components such as shaft and borehole covers may be preformed or consist of several preformed units that can be moved into place as integral units.

An additional consideration in the selection of seal materials is that the chemical interaction between the seals and the ground water should not alter the ground-water chemistry to the degree that this could have a significant deleterious effect on waste package performance. Deleterious ground-water effects may include

- increasing waste package container corrosion rates,
- increasing waste form solubility or dissolution rates, and
- increasing the solubility and mobility of radionuclides.

Corrosion of the metallic waste package container may be increased by lowering the ground-water pH or increasing anion concentrations (e.g., carbonate, chloride, fluoride). Silica glass waste form solubility may be enhanced by increased pH (alkaline conditions), reduced silica activity, or increased alkali or alkaline earth activities in ground water. Several anionic species may increase the solubility and mobility of radionuclides by forming soluble aqueous species with radionuclides; these include carbonate, fluoride, sulfate, nitrate, and phosphate. A goal for seal materials selection, therefore, is to ensure that alteration of ground-water composition by interaction with seal materials results in little change in pH and little increase in cation and anion concentrations.

A final set of parameters that contribute to the types of materials required for sealing a tuff repository are the conditions to which the material will be exposed and under which the material must meet design requirements after emplacement in a seal zone. Among the conditions that may affect seal performance are the following:

- range of temperatures;
- host rock mineralogy, and chemical and physical properties;

- moisture regime, including the degree of saturation and total water available in a seal zone;

- chemistry of water associated with a seal horizon; and

- local stresses, imposed by overlying seals and backfills, in situ stresses, tectonic stresses, and seismic loads.

It will not be practical to select a different material for every possible combination of host rock properties and environmental conditions because this would lead to an impracticable number of materials, each requiring detailed evaluation. Moreover, it is important to note that rock properties, groundwater chemistry, and other environmental conditions can vary locally within a seal zone as well as on a larger scale. Each material should thus be selected to be compatible with a range of environmental conditions. The parameters considered to be most significant in screening materials are the host rock type and the expected temperature range. A suggested classification is as follows:

Host Rock	Temperature
• alluvium	surface -27° to 53°C (Eglinton and Dreicer, 1984, Table 19)
• welded tuff (Topopah Spring or Tiva Canyon)	near surface -27° to 53°C (ibid.)
• welded tuff	ambient to >100°C
• Calico Hills	ambient to <100°C

This matrix of environmental conditions is used in combination with the design requirements for hydraulic conductivity, strength/density, and emplacement considerations to identify 10 materials types (Table 7-2). These materials types and their associated design requirements are the basis for ongoing testing of concretes, grouts, and earthen materials and for the assignment of properties in design analyses. The materials types may be modified in the future on the basis of further testing. For example, testing may determine that a single concrete, or a single grout, may be applicable to all of the host rock/temperature range combinations.

7.4 PROPERTIES OF CANDIDATE SEAL MATERIALS

7.4.1 Properties of Portland Cement-Based Concrete and Grout

The experience with portland cement-based materials is very extensive, and a large data base on concrete and grout performance, chemistry, emplacement, and

Table 7-2. Summary of Candidate Materials for Seal Components in Tuff

	Design Requirements				Emplacement Conditions			
	Hydraulic Conductivity	Density and Strength	Emplacement Methods	Effect on Ground-water Chemistry	Host Rock	Temperature	Components	Candidate Material
1	low	high	bulk or preform	less important	Alluvium	-27° - 53°C	surface cover[a] (shaft, borehole)	standard concrete 1
2	low	high	bulk	less important	Alluvium/ Tiva Canyon	-27° - 53°C	anchor-to-bedrock plugs (shafts and ramps)	standard concrete 2
3	low	high volumetric stability	bulk	less important	Topopah Spring	ambient to <100°C	station plug, drift seal (perimeter, drift or main)	compacted earth* or standard concrete 3
4	low	high	pumpable	less important	Tiva Canyon/ Topopah Spring	ambient to <100°C	grout[b]	standard cementitious grout 1
5	low	high volumetric stability	pumpable or tamped	less important	Calico Hills/ Topopah Spring	ambient to <100°C	borehole plug	standard cementitious grout 2 or compacted earth*
6	low	high volumetric stability	bulk	more important	Topopah Spring	ambient to >100°C	bulkhead (emplacement drift)	compacted earth* or high-temperature concrete
7	low	high	pumpable	more important	Topopah Spring	ambient to >100°C	emplacement borehole seals	high-temperature cementitious grout
8	no requirement[c]	volumetrically stable	bulk	more important	Tiva Canyon/ Topopah Spring/ nonwelded tuff	ambient to >100°C	backfill (shafts, ramps, drifts)	crushed tuff
9	highly permeable	volumetrically stable	bulk	more important	Topopah Spring	ambient to >100°C	drainage channel backfill, fault sump backfill	crushed tuff (fines removed)

*The preferred option is crushed tuff with 10% to 30% clay or fines from crushed tuff.
(a) Surface cover for shafts and ramps not currently proposed in the NNWSI Repository Sealing Program.
(b) Used in conjunction with standard concrete to seal around bulkheads.
(c) For shaft and ramp fill, a hydraulic conductivity of less than 10^{-2} cm/s is proposed.

durability is available. Concretes (i.e., cementitious materials with fine and coarse aggregate) are known chiefly for high strength, but mix and curing variables (such as the types and proportions of key ingredients and admixtures) can result in materials possessing a wide range of properties, including low strength, somewhat plastic materials. For example, the compressive and shear strength of concrete can be controlled within a wide range of values that overlaps the strength range of the rocks in which seals will be emplaced. The compressive strength of concretes typically falls in the range 7 to 124 MPa (B. Mather, 1966, 1967; K. Mather, 1965), depending on factors such as temperature, age, water:cement ratio, aggregate type and content, additives, environmental factors such as drying, etc. The compressive strength of grouts (i.e., pumpable cementitious materials with no coarse aggregate) can display similar wide variations.

The range of typical concrete permeabilities is $<1\times10^{-5}$ to 1×10^{-3} Darcy, or in terms of hydraulic conductivity, $<1\times10^{-8}$ to 1×10^{-6} cm/s, although hydraulic conductivities less than 10^{-10} cm/s are achievable (see B. Mather, 1967). Values obtained by the NNWSI Repository Sealing Program for two grouts and one concrete ranged from 1.6×10^{-10} to 9.5×10^{-10} cm/s (Appendix G, Table G-4). The hydraulic conductivity of these materials is comparable to that of unfractured tuff (Table 2-2). Laboratory and field studies indicate, however, that the interface between the rock and cementitious materials may be more permeable than either the seal or rock and may be a preferred flow path (e.g., Peterson and Christensen, 1980; Wakeley and Roy, 1982). As noted in Chapter 6, the MPZ around an excavation may also be a preferred pathway.

The longevity of portland cement-based concretes and grouts is perhaps a significant issue related to their use in sealing. Cured materials are probably geochemically metastable under repository conditions, and changes will undoubtedly occur in the chemistry, mineralogy, and microstructure of the cementitious seal materials over long periods of time and under high temperatures. These changes could also induce volume changes. Characterization of cementitious-based materials under elevated temperature is being addressed by continued materials research and testing.

A specific concern related to the longevity of materials is that chemical interaction between seal materials and the ground water should not have an adverse influence on ground-water chemistry (Section 7.3). Current studies suggest that under repository conditions, including temperatures up to 250°C, cementitious grouts and concretes will interact with ground water and alter its composition. For example, Scheetz and Roy (in preparation, pp. 42-45) show that a portland cement-based concrete using tuff aggregate may increase the calcium, silica, sulfate, and pH of J-13 reference ground water when reacted in a closed system at 150° and 200°C. The ground-water composition appears to change as a function of both temperature and time. Changes in some chemical components in the ground water do not vary consistently or systematically with respect to time and temperature, however, and the growth of secondary phases appears to control solubilities of several species (silica,

sodium, calcium sulfate) in long-term experiments. In evaluating these results, it should be noted that the experimental data may not accurately reflect actual ground-water/concrete interactions in situ, i.e., under repository conditions, interactions will occur in an open system. The results from these relatively short-term (6 weeks) experiments may not reflect long-term effects of ground-water/concrete interactions on ground-water compositions and do not represent the effects of changing physiochemical conditions over time, such as temperature, and the growth of stable secondary-phase assemblages. Also, the effects on the composition of any ground water that contacts the waste will depend on the water flow rate through the seals and the effects of mixing with water flowing from the host rock. Nonetheless, the data do suggest that cementitious materials have the potential to alter the ground-water chemistry.

7.4.2 Properties of Earthen Materials (Clays and Crushed Rock)

There is a considerable data base regarding the properties of earthen materials obtained from general geological studies, the use of clays in water-retaining structures such as dams and slurry walls, and from research related to nuclear or hazardous waste disposal. A review of hydrologic properties (porosity and hydraulic conductivity) is given in Appendix D for crushed tuff and rock-clay mixtures. A wide range of properties can be obtained depending on particle size distribution, clay type and content, water content, and degree of compaction. Porosities can range from about 25% to 50%, with values in the range 30% to 35% expected for a compacted, crushed tuff backfill. Hydraulic conductivity can vary over a very wide range. For cohesionless materials (i.e., with no clay), values may range from as high as 100 cm/s for a clean, coarse gravel or rock fill to 10^{-5} cm/s for a fine silt. Specific values within this range can be engineered by crushing and screening crushed tuff. Lower values of hydraulic conductivity can be obtained by adding clay to crushed tuff. For example, a value of about 10^{-10} cm/s can be obtained from a mixture of crushed tuff with 30% Na-bentonite (see Appendix D).

To achieve low, effective hydraulic conductivities in earthen materials, it is necessary to avoid cracking or gap formation at the interface between the backfill and the host rock. Cracking may result from drying, chemical reactions accompanied by volume reduction, tectonic effects, or structural effects (e.g., compaction, unequal loading). The effects of cracking might be minimized by use of a swelling clay such as bentonite, although further research is needed to verify this supposition. The potential for gap formation can possibly be reduced by the careful selection of seal materials. For example, increased cation concentrations in ground water may alter clays or zeolites that are emplaced in the facility or occur naturally in the tuff. Alteration of clays such as smectites may reduce their swelling capacity. As indicated in Scheetz and Roy (in preparation, pp. 42-45), a portland cement-based concrete together with J-13 water may increase the concentrations of several cations within the water. Therefore, use of concretes together with clays could affect the swelling capacity of the clay. If the swelling capacity of

the clay is determined to be important, then the use of a clay and concrete together should be avoided.

Materials used as room or emplacement hole backfill or in other seal components may be required to retain adequate performance over a long period of time. It is anticipated that crushed tuff will be geochemically stable under repository conditions, and clay minerals are generally stable over a wide range of geological environments (see Grim, 1968). Further research may be required if clays are to be used at temperatures greater than 100°C.

As with cementitious materials, a specific concern regarding longevity is the effect of interaction between seal materials and the ground water on ground-water chemistry. Limited experimental evidence suggests that earthen materials seals, containing clays, crushed tuff, or a combination of these materials may alter ground-water composition upon interaction under repository conditions. For example, Grim (1968, pp. 446-448) indicates that silica may be dissolved from clays (specifically montmorillonite and attapulgite, 90°C, 4:1 water:solids ratio) in neutral to slightly alkaline solutions. The results quoted by Grim are not steady-state values, and the potential for significant silica removal from clays in alkaline solutions (presumably with low silica activities) is inferred. A study of Na-montmorillonite interaction in a closed system with calcium-free, basaltic (high pH) ground water (25:1 water:solids ratio) at 150° and 200°C for up to 6 months showed increased sodium, potassium, calcium, magnesium silica, and aluminum in the altered fluid (Howard and Roy, 1985, p. 83). These limited data suggest that under repository conditions clay/ground-water interaction may result in the release of several species to the ground water. These experimental results must be cautiously applied to the tuff repository environment, however, because the starting fluid compositions in these experiments are not representative of the tuff repository. For example, the silica concentration used by Howard and Roy (about 100 ppm) was lower than the silica concentration expected in associated ground water under tuff repository conditions (over 250 ppm at 150°C) (Oversby, 1984, p. 17). In addition, the experimental fluids had starting pH values (9.8) that are higher than the pH expected in the tuff repository ground water (generally less than 8.0, Oversby, 1984, p. 47). Based on these limited experimental results, it appears that clay/ground water interactions may result in small compositional changes in the ground water; however, this subject has not been fully addressed.

Interaction of J-13 ground water and crushed Topopah Spring tuff at 150°C resulted in increased potassium, silica, aluminum and pH, decreased magnesium and variable changes (usually decreased) in sodium and calcium in the ground water (Oversby, 1984, pp. 47, 49-51, 55-58). Similar reactions may be expected from reactions between ground water and crushed tuff. These reactions may recur at a faster rate because of the larger surface area in a crushed material. Additional work is required to determine whether some reactions could be different in a crushed material than in the host rock, for example because of differentiation of some chemical components during crushing.

7.5 MATERIALS RECOMMENDATION SUMMARY

The second materials screen (Section 7.3) indicated that the preferred seal materials are earthen materials (crushed tuff with fines added if low permeability is required) or cementitious materials (grout or concrete). With the assumption that a single grout and a single concrete cannot be applied over a wide temperature range, the material groups identified below may be required for all application:

- standard concrete 1 (freeze-thaw resistant),
- standard concrete 2 (temperature >0°C, <100°C),
- standard concrete 3 (temperature >100°C),
- standard grout 1 (temperature <100°C),
- standard grout 2 (temperature >100°C),
- crushed tuff and clay (low permeability), and
- crushed tuff, fines removed (high permeability).

The number of materials will be reduced if both a single grout and a single concrete can be applied over the whole temperature range from <0°C to >200°C, i.e.:

- concrete,
- grout,
- crushed tuff,
- crushed tuff and clay, and
- crushed tuff, fines removed.

Currently, the NNWSI Repository Sealing Program is conducting research on both cementitious and earthen materials. Our current preference is to propose earthen materials for as many applications as possible to minimize potential degradation of physical properties and potential adverse effects on groundwater chemistry in the repository environment. Within the higher temperature environments, i.e., within the emplacement drifts, earthen materials are anticipated to be more stable than concretes and grouts. Their impact on waste package corrosion, waste form solubility, and mobility of radionuclides is considered to be less severe than cementitious-based materials. This conclusion is reached by comparing potential increases in pH and cationic species. Therefore, earthen materials are preferred for use in emplacement drifts. In places where low strength is acceptable, earthen materials may also be used. The most likely application for cementitious materials is where high strength and low deformability may be required (e.g., in some bulkheads) or where high strength combined with pumpability may be required (e.g., for grouting fractures).

8.0 CONCLUSIONS

The purpose of this report is to provide a technical basis for the performance goals to be used in the NNWSI Repository Sealing Program and for the design requirements associated with specific, sealing components. These performance goals and design requirements are then used to provide a more definitive basis for the seal design development, including selection of suitable materials for sealing components. Conclusions supporting seal design development are summarized below.

By comparing the design-basis-performance goals* with the anticipated water flows into the repository, it is concluded that limited sealing efforts (e.g., emplacing crushed tuff fill in shafts and ramps) are sufficient to meet the quantitative NRC performance requirements in 10 CFR 60.113a.1.ii.B. However, to meet selected, unanticipated water flows into the repository, specific sealing measures may be required to provide additional assurance that performance objectives will be met. For the shafts and ramps, sealing measures are required for the first 1,000 years after closure. For the underground facility, sealing measures are required for the first 500 years following closure. To provide the assurance that the design-basis-performance goals can be met, even if unanticipated conditions are encountered, a broad range of sealing options and associated design requirements are proposed.

For shafts and ramps, the preferred option is the emplacement of an anchor-to-bedrock plug (a sealing component located in the upper portion of the shaft or ramp) together with a general fill in the majority of the shaft and ramp. This fill should have a saturated hydraulic conductivity of less than 10^{-2} cm/s. The shaft liner below the repository station should also be removed. Preferred sealing options in the underground facility for the vertical emplacement configuration is a combination of sumps and earthen dams that may, in selected nonemplacement areas, be supplemented with channels. In the underground facility, for the horizontal emplacement configuration, the preferred option would be to abandon emplacement boreholes in which water is encountered. For exploratory boreholes, the emplacement of a seal, where the Calico Hills unit is penetrated, is preferred. Two design requirements associated with exploratory boreholes include: (1) the underground facility does not intercept any borehole, and (2) a minimum distance of 15 m occurs between a drift excavation and the borehole.

Considering the design options and design requirements presented in the early portions of this report, potential sealing materials are identified. It is concluded that five materials are suitable for use as sealing components:

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*The design-basis-performance goals are performance goals selected for developing seal designs. The design-basis performance goals selected for use in this report are reduced from the maximum-allowable-performance goals to provide an added degree of conservatism.

- concrete,
- grout,
- crushed tuff,
- crushed tuff and clay, and
- crushed tuff, fines removed.

Our preference in the NNWSI Repository Sealing Program is to propose earthen materials for as many applications as possible to minimize potential degradations of physical properties and potential adverse effects on ground-water chemistry in the repository environment. Specifically, earthen materials are preferred in emplacement drifts and where low strength is acceptable. If high strength, low deformability, or pumpability is required, cementitious materials may be more appropriate.

Finally, it is important to indicate that the conclusions reached in this report are based on the available, site hydrologic information and engineering judgment. In this regard, these conclusions are preliminary and subject to change. Therefore, as results from the hydrologic tests conducted in the Exploratory Shaft Facility are obtained and as additional analyses are performed, the design bases, including design requirements will be modified.

APPENDIX A
HYDROLOGIC CALCULATIONS USED TO ESTABLISH THE NEED FOR SEALING

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APPENDIX A-1
SURFACE-WATER SUPPLY AND COLLECTION NEAR THE SHAFTS AND RAMPS

A-1.1 GEOLOGY AND MORPHOLOGY OF SHAFT AND RAMP PORTAL LOCATIONS

All of the proposed shaft locations and one of the proposed ramp portal locations are within broad valleys or on valley walls near washes. Although the washes are normally dry, flooding may occur in the valleys during heavy rainfall. This section summarizes the geology and topography present at each shaft and portal location and the current designs for flood control.

A-1.1.1 Exploratory Shafts

The Exploratory Shafts (ES-1 and ES-2) are located in a wide valley through which the north and south forks of Coyote Wash flow at the northern and southern margins (Figure A-1.1). The valley floor slopes moderately (approximately eight percent) to the east and is underlain by coarse alluvium and mud/debris flow deposits, with surficial, fine-grained sand probably of eolian origin. Bedrock (Tiva Canyon Member) is exposed in the steep valley walls to the north and south and headward in the valley to the west. Bedrock is exposed in the washes upstream of the ES location. Downstream, the washes lie on alluvium. At boreholes USW G-4, near the shaft location, and UE-25a#6, near the mouth of Coyote Wash, alluvium is 6.5 to 10 m thick (Fernandez and Freshley 1984, pp. 60,69).

At the conclusion of this study, alternative locations for ES-1 and ES-2 were proposed. These proposed locations are also shown in Figure A-1.1. Both shafts will be collared in bedrock. Because of the recent decision to relocate the exploratory shafts to the proposed locations, details of surface development of the ES area are not available to incorporate here. Figure A-1.2 is a north/south cross-section through the site showing the existing topography and the modifications (assuming the original locations of the ESs) planned for site development.

A-1.1.2 Men and Materials Shaft

The proposed location for the Men and Materials Shaft (MMS) lies on the prow of a ridge separating Wren Wash to the north from a smaller wash to the south (Figure A-1.3). The ridge is underlain by colluvium over bedrock. The surface in the vicinity of the shaft location slopes toward the washes to the north and south. The valley walls north of Wren Wash and south of the southern wash are steep (up to about 40%). Headward, the ridge on which the shaft is located, also steepens (Figure A-1.3). The MMS is located at the edge of the limits of the probable maximum flood (PMF) as defined by the U.S.G.S. (Squires and Young, 1984).

In the current preliminary plan, a rectangular pad about 12 acres in area will be constructed at the MMS location (SNL, 1987a). Construction of the pad will

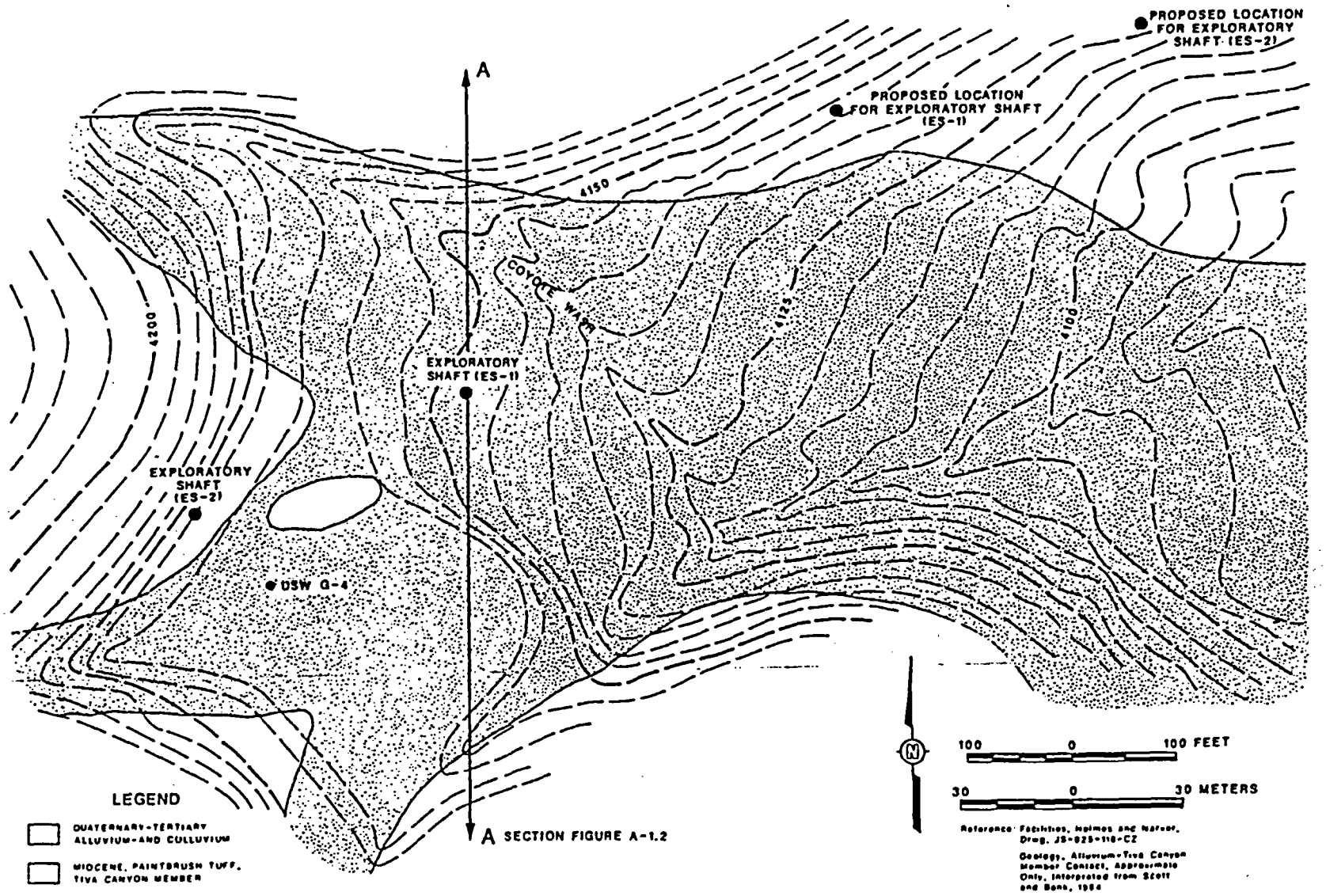

Figure A-1.1. Geology in the Vicinity of the Exploratory Shafts

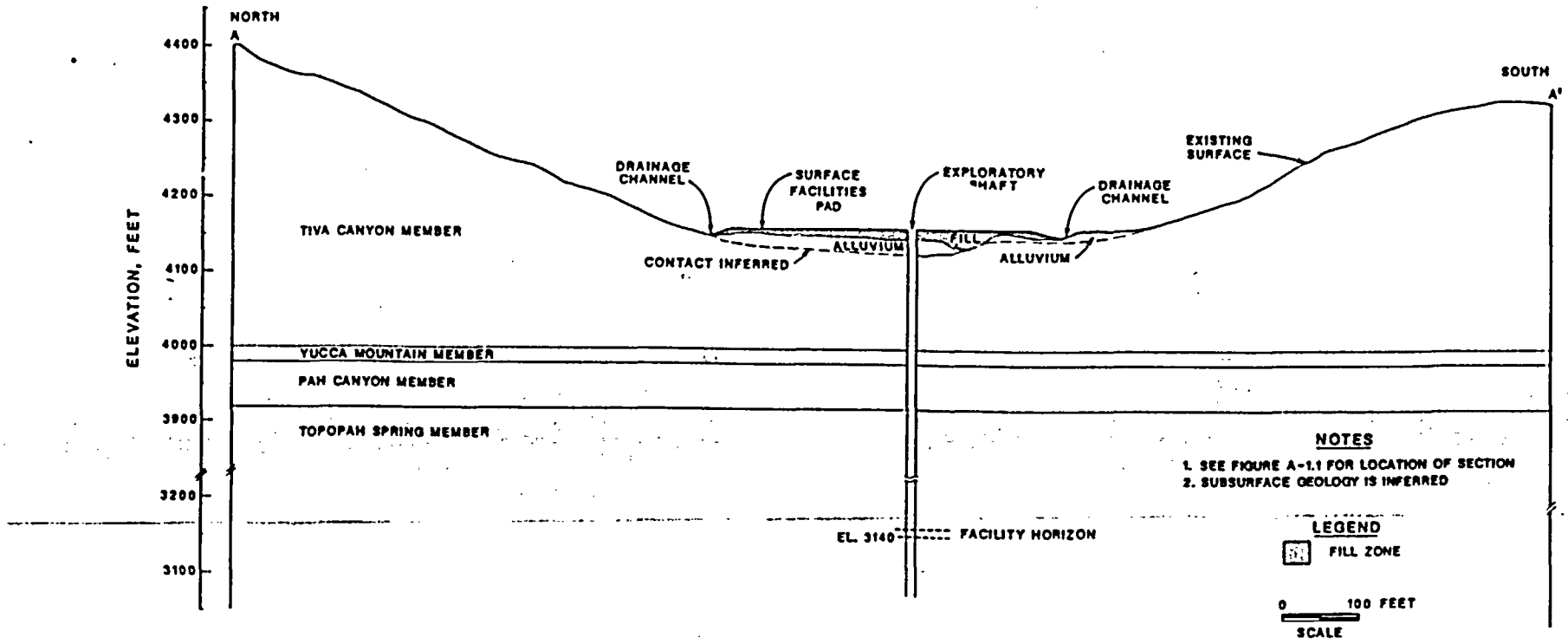

Figure A-1.2. Cross Section Through ES-1

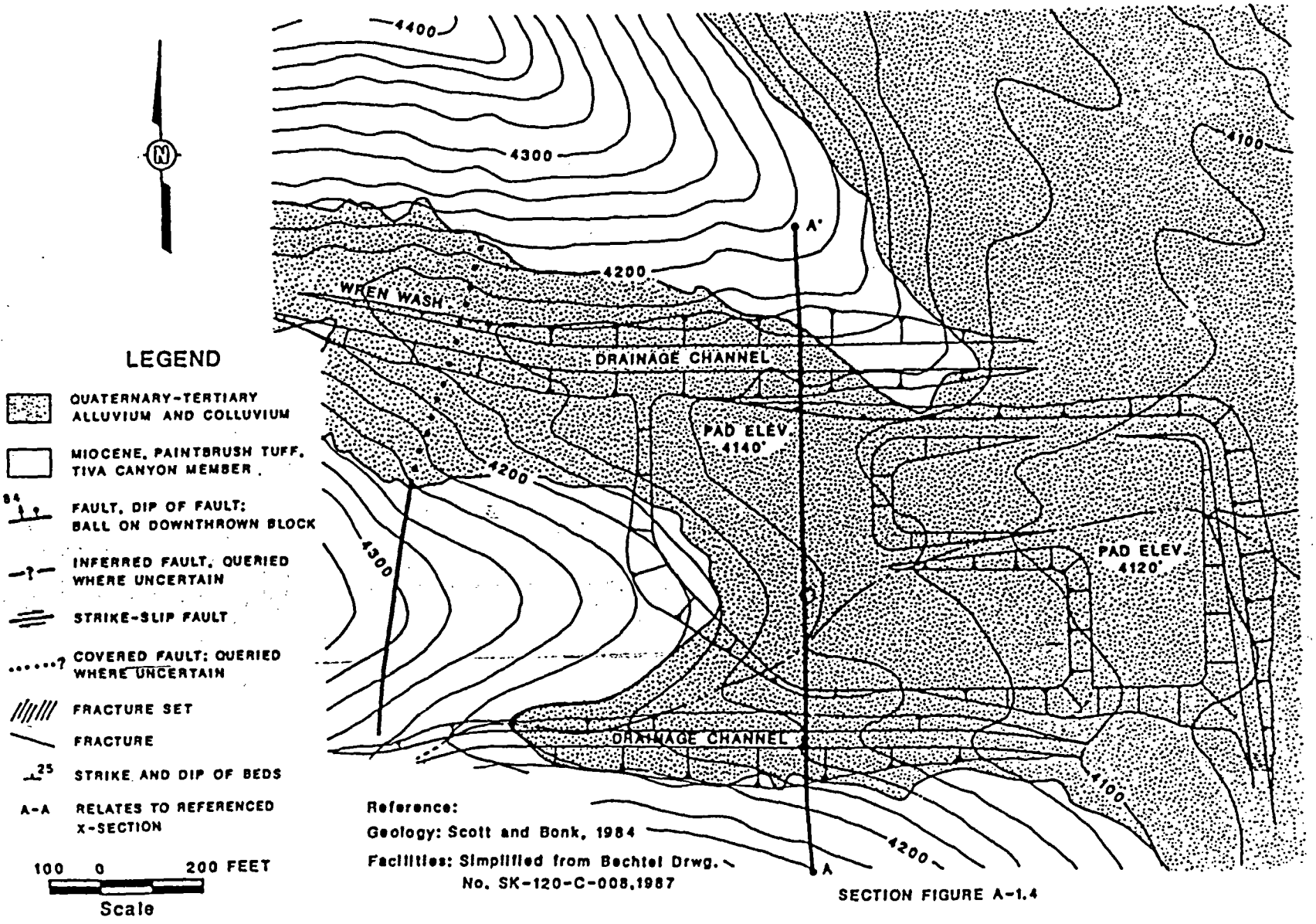

Figure A-1.3. Geology and Surface Facilities of Men and Materials Shaft and Environs

require up to 7.6 to 9.1 m of additional fill, particularly to the east, north, and south. Headward, however, surficial deposits and bedrock underlying the ridge on which the shaft is located (on the existing surface) will be cut away. North and south of the pad, two, approximately parallel, diversion channels will be constructed in the washes. In Wren Wash, the channel will have a bottom about 9.1-m wide. The southern channel will be about 7.6-m wide at the bottom. These will require excavation of bedrock forming the slopes lying north and south of the planned surface facilities as shown in Figure A-1.4, a north-south cross-section through the shaft location.

A-1.1.3 Emplacement Exhaust Shaft

The proposed location for the Emplacement Exhaust Shaft (EES) lies in the center of a broad valley surrounded on the north, south, and west by steep slopes underlain by bedrock (Figure A-1.5). The EES is located on alluvial/colluvial material covering the valley floor. The natural surface slopes gently away from the location to the north, south, and down-valley to the east. Washes lie along the north and south margins of the valley at the base of the steep valley walls. The preliminary site development at the EES (Figure A-1.5) calls for a pad to be constructed in the valley between the washes (SNL, 1987a).

A-1.1.4 Muck Handling Ramp Portal

The Muck Handling Ramp (MHR) portal proposed in the current design is located south of an unnamed wash about 76 m north of Pagany Wash. The portal will be excavated in bedrock (Tiva Canyon Member) and lies about 6.1 m above the floor of the adjacent wash to the north on a fairly steep (approximately 20% to 25%) slope (Figure A-1.6). The opposite (north) side of the wash is a broad, low terrace estimated in the field to be about 4.0 m below the portal (Figure A-1.7).

Several, preliminary, site development plans have been developed for the MHR Portal location (SNL, 1987a). Some plans call for diversion of runoff in the wash by berms. Alternative designs include blocking the wash or filling the valley and wash with waste rock.

A-1.1.5 Waste Handling Ramp Portal

The Waste Handling Ramp (WHR) portal is proposed to be constructed at the base of the eastern slope of Exile Hill (Figures A-1.8 and A-1.9). The location lies in alluvium/colluvium less that 15 m in thickness. There are no large drainages or washes on the east slope of Exile Hill, and no major diversion structures are planned at this site.

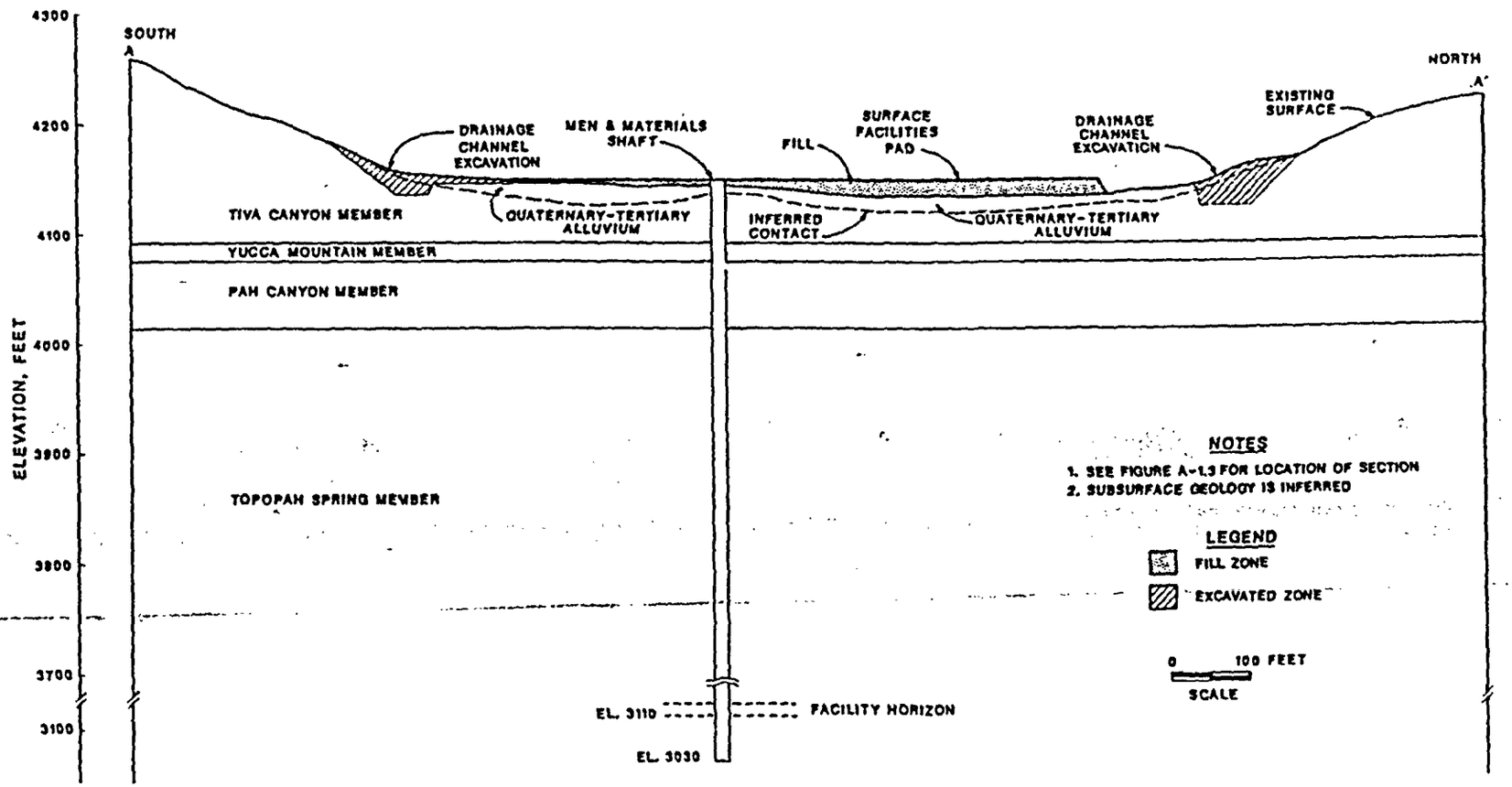

Figure A-1.4. Cross Section Through Men and Materials Shaft

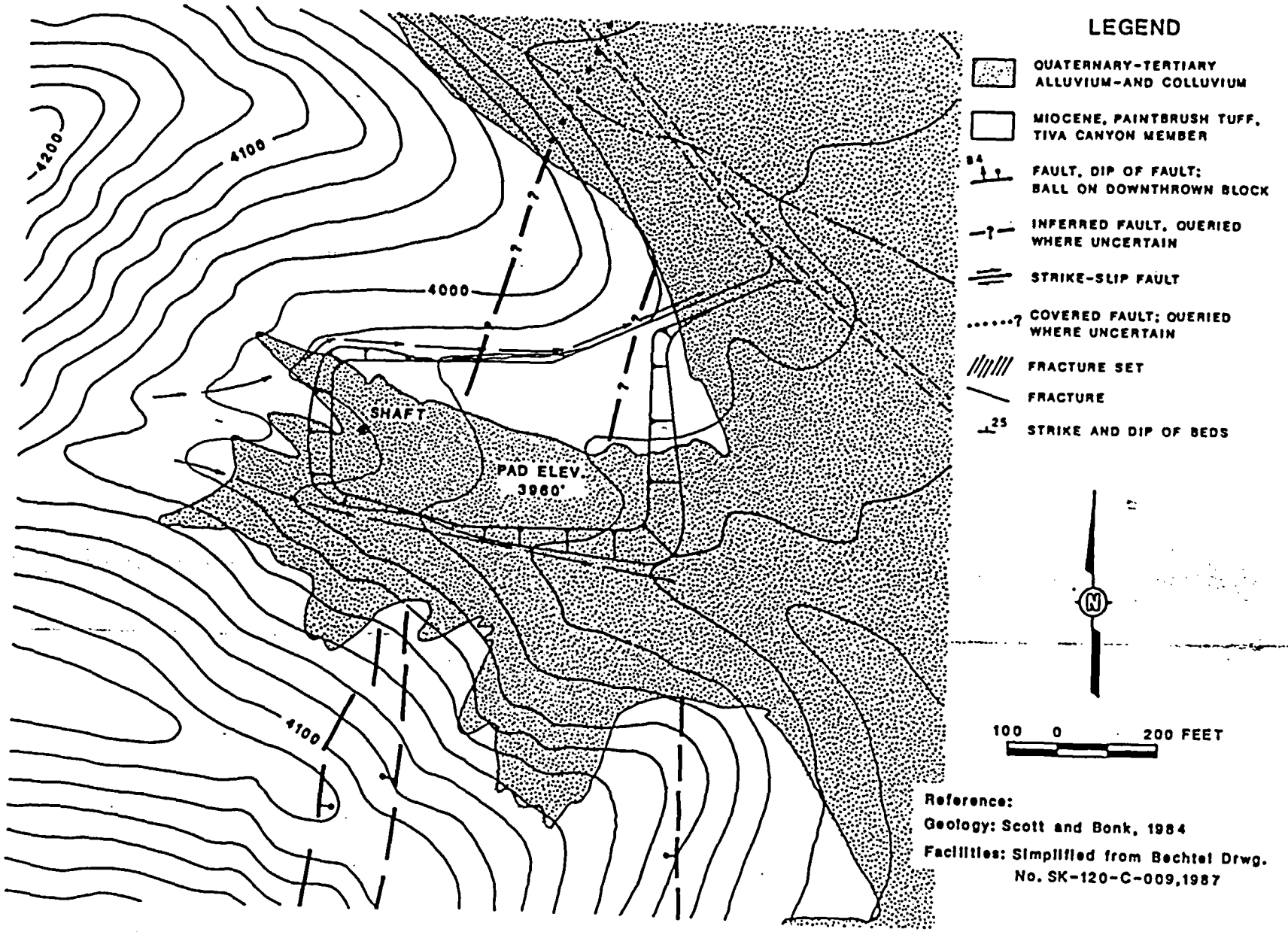

Figure A-1.5. Geology and Surface Facilities of Emplacement Exhaust Shaft and Environs

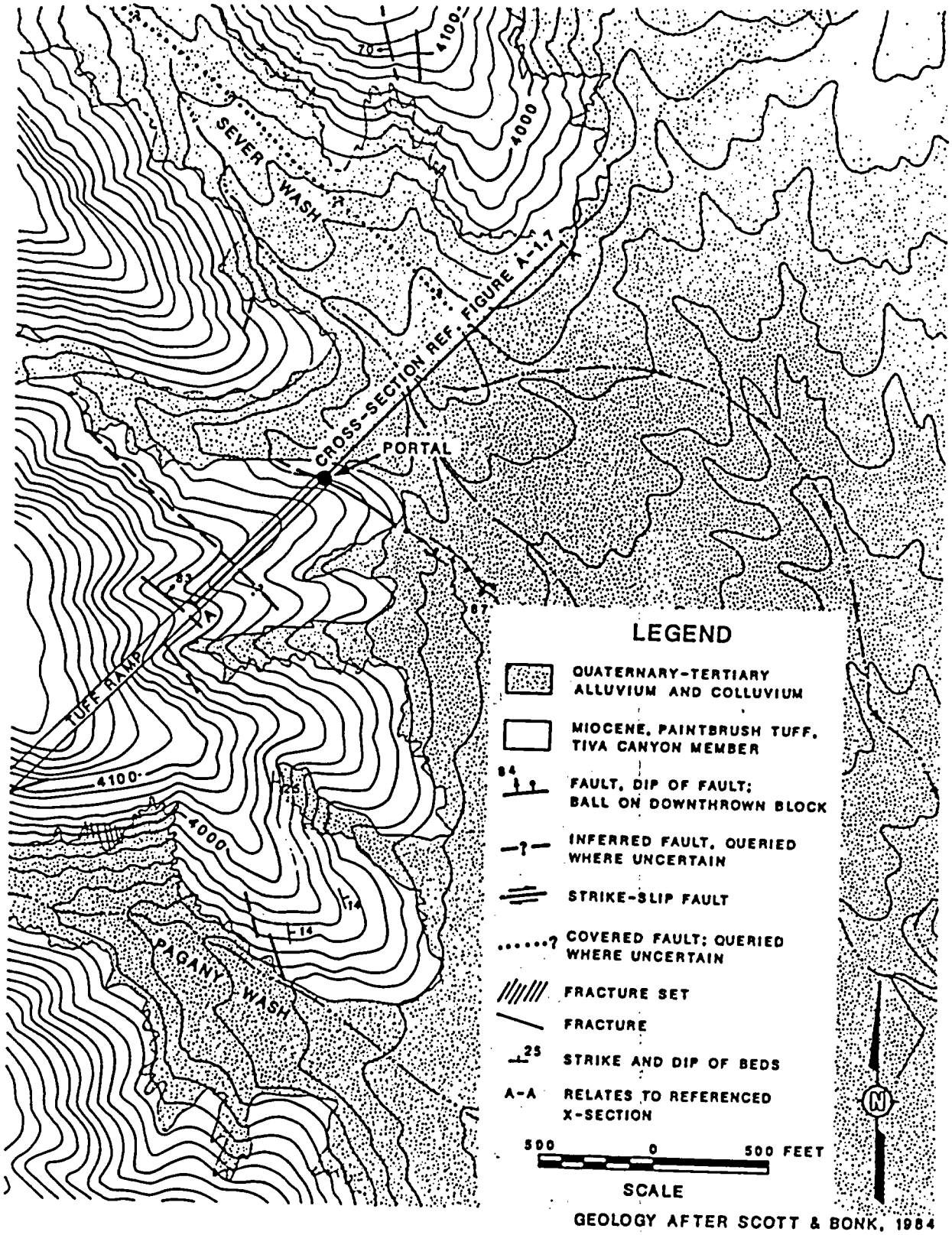

Figure A-1.6. Geology and Site Development of Tuff Ramp Portal

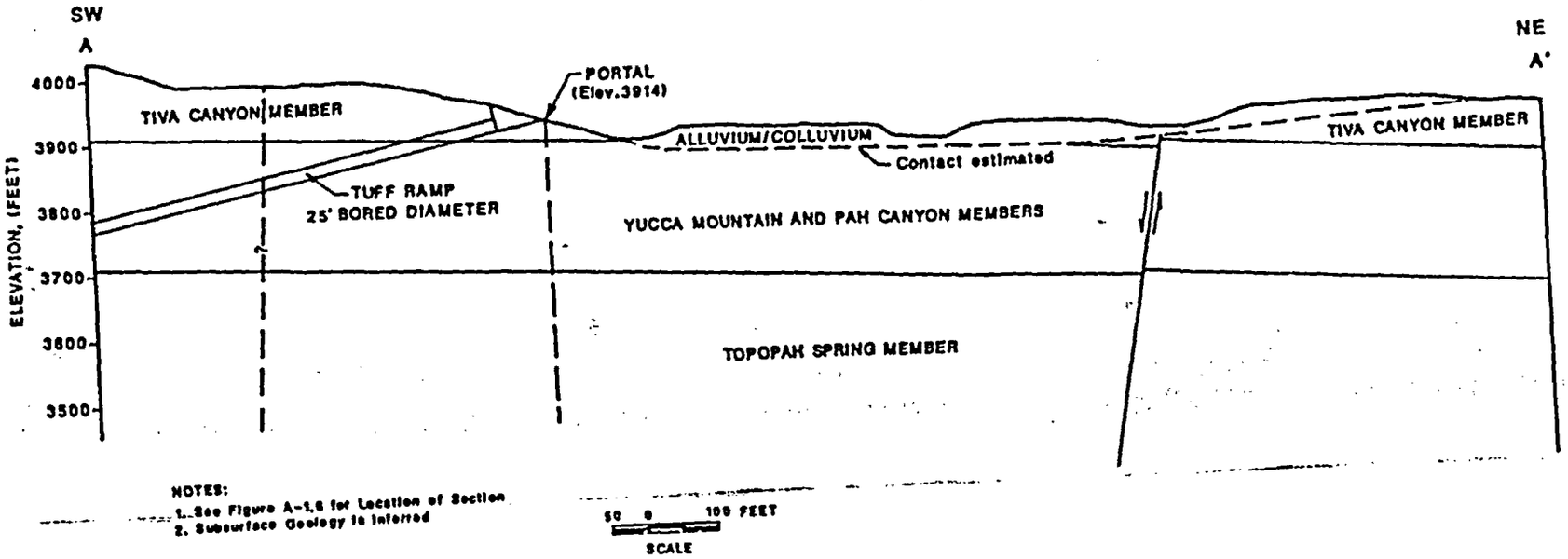

Figure A-1.7. Cross Section Parallel to Tuff Ramp East of Portal

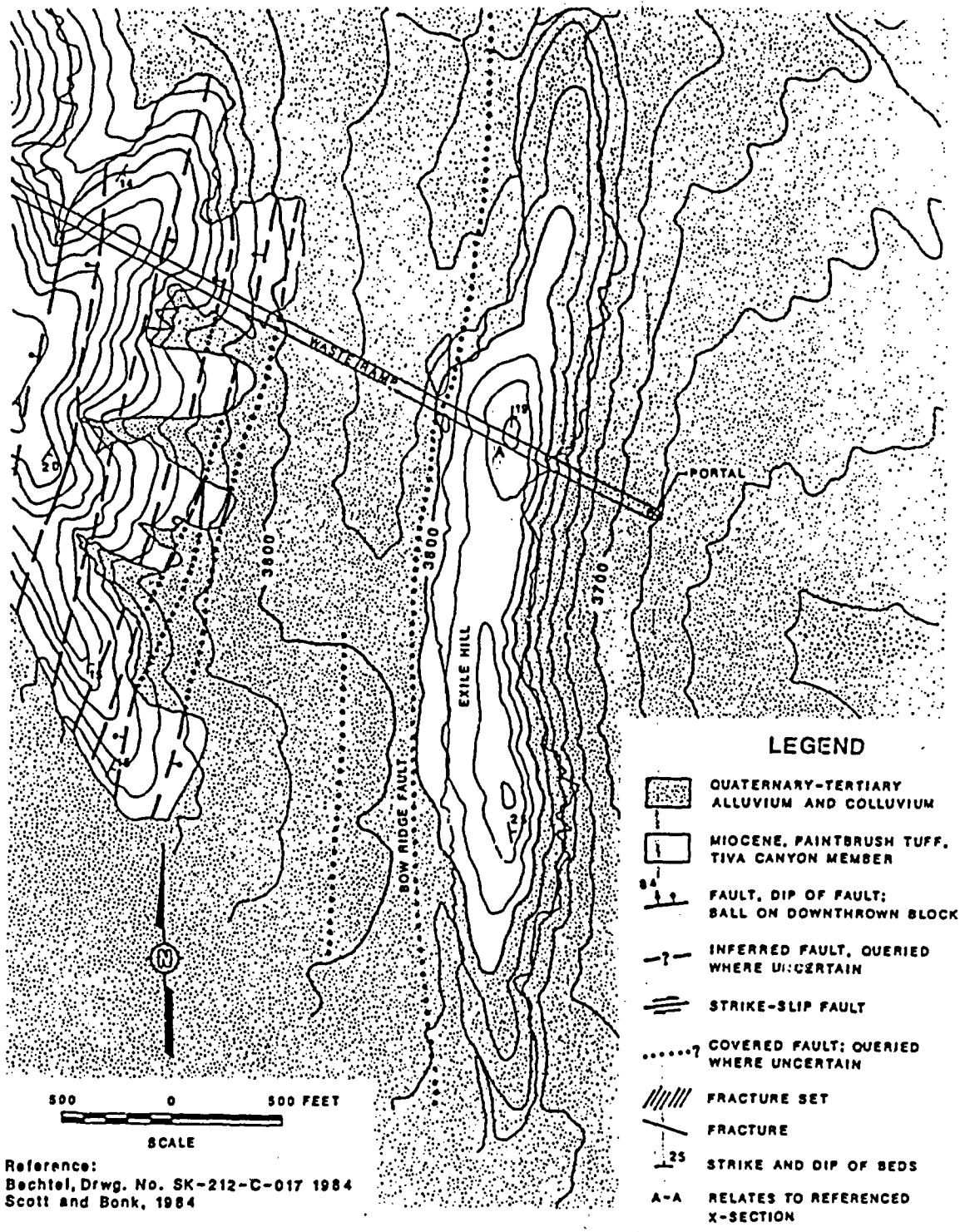

Figure A-1.8. Geology and Site Development of Waste Handling Ramp Portal

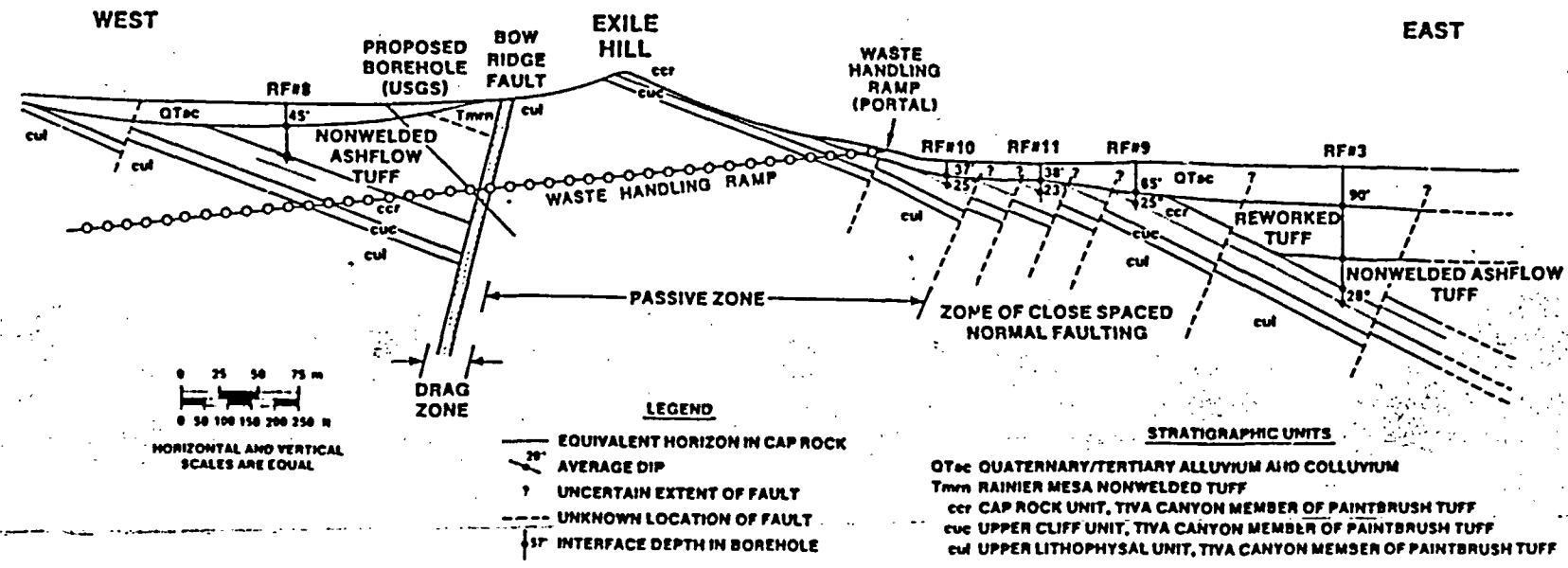

Figure A-1.9. Cross Section Along Waste Handling Ramp at Portal (After Neal, 1986)

A-1.2 FLASH FLOOD POTENTIAL

As described in the previous sections, the proposed locations for the shafts are within relatively steep washes. These washes are normally dry, but rapid runoff can occur during heavy rains. Figure A-1.10 shows the catchment area for each of the shaft and ramp locations. The flash flood potential is evaluated by several methods, as presented below.

Squires and Young (1984, p. 1) examined the flood potential in Drill Hole Wash and other major washes near Yucca Mountain. The following formulas were derived to calculate Q_{100} and Q_{500}, respectively, the peak discharges in cubic feet per second (ft^3/s) for floods with 100-year and 500-year recurrences:

$$Q_{100} = 482\ A^{0.565}$$

$$Q_{500} = 2200\ A^{0.571} \qquad \text{(A-1-1)}$$

where

A = area of the drainage basin in square miles.

Consider the example of ES-1 with a combined catchment area for the two forks of Coyote Wash. For a watershed area of 4.8×10^{-1} km^2 (0.19 mi^2), Q_{100} equals approximately 5.3 m^3/s (190 ft^3/s), and Q_{500} is about 24 m^3/s (850 ft^3/s) near the ES-1 location. Although these values may appear large, the U.S.G.S. (Squires and Young, 1984, p. 2) estimates that the peak discharges for Q_{100} and Q_{500} for Drill Hole Wash at its confluence with Forty Mile Wash are 65 and 280 m^3/s (2,300 and 10,000 ft^3/s), respectively. The regional maximum flood (i.e., peak flow rate) for Drill Hole Wash is estimated by the U.S.G.S. to be 2430 m^3/s (86,000 ft^3/s). Drill Hole Wash includes a drainage area of 39.9 km^2 (15.4 mi^2) and is considerably larger than the two forks of Coyote Wash.

The U.S.G.S. formulas indicate the peak flow of the flood in a wash but not the duration or the cumulative flow. Accordingly, hydrograph analyses were performed as part of the present study for the washes near selected shaft locations to determine peak and total flows (Table A-1.1) as well as the maximum and average depths of water in the washes during flooding. The hydrograph analyses were performed for the ESs, the MMS, and the EES. The ESs and MMS lie within washes that drain large areas and would appear to be more subject to flooding than the EES and ramp portals that lie within smaller drainage areas or well above adjacent washes.

A computer program, THYD, was used to generate synthetic, runoff hydrographs. The program is based on the Soil Conservation Service (SCS) triangular unit hydrograph method described in U.S.B.R. (1974, pp. 65-89) to generate a composite runoff hydrograph for humid environments. Input to the program consists of the watershed characteristics and rainfall data as shown in Table

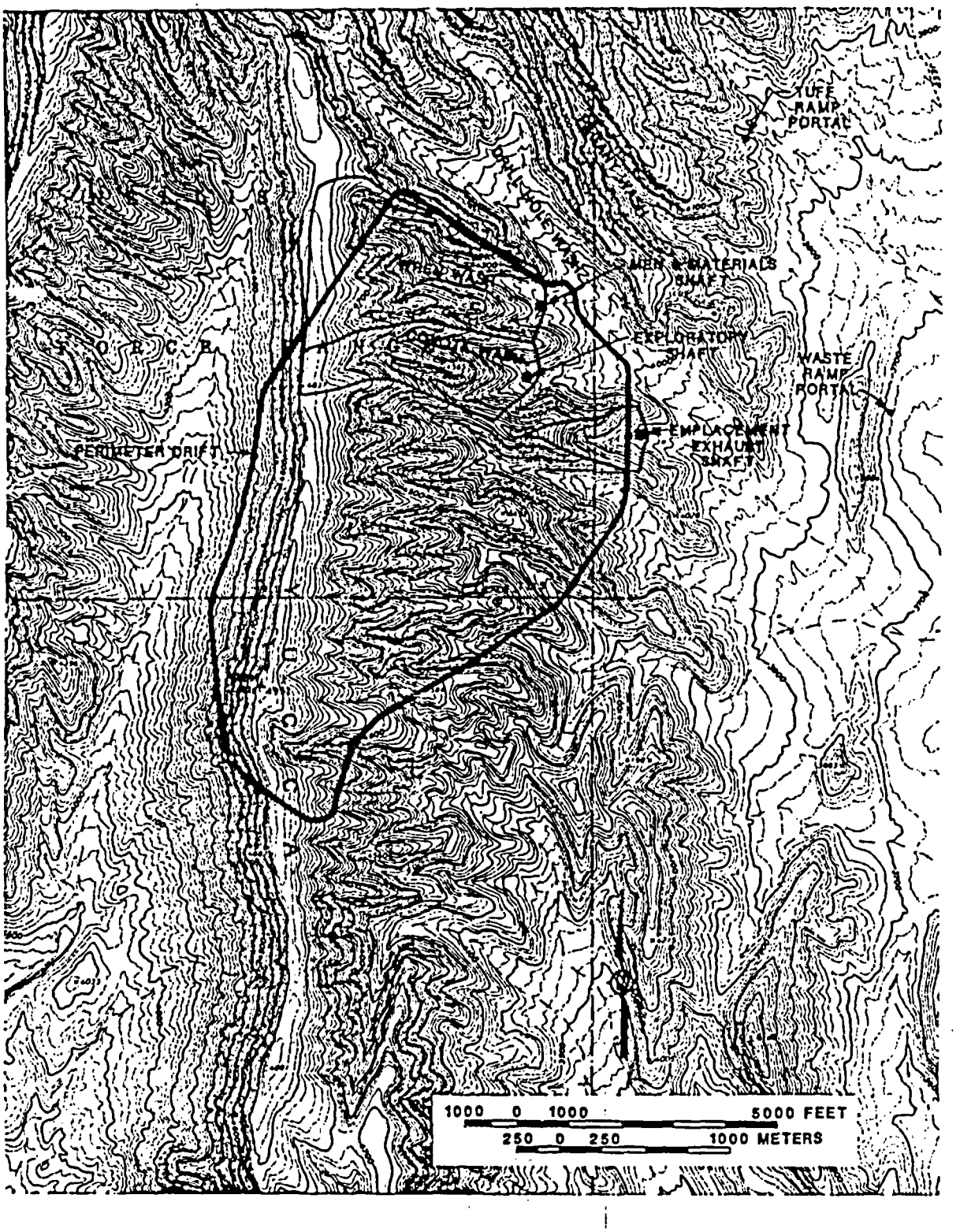

Figure A-1.10. Drainage Areas Above Shaft and Ramp Portal Locations

A-1.1. The analysis permits site-specific information such as hydrologic soil type, ground cover, and meteorological conditions to be taken into account. The present analyses were conducted for a storm that produces 18 cm (7 in.) of rain in 1 hour. This is the maximum expected rainfall in the southern Nevada area defined by the U.S. Bureau of Reclamation, (U.S.B.R., 1974, p. 53).

Figure A-1.11 shows the runoff hydrographs for the two forks of Coyote Wash. For the 1-hour event, the discharges rise sharply after about 0.5 hour to maximum values of 10 to 12 m^3/s (370 to 420 ft^3/s) and then decrease over the next 45 minutes to about 4 to 4.5 m^3/s (140 to 160 ft^3/s). The total runoff for the two washes combined is about 5.6×10^4 m^3 (45 acre-ft) for this storm event.

The runoff hydrographs for Wren Wash and its adjacent wash to the south near the MMS are similar to those discussed above. The maximum flow rates for a 1-hour, 18-cm (7-inch) event are 29 and 9 m^3/s (1,010 and 310 ft^3/s), respectively. The combined total runoff from the two washes is 9.9×10^4 m^3 (80 acre-ft).

The flood hydrographs that have been synthetically generated may be compared with a flood hydrograph for Eldorado Canyon (Glancy and Harmsen, 1975, p. 15). The actual hydrograph is illustrated in Figure A-1.12 and indicates a higher flow rate over a shorter duration than the synthetic hydrographs. This suggests that there may be difficulty in predicting peak flow rates and runoff by the SCS unitgraph method when small watershed areas are considered. For this reason, the Eldorado Canyon flood hydrograph was assumed to be typical of the expected flood hydrograph, and the relationship of surface water runoff to peak flow rate for this flood was used to calculate surface runoff from peak flow rates for the several shaft locations for comparison to other methods (Table 4-1).

Table A-1.1. Hydrograph Analyses at Various Shaft Locations

CATCHMENT	Coyote Wash (Exploratory Shaft)			Wren Wash (Men and Materials Shaft)			Drainage at Emplacement Exhaust Shaft		
	North Fork	South Fork	Combined Forks[a]	Wren Wash	Adjacent Wash to South	Combined Washes[a]	South Fork	North Fork	Combined Washes[a]
Area (ft^2)	2.4×10^6	2.8×10^6	5.2×10^6	6.7×10^6	2.0×10^6	8.7×10^6	1.4×10^6	5.7×10^5	1.9×10^6
Elevation change (ft)[b]	720	710	720	675	640		460	300	
Length (ft)	4500	4500	4500	4500	4000		2400	1300	
THYD Hydrograph[c]									
7-inch storm - 1 hr									
Peak flow (ft^3/s)	370	420	790	1010	310	1320	240	110	350
Mean flow (ft^3/s)	170	200	370	480	140	620	100	40	140
Total runoff (acre-ft)	20	25	45	60	20	80	13	5	18

[a] Sum of separate washes.
[b] Elevation difference in feet from highest point in the watershed to shaft location.
[c] The THYD hydrograph procedure was developed by IT Corporation and was based on U.S.B.R. (1974). In this analysis, the following assumptions were made: soil type has low run-off potential, probable maximum thunderstorm (1 hr) rainfall (PMTR) is 7 inches, the maximum area receiving the PMTR is 50 mi^2, and the ground cover is brush, sage, grass, or combination.

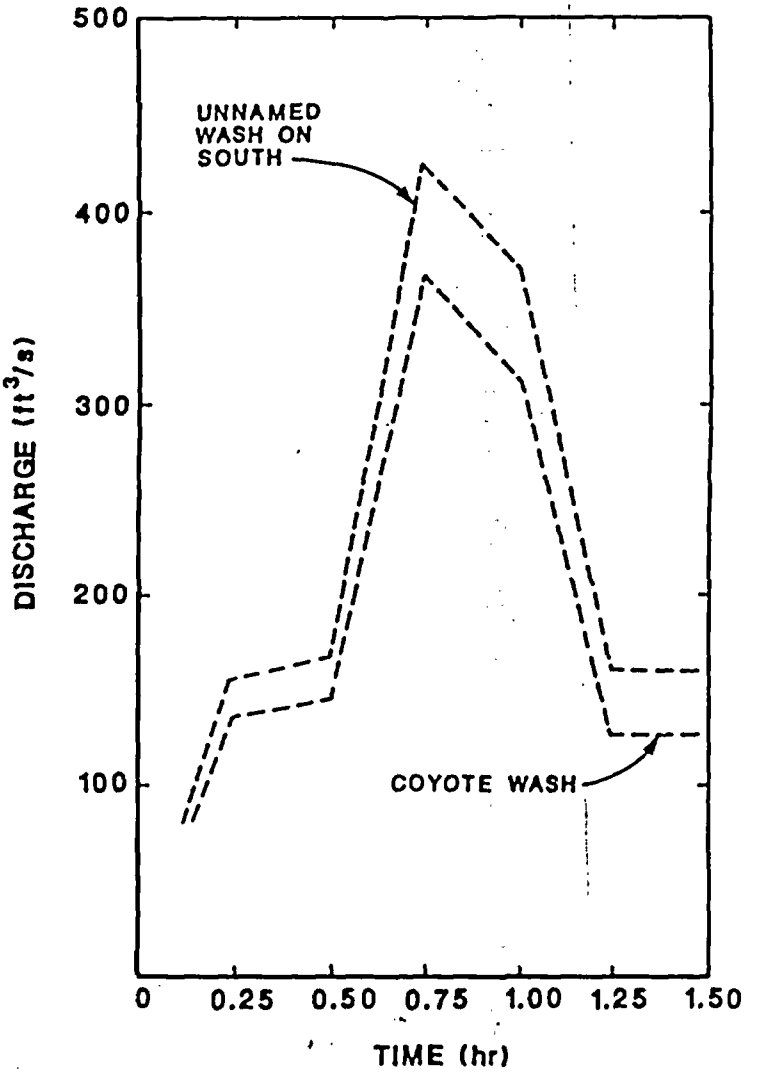

Figure A-1.11. Runoff Hydrograph for 1 Hour, 7-Inch Storm in Coyote Wash

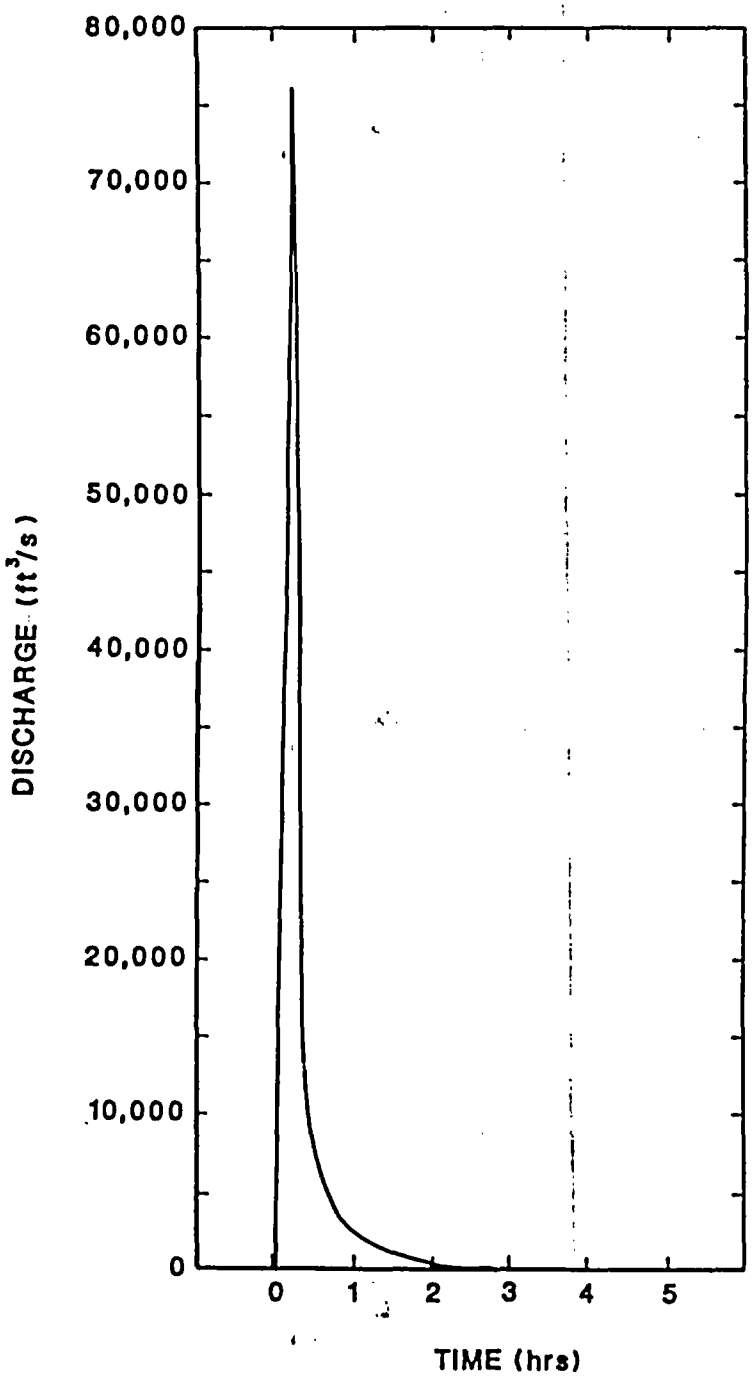

Figure A-1.12. Estimated Flood Hydrograph for September 14, 1974, Near Mouth of Eldorado Canyon (After Glancy and Harmsen, 1975)

APPENDIX A-2
GROUND-WATER INFLOW TO A SHAFT UNDER PERCHED WATER CONDITIONS

The expected hydrologic conditions at Yucca Mountain is that the average ground-water flux is less than 1 mm/yr (DOE, 1986, pp. 6-150 to 6-153). Under these conditions, the ground-water inflow to a shaft will be very small, as described by Fernandez and Freshley. Higher inflows to a shaft could occur if local saturation, i.e., perched water conditions, develops in the vicinity of a shaft. Theoretically, perching can occur where downward infiltration is impeded by permeability contrasts or capillary barriers.

A permeability contrast may occur where alluvium overlies Tiva Canyon bedrock in the washes. At the Exploratory Shaft-1 (ES-1) location, for example, the depth of the alluvium is about 9.1 m (Bentley, 1984, Table 1; see also Figure A-1.2). The hydraulic conductivity of the alluvium has not been measured, but a relatively low value for gravel is 10^{-2} cm/s (Freeze and Cherry, 1979, p. 29). In contrast, the bulk hydraulic conductivity of the Tiva Canyon is approximately 10^{-3} cm/s (Sinnock et al., 1984, p. 12).

A simple calculation may be performed to show that the alluvium below a wash could become saturated down to bedrock during runoff. The calculation uses the Green-Ampt solution (Section A-3) to calculate the infiltration rate into the alluvium and the depth of the wetting front against time. The required parameters are the height of ponding at the surface [taken as 0.39 m from a hydrograph for a 6-hour, 18-cm (7-inch) storm performed using the same method described in Section A-1.2], the initial pressure head -100 cm; Section A-3], and the porosity and hydraulic conductivity of the alluvium. The following table shows the depth of wetting against time for alluvium with a porosity of 0.3 and a saturated hydraulic conductivity of 10^{-2} cm/s.

Depth of Wetting (m)	Time (hr)
0.30	0.03
1.5	0.4
3.0	1.3
6.1	3.2
9.1	5.4

According to this analysis, water would penetrate to 9.1 m within a 6-hour storm, even with relatively low conductivity alluvium. With a more permeable alluvium (such as clean gravel with hydraulic conductivity of 10 to 100 cm/s), water would penetrate to bedrock within a few minutes and could then spread laterally across the bedrock surface toward the shaft. This analysis does not allow for caliche layers within the alluvium that might have low permeability and might slow infiltration.

A permeability contrast also exists where the fractured, welded Tiva Canyon overlies the nonwelded Pah Canyon Member of the Paintbrush tuff. From Sinnock

et al. (1984, p. 12) and Scott et al. (1983, p. 299), the saturated, bulk hydraulic conductivity of the Tiva Canyon is approximately 10^{-3} cm/s, whereas that of the Pah Canyon varies from 10^{-3} cm/s (vitric) to 10^{-6} cm/s (clayey). Following heavy rainfall, water could penetrate rapidly through fractures to the base of the Tiva Canyon, but it would be impeded from infiltrating into the Pah Canyon, especially if the underlying unit were clay rich (Scott et al., 1983, p. 325-327). Infiltration could also be impeded by air entrapment ahead of the wetting front (Montazer and Wilson, 1984, p. 47). According to either hypothesis, much of the water infiltrating from the surface could flow downdip (generally to the east) along the upper contact of the nonwelded Paintbrush Tuff.

APPENDIX A-3
INFILTRATION THROUGH SHAFT FILL

Two solutions are presented for downward infiltration through shaft fill. A steady-state solution is used to calculate flow rate based on the saturated hydraulic conductivity of the shaft fill. This method is useful for estimating the maximum flow rate, but it does not account for the time required to saturate the fill, which may be substantial if the hydraulic conductivity is low. Thus, a transient solution (Green and Ampt) is used to calculate the rate at which the wetting front moves. Both solutions assume pipe flow within a filled shaft; i.e., no flow is lost laterally into the walls of the shaft.

A-3.1 STEADY-STATE INFILTRATION

A possible approach to infiltration through shaft fill is to assume saturated conditions and steady-state infiltration rates. The maximum flow rate down the shaft under atmospheric conditions is obtained from Darcy's law where the hydraulic gradient is unity:

$$Q = K \cdot A \qquad \text{(A-3.1)}$$

where

Q = flow rate (m^3/s),
K = saturated hydraulic conductivity of the shaft fill (m/s), and
A = area of the shaft (m^2).

The calculated flows for a range of fill conductivity are shown in Table A-3.1.

Table A-3.1. Infiltration Through Shaft Fill

K, cm/s	Approximate Backfill Gradation[a]	Infiltration Rate,[b] m^3/s
10^2	coarse gravel/rock fill	10
10^{-1}	sand	10^{-2}
10^{-4}	silty or clayey sand	10^{-5}

[a] Appendix D.
[b] 3.7-m-diameter shaft.

A-3.2 GREEN AND AMPT TRANSIENT SOLUTION

Downward infiltration into an initially unsaturated fill will occur under the combined influence of suction and gravity gradients. As the water penetrates deeper and the wetted part of the profile lengthens, the average suction gradient decreases. This trend continues until eventually the suction gradient in the upper part of the profile becomes negligible, leaving the constant

gravitational gradient as the only remaining force moving water downward in the transmissive zone. Because the gravitational-head gradient has the value of unity, it follows that the flux tends to approach the hydraulic conductivity as a limiting value. In a uniform soil under prolonged ponding, the volumetric water content of the wetted zone approaches the saturated porosity.

A simple method for calculating vertical infiltration is the Green and Ampt solution (Hillel, 1971, pp. 140-143), which has been found to give good results for cases of infiltration into initially dry soils. This solution gives no information about details of the moisture profile during infiltration but does offer estimates of the infiltration rate and the cumulative infiltration as functions of time.

The principal assumptions of the Green and Ampt solution are that there is a distinct and precisely definable wetting front and that the matrix suction at this wetting front remains effectively constant regardless of time and position. Furthermore, this approach assumes that, behind the wetting front, the soil is uniformly wet and of constant conductivity. The wetting front is thus viewed as a plane separating a uniformly wetted zone from a totally dry zone.

With gravity taken into account (i.e., vertical infiltration) the Green and Ampt solution gives

$$\frac{Kt}{\Delta\theta} = L - (H_p - H_{p_o}) \ln \left(1 + \frac{L}{H_p - H_{p_o}}\right) \qquad \text{(A-3.2)}$$

where

K = saturated hydraulic conductivity of the fill (m/s),
t = time (s),
$\Delta\theta$ = $\theta_t - \theta_i$,
θ_t = transmissive-zone, volumetric water content,
θ_i = initial water content,
L = distance from the surface to the wetting front [i.e., length of the wetted zone (m)],
H_p = pressure head at the entry surface (m), and
H_{p_o} = initial pressure head at the wetting front (m).

Assumptions involved in this analysis include the following:

- The ponding depth is negligible.
- The surface is maintained at a pressure head of zero.
- Full saturation is assumed behind the wetting front.
- The initial water content (θ_i) equals zero.

- The effective pressure head (Hp_o) at the wetting front is equal to -100 cm H_2O (found by experiment to be typical for infiltration into initially dry soil; Hillel, 1971, p. 143).

Equation (A-3.2), after being rearranged, gives the time at which the wetting front reaches a given depth in the shaft

$$t = \frac{L - (H_p - H_{p_o}) \ln \left(1 + \frac{L}{H_p - H_{p_o}}\right)}{K} \Delta\theta \qquad \text{(A-3.3)}$$

The time taken for the wetting front to reach a given depth is therefore inversely proportional to the saturated hydraulic conductivity of the fill. For a fill with a saturated hydraulic conductivity of 10^2 cm/s and a porosity of 0.3, the time required for the wetting front to penetrate to 300 m is about 2 minutes (Figure A-3.1). For a fill with conductivity 10^{-4} cm/sec, the time required to penetrate 300 m is about 1,000 days, i.e., proportionally six orders of magnitude greater.

The average rate of advance of the wetting front (L_R) over a given time increment is given by

$$L_R = \frac{\Delta L}{\Delta t}. \qquad \text{(A-3.4)}$$

The infiltration rate (I) is thus given by

$$I = \Delta\theta L_R = \Delta\theta \frac{\Delta L}{\Delta t}. \qquad \text{(A-3.5)}$$

From these relations it can be shown that the infiltration rate approaches the steady-state rate, which is equal to the saturated hydraulic conductivity for a unit gradient. For a fill with a saturated hydraulic conductivity of 10^{-4} cm/s the infiltration rate approaches the steady-state rate after about 70 days (Figure A-3.2), which is only 10% of the time required for the wetting front to penetrate 300 m. Initially, the fill will accept a higher flow rate than that predicted by the steady-state approach. Over the first hour, the average infiltration rate is 1.2×10^{-3} cm/s, or 12 times the steady-state rate. Over the first day, the average flow rate slows to 5×10^{-4} cm/s, or five times the steady-state rate.

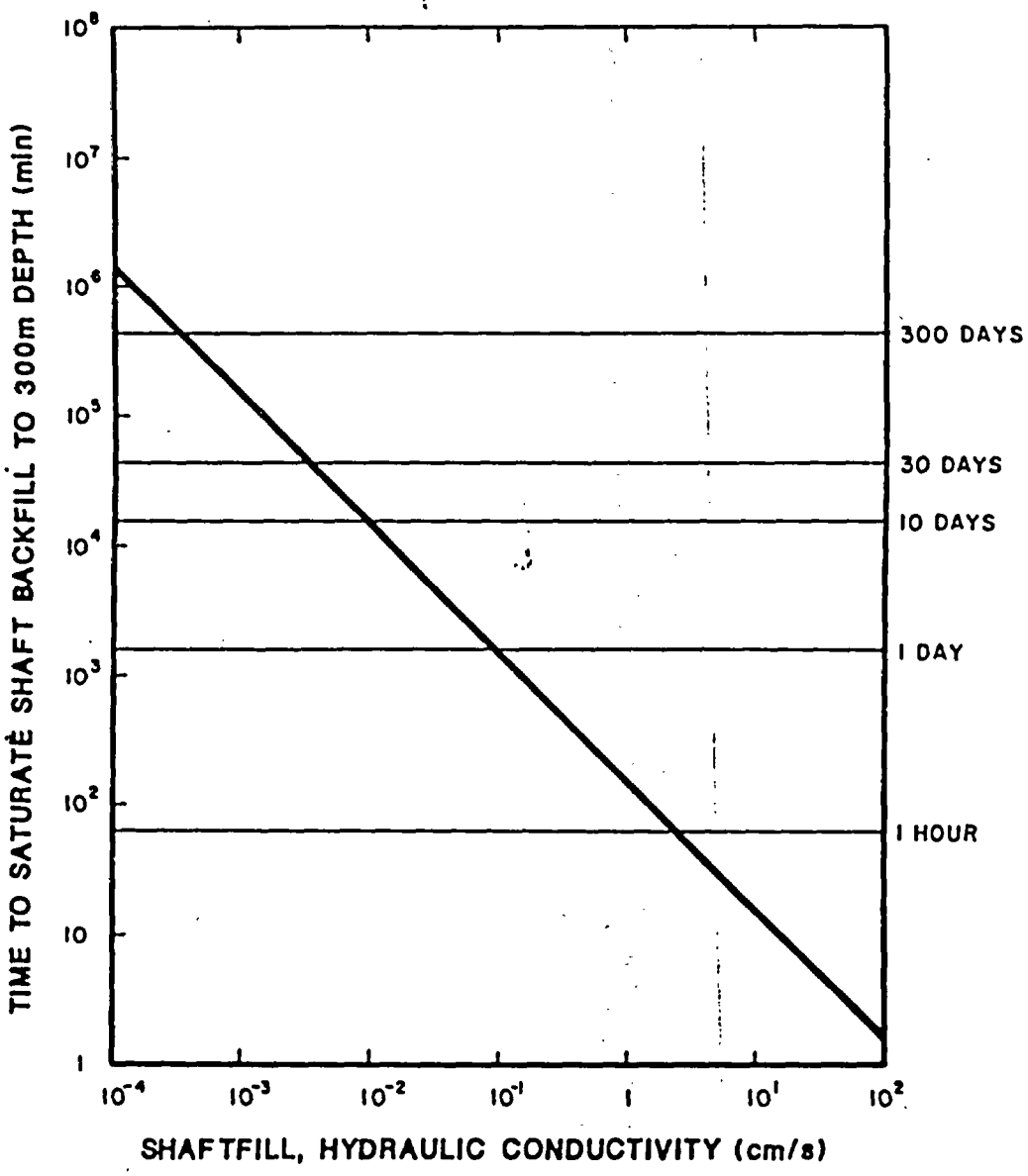

Figure A-3.1. Results From Green and Ampt Solution for Transient Flow Through Shaft Fill

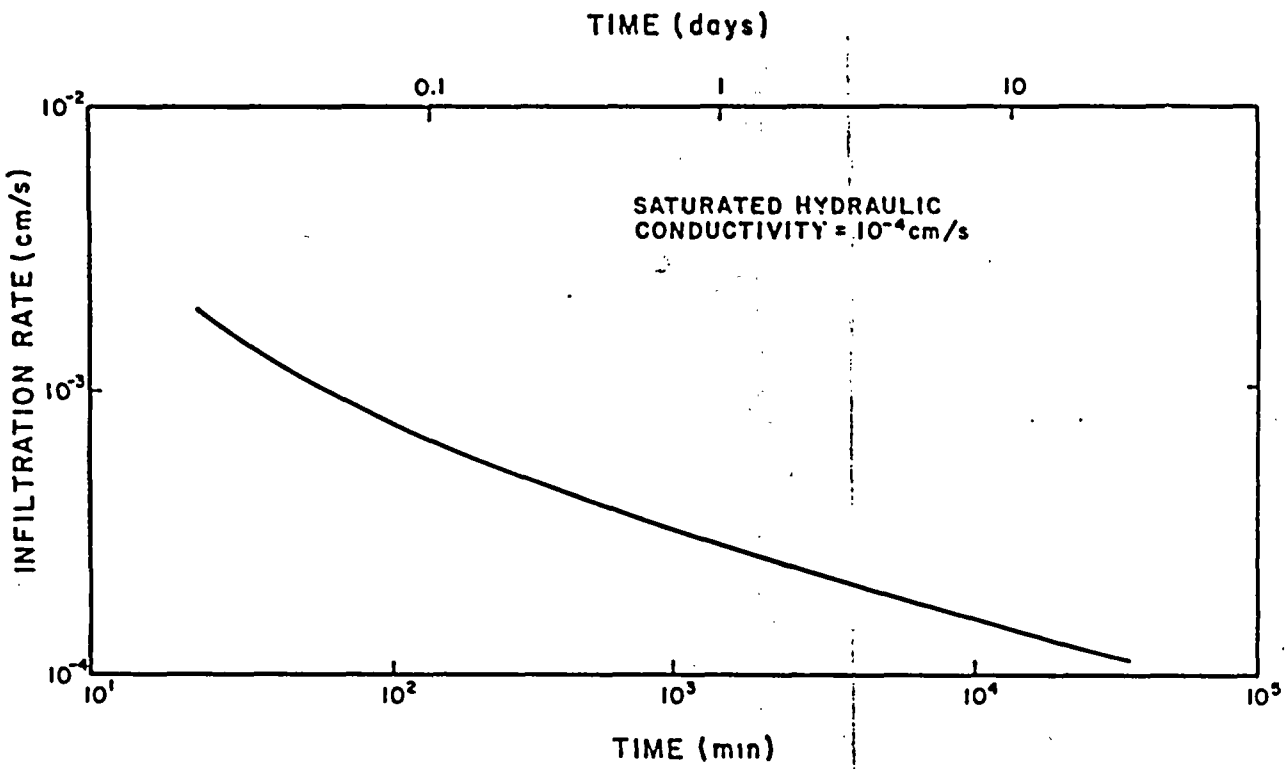

Figure A-3.2. Infiltration Rate Through Shaft Fill as a Function of Time

A-3.3 DISCUSSION

The calculations in the previous section show that an assumption of steady-state conditions is reasonable for a fill with high hydraulic conductivity (e.g., 10^{-2} to 10 cm/s or higher). For such a material, both the time required for the wetting front to travel down the shaft and the difference between the transient- and steady-state-infiltration rates can be ignored for practical purposes.

The transient condition cannot be ignored for shaft fills with low hydraulic conductivity (e.g., 10^{-2} to 10^{-3} cm/s or lower). For example, consider a scenario where water is impounded above the shaft and remains for a nominal 30 days before draining or evaporating away. Considering a fill with a saturated hydraulic conductivity of 10^{-4} cm/s, the wetting front would penetrate only about 11 m into the shaft. The volume of water that would enter the shaft would be about 34 m^3, assuming a fill porosity of 30%. With a conductivity of 10^{-2} cm/s, the wetting front would penetrate to the bottom of the shaft in about 10 days (Figure A-3.1).

The calculations presented above consider flow in a pipe that has impermeable sides. This is approximately the case for a shaft lined with steel or good quality concrete when flow through the host rock around the shaft is ignored.

APPENDIX A-4
ALTERNATE SOLUTION FOR INFLOW TO A SHAFT

The alternate solution assumes the alluvium is instantaneously saturated by the flood event with water perched on the Tiva Canyon-alluvium contact. The desaturation of the alluvium is assumed to occur under unconfined conditions. The vertical infiltration through the Tiva Canyon bedrock is assumed equal to the saturated hydraulic conductivity of the bedrock times the cross-sectional area in plan out to the radius of influence, but excluding the area occupied by the shaft and modified permeability zone (MPZ). This assumption is conservative for purposes of estimating shaft flow because the initial gradients for vertical infiltration may be much greater than one.

The general assumptions given above are identical to the assumptions made using the computer program, TOTAL.* Two significant differences in the alternate analysis are the computation of H_o, the hydraulic potential at the shaft wall, in the radial formula for unconfined flow (A-4.1) and the introduction of a "capture zone."

The radial flow toward the shaft is assumed to occur under unconfined conditions under the Dupuit assumption; i.e., flow occurs horizontally and is uniform in vertical section. The equation of flow is given by Todd (1980, p. 118):

$$Q_s = \frac{\pi K \, (H^2 - H_o^{\,2})}{\ln \left(\frac{r}{r_i}\right)} \qquad \text{(A-4.1)}$$

where

Q_s = flow rate toward the shaft,
H = hydraulic potential at the radius of influence,
H_o = hydraulic potential at the radius of the shaft,
r = radius of influence,
r_i = radius of the shaft, and
K = hydraulic conductivity of the alluvium.

Water that flows radially towards the shaft then flows vertically downward through the fill or plug and a surrounding MPZ. The top of the fill or plug coincides with the contact zone between the Tiva Canyon and the alluvium. Darcy's Law may be written for vertical flow down a shaft:

$$Q_s = K_e \bullet A \bullet i = (K_p A_p + K_{mp} A_{mp}) \left(\frac{H_o + L}{L}\right) \qquad \text{(A-4.2)}$$

*TOTAL is a FORTRAN listing used to facilitate the numerical calculations used to describe the flow into the upper portion of a shaft in Section 4.1.3.

where

K_e = effective hydraulic conductivity of the shaft fill and MPZ,
A = total area,
i = vertical hydraulic gradient,
L = length of the plug or fill,
K_{mp} = conductivity of the MPZ,
A_{mp} = area of the MPZ,
K_p = conductivity of the fill, and
A_p = area of the fill.

In the case of a filled shaft, the length of fill (L) may be much greater than the height of perched water (H_o) such that the vertical hydraulic gradient [$(H_o + L)/L$] is approximately one and the shaft fill is draining under approximately unconfined conditions.

According to the flow-continuity principle, the radial flow toward the shaft (A-4.1) must equal the vertical flow down the shaft (A-4.2) or

$$\frac{\pi K(H^2 - H_o^2)}{\ln(\frac{r}{r_i})} = (K_p A_p + K_{mp} A_{mp}) \left(\frac{H_o + L}{L}\right). \qquad \text{(A-4.3)}$$

Therefore, the following relationship is used to solve for H_o:

$$\frac{\pi K}{\ln(\frac{r}{r_i})} H_o^2 + \left(\frac{K_p A_p + K_{mp} A_{mp}}{L}\right) H_o + \left(\frac{K_p A_p + K_{mp} A_{mp}}{1}\right) - \frac{H^2 \pi K}{\ln(\frac{r}{r_i})} = 0 \qquad \text{(A-4.4)}$$

In the solution of the above equation, it is possible that the fill and surrounding MPZ could have a vertical drainage capacity that exceeds the radial flow under all circumstances of perched water ($0 \le H_o \le H$). In this case, it is assumed that the hydraulic potential over the shaft H_o is set to zero.

The second major difference between the two analyses is the incorporation of the "capture zone" (Figure A-4.1). All water flowing down the wash that lies within the capture zone will eventually flow down the shaft. In this zone, the radial flow velocity induced by the drawdown of the water surface near the shaft is sufficiently strong to overcome the tendency for flow to occur laterally down the alluvium in the wash. Outside this zone, the converse is true. According to Keely and Tsang (1983, p. 703), the width of the capture zone is 2π times the stagnation radius (r_s). The stagnation radius represents the point where the alluvial flow velocity "V_d" down the wash is equal to the radial flow velocity "V_s" or

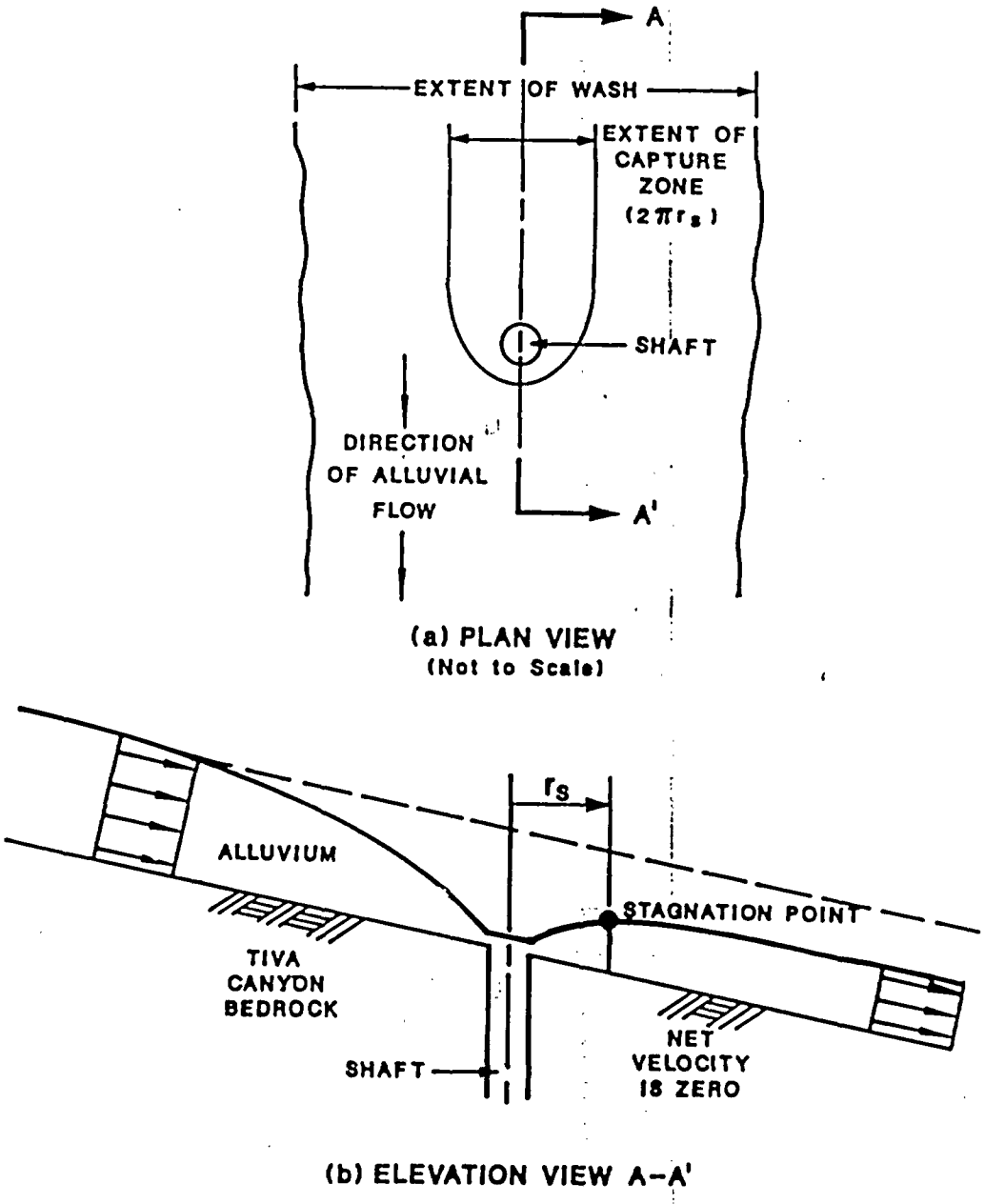

Figure A-4.1. Capture Zone Near a Shaft

$$V_d = V_s \quad \text{and}$$

$$K \bullet i = \frac{Q_s}{2\pi r_s H} \qquad \text{(A-4.5)}$$

where

Q_s = radial flow towards the shaft (defined below),
V_d = Darcy velocity in the alluvium,
V_s = shaft velocity,
r_s = stagnation radius,
K = hydraulic conductivity of the alluvium, and
i = hydraulic gradient assumed equal to the topographic gradient of the wash.

The term "Q_s" is the radial flow toward the shaft and is assumed to occur under unconfined conditions under the Dupuit assumption of Equation A-4.1; i.e., flow occurs horizontally and is uniform in vertical section.

The area dominated by alluvial flow can be computed from the radius (r_s). At this stage the alluvial, Tiva Canyon, shaft, and MPZ flows are computed for a small time step. The change in storage in the alluvium from the three components of flow over the time step is then computed. At the end of the time step, the drawdown or desaturation of the alluvium corresponding to the change in storage is calculated.

A comparison of the results for the Exploratory Shaft-1 (ES-1) obtained using this approach and the approach described in Section 4.1.5 (TOTAL approach) is illustrated in Figure A-4.2 for the 500-year flood volume and for Tiva Canyon hydraulic conductivities of 10^{-5} and 10^{-2} cm/s. A small difference of about 7 m^3 is observed on the left portion of both sets of curves. This is attributed to the assumption in the alternate calculation that a small wedge of water contained within the alluvium drains immediately into the ES-1. At the peak for the 10^{-5} cm/s Tiva Canyon curve, there is little difference between the two calculations. At high-alluvial conductivities, the results indicate that, for both the 10^{-5} or 10^{-2} cm/s Tiva Canyon curves, the TOTAL calculation provides a conservative estimate (slightly higher) of the volume of water entering the shaft. This is attributed to differences between the two approaches in computing alluvial flow; i.e., the capture zone approach results in a smaller amount of water flowing towards the shaft. Considering the general nature of the alternate solution, the results are in good agreement.

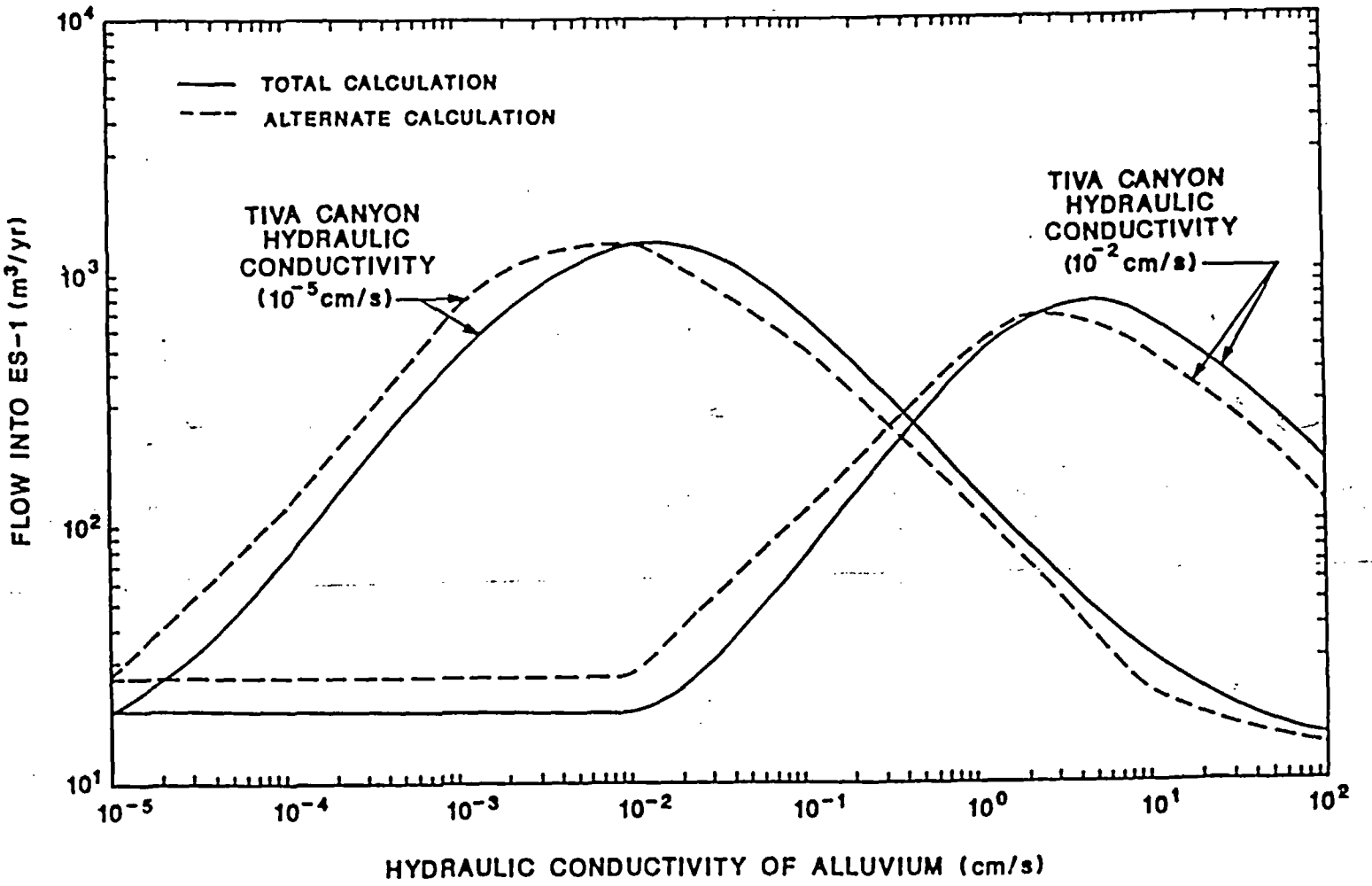

Figure A-4.2. Comparison of Shaft Inflow Calculations

APPENDIX A-5
DRAINAGE THROUGH THE SHAFTS AND DRIFT FLOORS

A-5.1 DRAINAGE FROM A SHAFT

Drainage to the surrounding fractured tuff after standing water develops in either the shaft or sump is evaluated using formulas developed for calculation of hydraulic conductivity in cased or open boreholes above the water table. Stephens and Neuman (1982a) give two, steady-state solutions for drainage from a cased borehole and three solutions for an open hole. All solutions are for boreholes above a deep water table. The solutions are derived using free-surface theory where the flow region is completely saturated and the free surface is a boundary across which no flow occurs (Figure A-5.1). Solutions derived for cased boreholes may be used for drainage from a lined shaft if the concrete shaft liner is assumed to be impermeable. In this case, flow occurs only through the base of the shaft. Solutions derived for open boreholes may be used for the case where the liner has been totally or partly removed. In this case, flow occurs through the walls of the shaft as well as through the base.

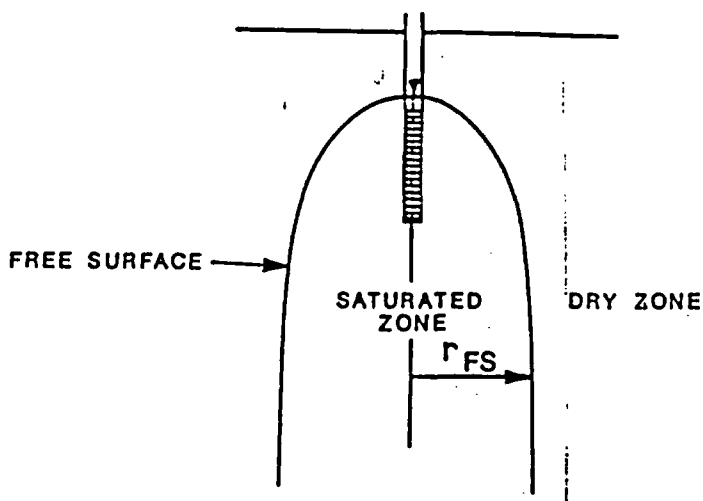

Figure A-5.1. Classical Free-Surface Concept of Flow From a Borehole Above a Deep Water Table (Stephens and Neuman, 1982a)

A-5.1.1 Drainage from a Fully Lined Shaft or Sump

Flow from the base of a lined shaft was calculated by using a formula developed by the U.S. Bureau of Reclamation (Stephens and Neuman, 1982a, p. 625) for flow from the bottom of a cased borehole:

$$Q = 5.5\ r\ H\ K \qquad \text{(A-5.1)}$$

where

Q = flow rate,
r = radius of shaft,
H = height of standing water in the shaft (total head), and
K = saturated, effective hydraulic conductivity of fractured tuff.

The author of this solution (hence called the "U.S.B.R. solution") and the details of its derivation are unknown (Stephens and Neuman, 1982a); however, some empirical observations regarding its use have been made by the U.S.B.R.:

- The thickness of the layer being tested should be equal to at least 20 times the radius (r) of the borehole.
- The base of the borehole should be located at least a distance of 10 times r above the water table.
- The base of the borehole should be located at least a distance of 10 times r below the top of the layer being tested.

Also, by implication, the base of the unit tested should be below the water table.

The MMS and EES will bottom out in the Topopah Spring, about 90 m and 80 m above the base of the units (Figure 2-10). The water table is below the base of the Topopah Spring in both cases. For these shafts, the U.S.B.R. solution appears to be valid, and it is appropriate to use the hydraulic conductivity of the Topopah Spring over a range of 10^{-2} to 10^{-5} cm/s (Section 2.2.2).

The ES-1, as proposed, will bottom out in the Calico Hills about 25 m below the base of the Topopah Spring and about 110 m above the water table. The U.S.B.R. solution also appears valid for this shaft, but it is appropriate to use a "weighted average" hydraulic conductivity to account for the lower conductivity at the base of the shaft.

Drainage from a shaft calculated using the U.S.B.R. formula is shown in Figure A-5.2 for a host rock, hydraulic conductivity of 5×10^{-6} cm/s. This is approximately a lower bound for the saturated hydraulic conductivity of both the vitric Calico Hills and Topopah Spring units. For a low head (1 m), the drainage from a 3.6-m-diameter shaft is about 10^{-7} m^3/s. For a head of 30 m, the drainage rate increases to about 10^{-5} m^3/s.

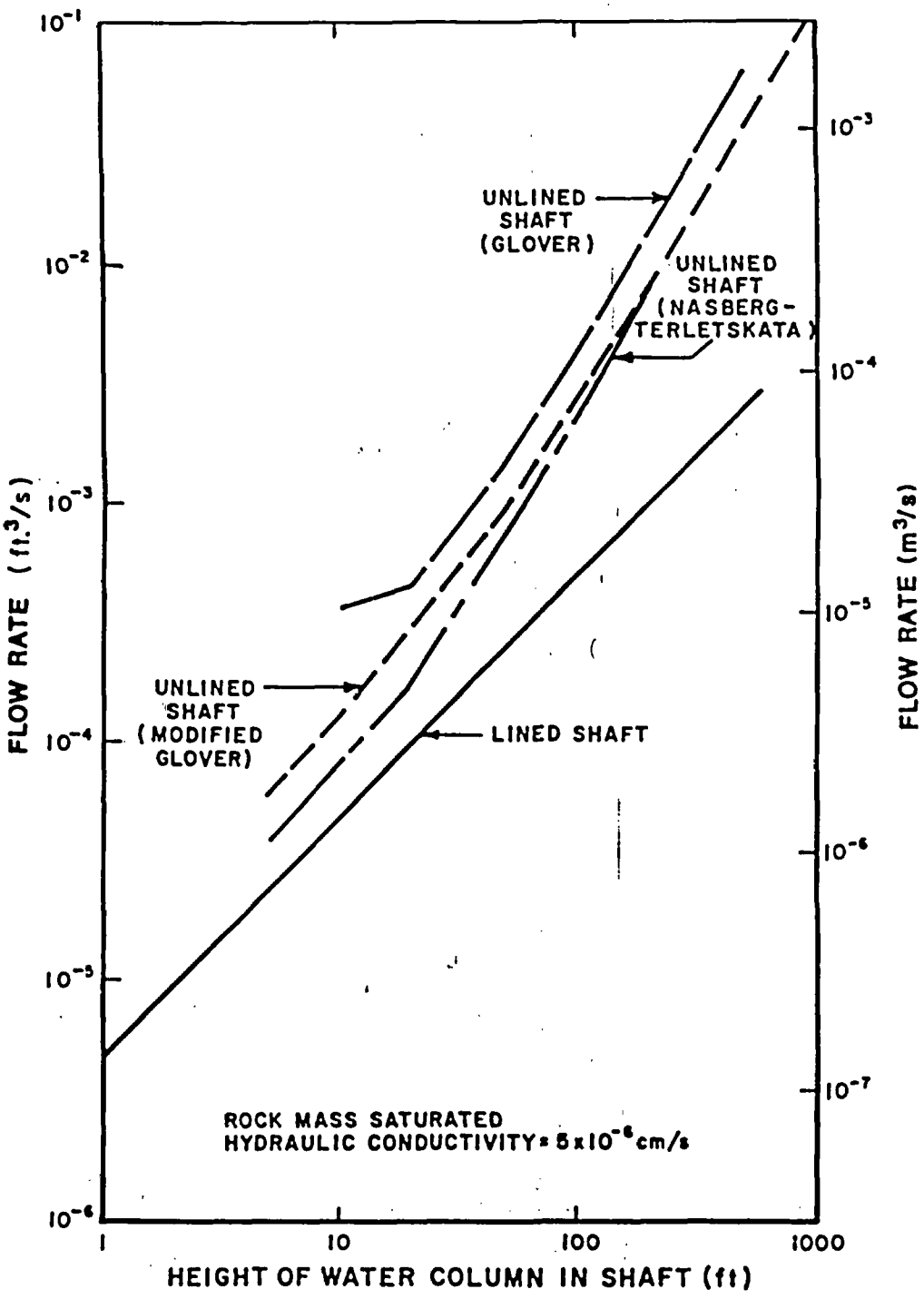

Figure A-5.2. Comparison of Solutions for Drainage From a Lined or Unlined Shaft

The second solution given in Stephens and Neuman (1982a) for drainage from a cased borehole was developed by Cecen. This solution involves use of a flow net and a scaling factor, and it is not thought to be appropriate for drainage from a shaft in fractured rock.

A-5.1.2 Drainage from an Unlined Shaft or Sump

Three solutions for infiltration from an open borehole are discussed by Stephens and Neuman (1982a, p. 634); those of Cornwall, Glover, and Nasberg-Terletskata. In addition, a modified Glover (half-source) solution is discussed by Reynolds et al. (1983, p. 257). Cornwall's solution was developed for packer tests below the water table and is not appropriate for application to shaft drainage calculations.

The solution developed by Glover is

$$Q = K\, C_u\, r\, H \tag{A-5.2}$$

where

$$C_u = \frac{2\pi(H/r)}{\sinh^{-1}(H/r) - 1} \tag{A-5.3}$$

and all terms are as defined previously.

The modified Glover or half-source solution is given by

$$Q = \frac{K\, 2\pi H^2}{c} \tag{A-5.4}$$

where

$$c = 4\left[\frac{1}{2}\sinh^{-1}\left(\frac{H}{2r}\right) - \sqrt{\frac{r^2}{H} + \frac{1}{4}} + \frac{r}{H}\right]. \tag{A-5.5}$$

This solution corresponds to a line source extending from the bottom of the well to halfway to the water surface. This solution does not include the effects of gravity drainage. The proportion of the total flow due to gravity drops off rapidly as the H/r ratio increases because an increasing amount of outflow is through the sides of the borehole. For example, H/r = 10 yields a gravity flow component of only 1.5% of total outflow (Reynolds et al., 1983, p. 259). In the present analysis the range for (H/r) is $0.8 \le H/r \le 200$. Thus, for most shaft/sump drainage analyses, the effects of gravity flow are insignificant.

The assumptions associated with both the Glover and the modified-Glover solution are as follows:

1. The borehole can be replaced by a line source whose strength is zero at the top and varies linearly with depth.

2. The cylindrical geometry of a borehole is approximated by prescribing constant head along the radius at the bottom of the borehole.

Glover determined that the solution is reasonably accurate for $H/r \geq 10$. Reynolds et al. studied the modified solution for $5 \leq H/r \leq 10$, stating only that the modified version is superior within this range.

The Nasberg-Terletskata equation for drainage from an unlined shaft (Stephens and Neuman, 1982a, p. 625) is

$$Q = \frac{KH^2}{0.423} \frac{1}{\log(2H/r)} \qquad \text{(A-5.6)}$$

where symbols are defined previously. This equation was derived assuming the following:

- The borehole can be replaced by a line source.
- The source strength is constant.
- Flow rate is uniform along the borehole wall.
- A prescribed head is imposed at the bottom of the borehole along its radius to approximate the borehole's cylindrical geometry.
- The medium is homogeneous and isotropic.

Stephens and Neuman (1982b) indicate that the Nasberg-Terletskata solution is best suited for $50 \leq H/r \leq 200$ probably because it explicitly takes into account the existence of a free surface. The Nasberg-Terletskata solution does not account for a nonuniform rate of flow out of the borehole.

A concept presented by Fernandez and Freshley (1984, p. 35) is that the shaft sump should act as a drain such that none of the surface water or ground water entering the shaft would flow into the repository through the shaft station. For purposes of calculating shaft drainage, the maximum value for H used in both the Glover and Nasberg-Terletskata solutions is thus taken to be the

depth of the shaft sump below the repository level. Values of H/r calculated for each of the shafts as currently designed are shown below.

Shaft	H (m)	r (m)	H/r
Exploratory Shaft (ES-1)	140	2.2	64
Men and Materials	24	3.4	7.1
Emplacement Exhaust	3.0	3.4	0.9

H = approximate depth of sump below repository level at shaft station (Figure 2-10)
r = shaft radius

For the ES-1, since $H/r \geq 50$ (approximately), both the Glover and Nasberg-Terletskata solutions are valid. For both the MMS and EES, the planned sumps are relatively shallow, but the drainage may be estimated from Figure A-5.2. As noted by Stephens and Neuman (1982b), there is no preference as to which solution to use as H/r approaches unity. This is because the drainage from the base of the shaft dominates the total discharge.

A-5.1.3 Discussion

The analytical solutions used in the previous two sections involve a number of assumptions and simplifications. An important assumption is that the rock is isotropic and homogeneous and can be modeled as a continuum with an equivalent, porous-medium, hydraulic conductivity. This assumption is reasonable for the nonwelded Calico Hills, but it may be questionable for the fractured Topopah Spring tuff. On examination, however, the assumption that the Topopah Spring acts as a continuum may be reasonable (as a first approximation), given that the typical fracture density (40 fractures/m^3; Sinnock et al., 1984, pp. 11-12) is high relative to the size and zone of influence of the shafts. Moreover, the fractures are dominantly vertical, which should enhance drainage both through the base of the shaft and through the walls. A related assumption concerns the equivalent, porous-medium, hydraulic conductivity for the fractured Topopah Spring tuff. Scott et al. (1983, p. 299) quote a range of 10^{-2} to 10^{-5} cm/s.

Other simplifications or assumptions are implicit in the analytical solutions. These include geometrical factors, the assumption of steady-state flow rather than transient flow, and the assumption that the region outside the free-surface boundary is dry, both initially and as flow is occurring. In reality, the Topopah Spring is partially saturated with saturations as high as 91% (Scott et al., 1983, p. 325). Also, a saturated-unsaturated analysis by Stephens and Neuman (1982b) indicated that a significant portion of the total flow may take place outside the region defined by the classical free-surface envelope.

A-5.2 DRAINAGE THROUGH THE DRIFT AND RAMP FLOORS

Drainage through the floor is estimated from Darcy's law using an equivalent, porous-medium, saturated hydraulic conductivity to account for fractures. Because the Darcy approach is valid only for saturated conditions, it is assumed that the conductive fractures are saturated quickly. A unit gradient is assumed for one-dimensional downward flow. Drainage is calculated by:

$$Q = KA \tag{A-5.7}$$

where

Q = flow rate (m^3/s),
K = saturated hydraulic conductivity (m/s), and
A = area (m^2).

Figure A-5.3 shows the calculated drainage rate as a function of floor area and hydraulic conductivity over a range 10^{-2} to 10^{-6} cm/s. In practice, the permeability of the floor could be reduced by fines generated during construction, although these could be removed in appropriate areas at decommissioning. Figure A-5.3 also shows the drainage capacity of a single disposal room (408 m x 4.88 m) as a function of the hydraulic conductivity of the floor. From Section A-5.1, the inflow from a fault could be as high as 10^{-2} m^3/s, assuming convergent flow and a hydraulic conductivity of 10^{-2} cm/s. In order for this inflow to drain, the hydraulic conductivity of the floor would have to be in the range 10^{-3} to 10^{-4} cm/s. However, it should be noted that an inflow of 10^{-2} m^3/s, on a continuous basis, is improbable.

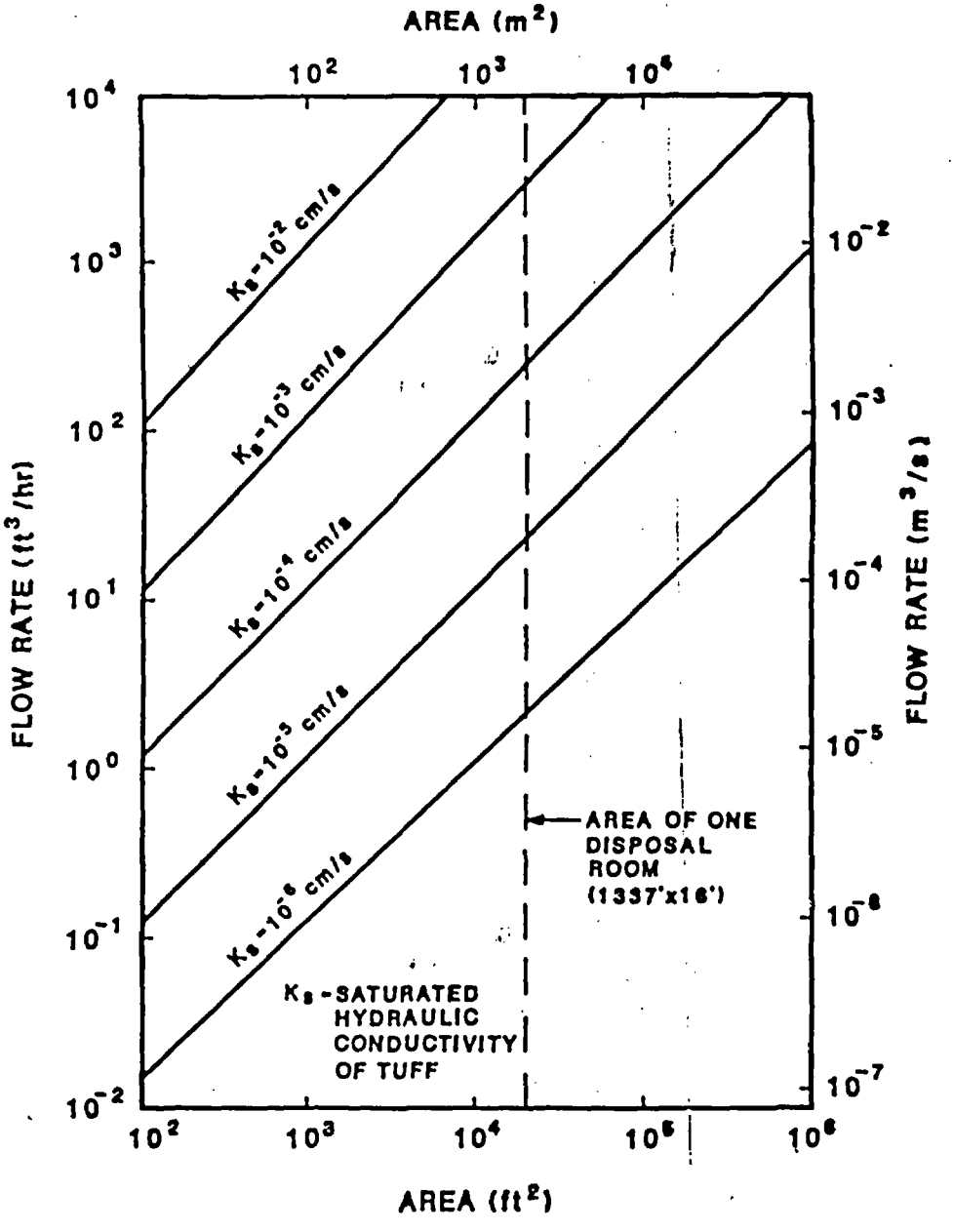

Figure A-5.3. Drainage as a Function of Floor Area and Hydraulic Conductivity

APPENDIX B
SPARTAN RADIONUCLIDE RELEASE ANALYSIS

To illustrate how the hydrologic performance goals can be computed, assuming all radionuclide releases are controlled by matrix dissolution, the following example is used to to compute the hydrologic performance goals. A total quantity of water 'q' is needed to compute the dissolution of uranium with time. For the purpose of this analysis, a total amount of water 'q' of 16.5 m^3/yr per 70,000 MTHM is assumed.

Using this value of 'q' and a solubility limit of the uranium matrix of 5 μg/ml (5×10^{-3} kg/m^3) the annual, mass release of specific radionuclides '$D_j(t)$' can be computed. These values are then compared to the appropriate NRC limits for each radionuclide. This comparison is made in Figure B-1 for the predominant radionuclides of concern. The ordinate axis on Figure B-1 is represented mathematically by

$$\frac{D_j(t)}{\text{NRC limit}_j} = \frac{(q \cdot S_u)\,[m_j(t)/m_u(t)]}{\text{NRC limit}_j} \qquad \text{(B-1)}$$

where the terms are the same as presented in Equation 3-5. The NRC limit as used in SPARTAN is the amount in kilograms of the 'jth' radionuclide that can be released from 70,000 MTHM. A ratio less than one represents compliance with the NRC release limits for the 'jth' radionuclide. In Figure B-1, all of the radionuclides from 300 years to 10^6 years are in compliance with the NRC release limits for the case analyzed. Radionuclides Pu-238 and Am-241 dominate the releases during the period of 300 to 1,000 years. Cs-137 and Sr-90 have comparatively high initial releases as compared to the other radionuclides but their releases drop off rapidly because of their short half-lives. Radionuclides Pu-242, Zr-93, and Tc-99 dominate the releases out to 10,000 years after closure. Other radionuclide releases increase after the 300 years following emplacement of the waste canisters. This is due to the buildup of daughter radionuclides by radionuclide decay.

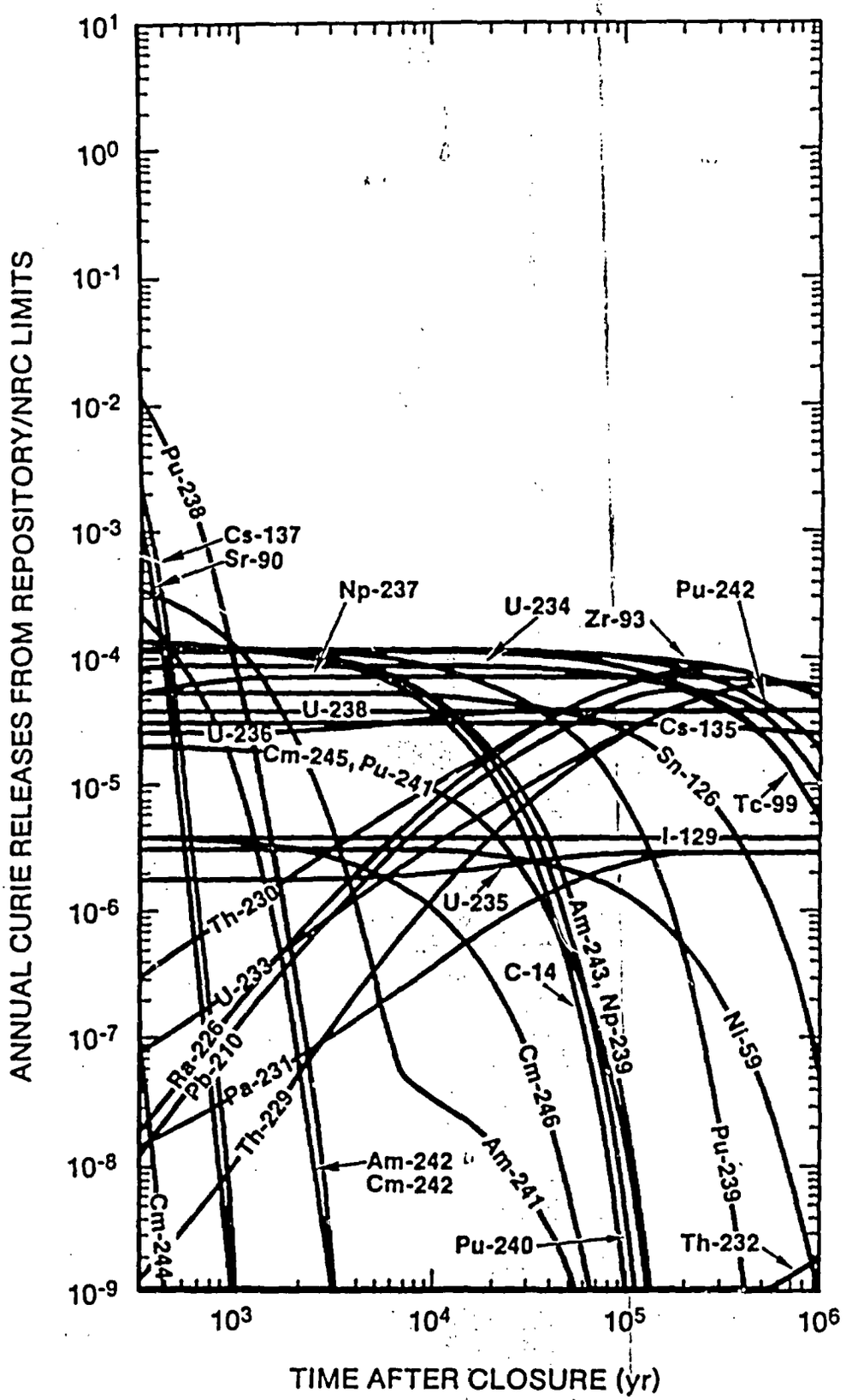

Figure B-1. Radionuclide Releases in Comparison to the NRC Limits at the Edge of the Waste Package (Using the SPARTAN Code and Assuming a Total Flow (Q) of 16.5 m^3/yr/70,000 MTHM)

APPENDIX C
AIR FLOW OUT OF THE REPOSITORY

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APPENDIX C
AIR FLOW OUT OF THE REPOSITORY

For a repository located above the water table, there is the possibility of release of radionuclides by air flow out of the repository through the shafts or through the host rock. Air flow may be enhanced by heating, which provides a mechanism for convective flow and also dries out the rock around the repository, increasing the effective air conductivity. The objectives of this appendix are to evaluate the potential magnitude of air flow from the repository and to compare the relative influence of the shafts, ramps, and host rock in allowing air flow. More specifically, the calculations examine the influence of the modified permeability zone (MPZ) around the shafts and ramps, and the degree to which flow can be limited by filling or sealing the shafts.

C-1 FLOW MECHANISMS

Two mechanisms for air flow out of the repository are considered. Both involve convective, air flow. Mechanism A (Figure C-1) assumes that little or no flow occurs through the host rock relative to flow through the shafts, ramps, and drifts. The Exploratory Shaft-1 and -2 (ES-1 and ES-2) are within the repository area and the temperature is above ambient. The Men and Materials Shaft (MMS) and the Emplacement Exhaust Shaft (EES) and the ramps are located outside or just inside the repository perimeter, and the temperature is close to ambient. In response to the temperature gradients, air will tend to rise in ES-1 and ES-2 and air will be drawn in through the other entries. This mechanism may occur if the shafts and drifts are open or if the shaft fill and drift backfill are relatively permeable so that the resistance to air flow through the fill and backfill is less than that through the rock. Mechanism B (Figure C-1) examines whether significant flow can occur through the host rock. The waste disposal areas are relatively hot, and the heated air tends to rise vertically through the rock as well as through the ES-1 and ES-2. Air is drawn in through the peripheral entries maintaining pressure in the rooms.

C-2 METHOD OF ANALYSIS

For purposes of a simplified analysis, the mechanisms of convective, air flow through a heated repository are considered to be analogous to the problem of air flow through an underground mine resulting from natural ventilation. Air flows induced by thermal convection are calculated using a network resistance model by a method similar to that used in mine-ventilation studies (Hartman, 1982, pp. 239-245). Flow is calculated using Darcy's law, in which the major input parameters are the resistance to air flow of the underground openings and the host rock, and the pressure gradient calculated from the difference in pressures between the inlet and outlet points.

The method of calculating flow through a heated repository based on methods normally used for mine ventilation studies is an approximation of a complex

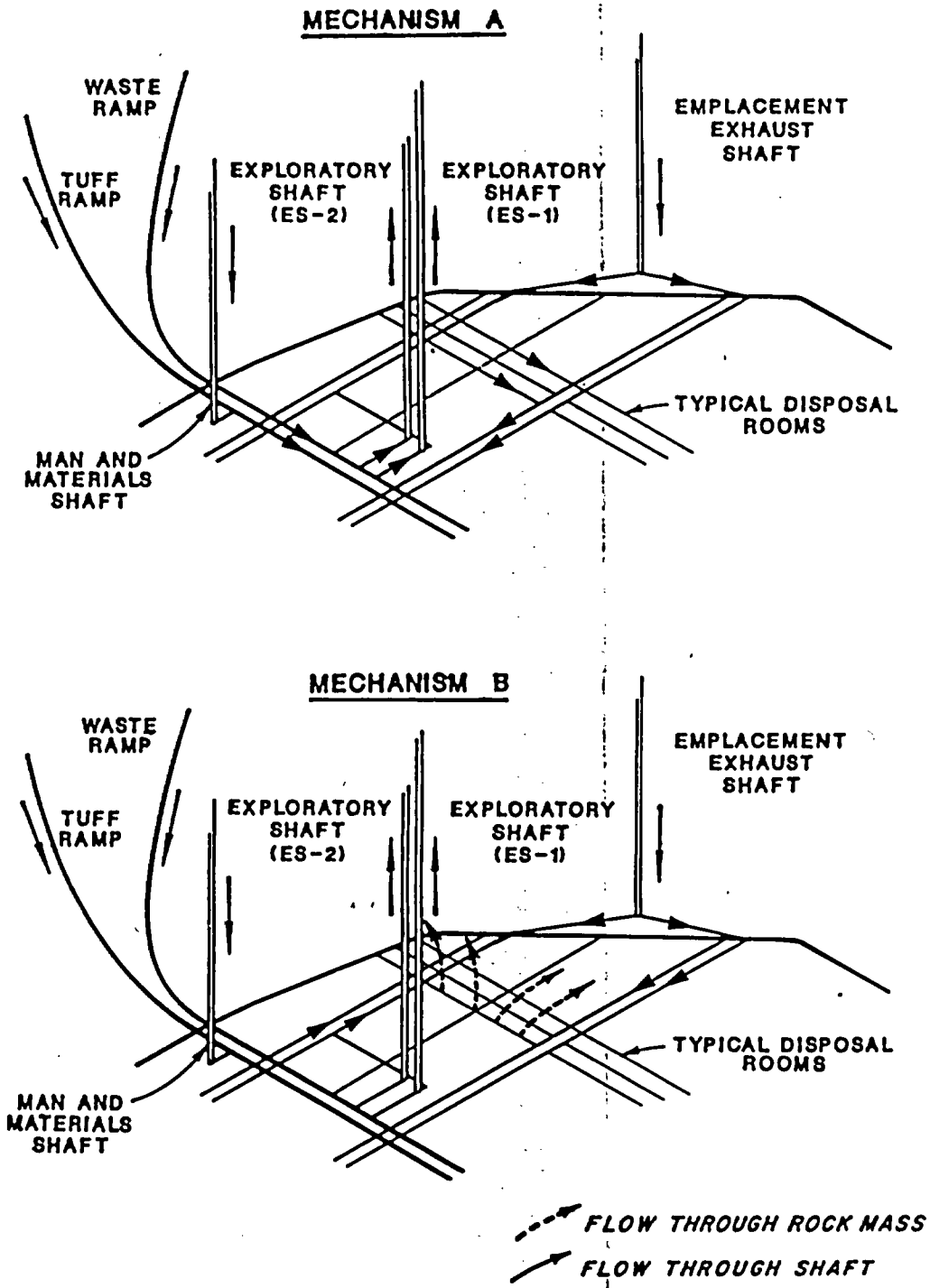


Figure C-1. Mechanisms for Convective, Air Flow

problem in thermodynamics. Simplifications or assumptions involved in the analyses are given below.

1. Darcy's law is valid

   The resistance to air flow through open or backfilled drifts may be characterized as either laminar or turbulent. In turbulent flow, resistance is nonlinearly related to potential. In laminar flow, resistance is linearly related to potential, and flow may be calculated using Darcy's law.

   The results of the analyses were used to check the validity of Darcy's law by calculating the Reynolds number from the air velocity or specific discharge, air kinematic viscosity, and the characteristic dimension. In the case of laminar flow through a straight, open drift, the characteristic dimension is the tunnel diameter, and the calculated Reynolds number should be less than 2,000 (Daugherty and Franzini, 1965, p. 193). In the case of laminar flow through backfill, the characteristic dimension is the mean grain diameter, and Darcy's law is valid as long as the Reynolds number does not exceed a value between 1 and 10 (Freeze and Cherry, 1979, p. 73). In both cases, the calculated Reynolds numbers were within the specified limits and the assumption of head loss varying linearly with flow rate was found to be justified.

2. Air temperatures in the shaft are the same as in the adjacent rock

   Convective, air flow through a heated repository will involve a complex coupling of heat transfer from the rock to the air, which will tend to drive air flow, and cooling of the rock by passage of the air, which will tend to reduce the driving mechanism. In the modeling which follows, the effects of cooling of the rock are ignored. The air is assumed to be at the same temperature as the adjacent rock at all points in the repository, including the shafts.

   Intuitively, this simplified approach is most valid for the case of a backfilled repository in which air flows relatively slowly and temperatures are able to equilibrate. The faster the air flows, the greater the volume of air moving through the repository, and it is more likely that the rock will be cooled to the extent that convection slows down. A converse effect of rapid

air flow could occur if the air flow is not sufficient to cool the rock in the repository significantly. Flow through the repository would be greater than that calculated using our simplified approach if air in the exit shafts (or rock) is not cooled by heat transfer to the rock. As described in Section C-3, the driving pressure could then be about three times higher than that calculated with the assumption of equilibrated temperatures. This higher driving pressure occurs, however, because air is expelled at the ground surface at the same temperature as the temperature of the repository rooms, a condition which is intuitively overconservative.

On balance, the approach adopted in these calculations is considered to be valid for scoping studies, particularly when (as noted below) input parameters for temperatures and air resistances are not well known.

3. Air flow is incompressible and the air is dry

Since convective transport evolves from air buoyancy effects dependent on temperatures, thermal properties such as air density and air viscosity will change through the circuit. In reality, flow is compressible with the actual resistance to mass flow rate dependent on density and viscosity. In the analyses presented in this report, air compressibility effects on fluid flow are ignored for reasons of simplification. This assumption is considered to be reasonable given that the pressures involved are small (i.e., <0.1 psi, Section C-3). According to Hartman (1982, p. 160), compressibility effects may be ignored for mine static head pressure drops of less than 5 kPa (0.72 psi) or where differences in elevation are less than 430 m.

Convective transport can, in general, involve both the transport of air and water vapor. The development of high temperatures at the repository horizon will result in drying and lowering of moisture content of the rock. It is thus assumed that the air may be dry at the time at which peak temperatures are reached. This assumption is conservative because the effect of adding moisture to the convective flow will be to increase the work required to lift the air to the surface and to thus reduce flow rates.

4. Air circulation occurs along specified paths
The model assumes that a particular path for air circulation (Mechanism A or B; Figure C-1) is established and that flow is one-dimensional through either shaft or ramp fill, open drifts, or through damaged or undamaged tuff. The model ignores the development of secondary circulation currents that might develop in the host rock above or below the repository away from the waste containers.

C-3 MODEL DESCRIPTION

The analytical method used to solve for air flow involves assembling a "network stiffness matrix" (Zienkiewicz, 1977, p. 12-13) of various resistances representing the network of underground openings and (as appropriate) the rock mass. The pressure boundary conditions are applied and a system of linear, simultaneous equations is set up to calculate the nodal pressures, which in turn are used to calculate air flows through the network. The following sections describe the temperature and pressure boundary conditions, air conductivities (material properties), and model geometry (networks) used in the analyses.

C-3.1 Temperature and Pressure Distributions

The draft pressure was calculated for both mechanisms using the accepted mine ventilation practice of computing pressure gradients on the basis of differences in air density at the inlet and outlet points. A separate calculation based on thermodynamic considerations was performed to provide a more conservative upper bound to the calculation of draft pressure.

The first step required in the analysis is to determine the temperature profiles at the potential repository inlet and outlet points. At present, no analyses have been conducted to determine specific temperature histories at the shaft and ramp locations, and temperature profiles at certain times cannot be obtained. For purposes of the present analyses, a peak temperature profile was estimated for the ES-1 based on a peak temperature of 115°C at the repository horizon. At the time at which this peak temperature is attained, the temperature at the other entries outside the repository will be considerably lower. For a conservative analysis (i.e., with a maximum-temperature difference), the temperature profile at the inlet shafts and ramps was constructed from the natural, geothermal gradient, assuming a temperature of 13°C at the ground surface.

The draft pressure was calculated by integrating air density over depth for the ES-1 and the other shaft profiles to determine two pressures at the repository horizon and then taking the difference in these two pressures. The calculated, draft pressure (i.e., pressure difference) was 0.350 kPa (5.1×10^{-2} psi), which corresponds to 1.4 inches of water gage. By comparison, according

to Hartman (1982, p. 240), the natural ventilation pressure generated by natural geothermal energy in mines is usually less than 0.5 inches water gage and seldom exceeds 3 inches except in extreme cases. The calculated draft pressure falls within this range for both mechanisms and would be expected to be higher than 0.5 inches, since the generation of heat in an underground, nuclear-waste repository results in larger temperature contrasts than those experienced in a typical underground mine.

To provide a bounding calculation for the ventilation pressure, the air circulation was also modeled as a simple joule cycle (Hall, 1981, p. 110-112). This cycle assumes adiabatic compression for air flow down the inlet shafts or ramps, heating of the air to the repository temperature, and adiabatic expansion up the outlet shaft. The net ventilation pressure was calculated to be approximately three to four times higher than the previous calculation. This occurs because heat is not transferred from either the upcast or downcast shaft, and air is expelled at the ground surface at or near the repository temperature.

C-3.2 Air Conductivities

The resistance to air flow for incompressible fluid flow through shafts and drifts is dependent on the lengths and cross-sectional areas of the flow regime and the air conductivities of the backfill and the surrounding MPZs. In the present analyses, the MPZs were modeled around the shafts, but not around the drifts (see below). The cross-sectional areas and lengths for vertical and horizontal emplacement were used in the analysis. The cross-sectional area of the MPZ developed around the shafts was assumed to extend out one radius from the wall. For ramps that have a noncircular cross section, the MPZ area was calculated from the equivalent radius of a circle with the same area.

Shaft fill, air conductivities were varied over a range from 3.0×10^{-6} m/min to 3.0 m/min equivalent to a range of hydraulic conductivity from 10^{-4} to 100 cm/s. The upper bound for air conductivity corresponds to a gravel, while the lower bound corresponds to a silty sand (Freeze and Cherry, 1979, p. 29). The lower bound might also correspond to a compacted backfill engineered for low permeability by adding silt or clay fines.

The equivalent air conductivity of the MPZ was taken to be 20 and 60 times higher than the conductivity of the undisturbed tuff averaged over an annulus one radius wide. The equivalent conductivity factor of 20 corresponds to expected conditions at depth. The equivalent conductivity factor under worst case assumptions ranged from 40 to 80 times the undisturbed conductivity. The average value of 60 was selected for analysis. For Mechanism A, the air conductivity of undamaged tuff was 3×10^{-7} m/min, corresponding to a hydraulic conductivity of 10^{-5} cm/s. The damaged zone thus had an air conductivity of either 6×10^{-6} m/min or 1.8×10^{-5} m/min. For Mechanism B, the equivalent conductivity of the overlying rock was determined, as explained subsequently, to

take into account strata with varying conductivities, and the MPZ was assumed to be either 20 or 60 times more permeable than the undamaged rock in each stratigraphic unit.

For flow through the roof above the waste emplacement drifts (Mechanism B), it is necessary to know the cross-sectional area of the flow regime for either vertical or horizontal emplacement as well as the air conductivity. This area was taken as the total roof area of the emplacement drifts containing waste. These areas were estimated to be 983,700 m^2 for vertical emplacement and 486,000 m^2 for horizontal emplacement. Roof areas above the mains and submains were included in this calculation since it is expected that thermal convection would develop circulation throughout the underground repository.

The equivalent conductivity for flow through the rock to the ground surface was calculated according to the relation (Freeze and Cherry, 1979, p. 34)

$$\frac{L}{K'_r} = \sum_{i=1}^{n} \frac{L_i}{K'_i} \qquad \text{(C-1)}$$

where

K'_r = equivalent conductivity of the tuff for vertical flow (m/min),
L = distance from the repository roof to the ground surface (m),
L_i = thickness of each unit (m), and
K'_i = air conductivity of each unit (m/min).

In the present analyses, the thickness of the welded units (Tiva Canyon and Topopah Spring) is 260 m and the thickness of the nonwelded Paintbrush is 40 m. The air conductivity* of the nonwelded Paintbrush was assumed to be either 3×10^{-7} m/min or 3×10^{-5} m/min. This corresponds to a range of hydraulic conductivity from 10^{-5} to 10^{-3} cm/s. The welded tuff units (Tiva Canyon and Topopah Spring) were assumed to have either an air conductivity of 3×10^{-7} or 3×10^{-4} m/min. This corresponds to a range of hydraulic conductivities of from 10^{-5} to 10^{-2} cm/s.

Equation C-1 indicates that the equivalent conductivity of strata in series tends to be dominated by the unit with the lowest conductivity. Table C-1 shows the equivalent conductivities calculated for the low, intermediate, and high combinations of conductivities of the welded and nonwelded units.

*Air conductivity may be derived by calculating an intrinsic permeability from the hydraulic conductivity relationship presented by Freeze and Cherry (1979, p. 27) and then by calculating the air conductivity using the fluid properties of air.

Table C-1. Conductivity of Rock to Water and Air

RUNS	K_w (cm/s)	K_{nw} (cm/s)	K'_w (m/min)	K'_{nw} (m/min)	K'_r (m/min)
7 - 12	10^{-5}	10^{-5}	3×10^{-7}	3×10^{-7}	3×10^{-7}
13 - 18	10^{-2}	10^{-5}	3×10^{-4}	3×10^{-7}	2×10^{-6}
19 - 24	10^{-2}	10^{-3}	3×10^{-4}	3×10^{-5}	1×10^{-4}

K_w and K_{nw} = hydraulic conductivity of welded and nonwelded tuff, respectively (see text for sources),

K'_w and K'_{nw} = air conductivity of welded and nonwelded tuff, respectively, and

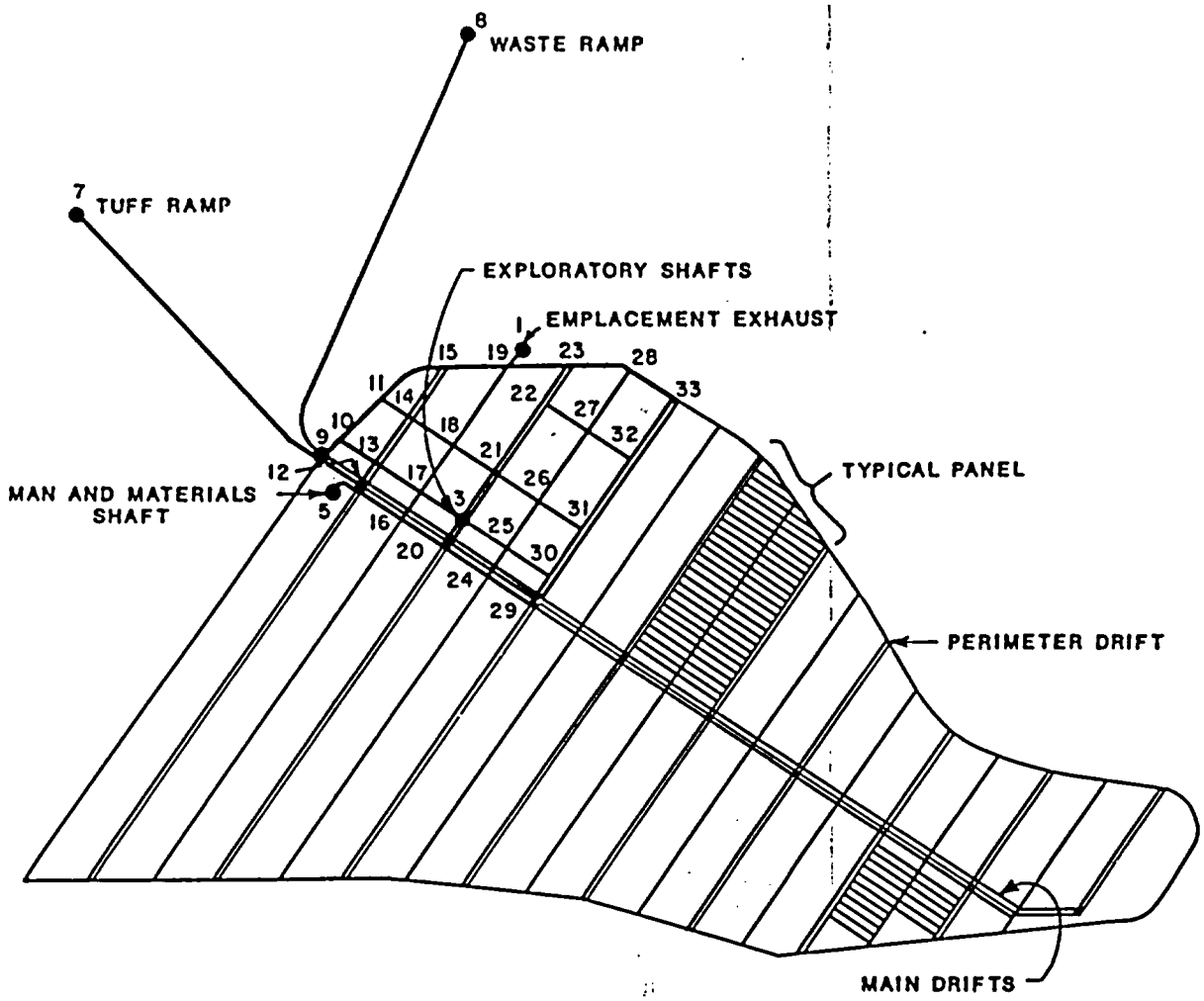
K'_r = air conductivity of the tuff units in series.

A comparison of the resistances calculated for open drifts and filled shafts indicated a very large contrast. For practical purposes in calculating total air flow, the resistance to flow from the open drifts may be neglected. However, differences in air flow resistance within the network of underground drifts may affect the distribution of air flow between emplacement drifts and development areas, as described later.

C-3.3 Model Geometry

Several networks were used to analyze convective transport. For Mechanism A, two networks to evaluate flow rate and underground flow distribution were used for combinations of filled shafts or ramps and open or backfilled drifts for vertical emplacement. First, a simple network in which the resistance to flow in drifts was assumed to be zero was used to calculate the total flow rate through the system. Second, the total flow rate calculated from the first network was imposed on an open drift network representing part of the repository (Figure C-2) to determine the distribution of flow in the repository between the mains and emplacement areas. This network was also used for analyses in which drifts were backfilled for the vertical emplacement configuration. A simple network was also used for the horizontal emplacement configuration to calculate total flow rate.

In the network used for Mechanism B, the drifts were assumed to be open and to offer no resistance to flow. The total, air-flow rate was calculated for series flow down the peripheral shafts and ramps (including associated damaged zones) and up the exploratory shafts and the tuff strata above the repository. (No attempt was made to calculate the distribution of flow between the emplacement drifts and the mains and access drifts for Mechanism B.). By definition in this case, all flow through the rock occurs in the emplacement drifts. This is a conservative definition because in reality some flow up the



NETWORK SHOWN BOLD SUPERIMPOSED ON REPOSITORY LAYOUT

(NOTE: Repository layout has been modified since analyses were performed)

Figure C-2. Network Used in Analysis of Convective, Air Flow

two outlet shafts could be short-circuited through the mains without passing through the emplacement drifts.

C-4 RESULTS

C-4.1 Total Flow

The results for total flow rates under various conditions for vertical and horizontal emplacement are presented in Table C-2 for the two mechanisms. The first set of analyses (1 through 6) applies to Mechanism A. Note that both open and backfilled drifts were evaluated. The second set of analyses (7 through 24) applies to Mechanism B, in which the drifts were open in all cases. Each of the analyses was run with high- and low-conductivity, MPZ models around the shafts and ramps. The tabulated results allow for comparison among the several runs of the effects of a MPZ, the presence of backfill in the drifts (Mechanism A), and the effects of variations in backfill and tuff conductivity.

The calculated total, air-flow rate for the base case Mechanism A (Analysis No. 1 with a relatively conductive backfill) is 0.06 m^3/min (2.1 ft^3/min). This flow rate may be compared directly with 0.08 m^3/min (2.8 ft^3/min) obtained for the same conditions for Mechanism B (Analysis No. 11). The total flow rate in Mechanism B is increased by about 25% relative to Mechanism A. Some of the increased flow therefore represents flow through the rock.

A detailed network analysis of Analysis No. 1 showed that approximately 35% of the flow would pass directly through the mains and access drifts to the ES-1. The estimated flow rate passing through the emplacement drifts is therefore about 65% of the total flow rate, or 0.04 m^3/min (1.4 ft^3/min).

C-4.2 Effects of System Components on Flow Rate

The convective, air-flow analysis results for Mechanism B are presented as a series of plots. The relationship of total flow rate out of the repository to shaft fill, air conductivity for vertical and horizontal emplacement configurations, and low- and high-conductivity, MPZ models are presented in Figures C-3 through C-6. The flow rate through ES-1 and ES-2 expressed as a percentage of the total flow rate out of the repository are presented in Figures C-7 through C-10. The three curves on each plot represent the low, intermediate, and high, rock conductivity combinations presented previously.

Effect of Shaft Fill, Air Conductivity

The distribution of flow through the shaft fill, the MPZ, and the tuff roof rock was found to be dependent on the shaft fill, air conductivity. With high, shaft fill, air conductivity of 1 m/min, the flow into and out of the repository is dominantly through the shaft fill with total flow ranging from approximately 1 to 10 m^3/min, depending on the conductivity of the roof

Table C-2. Results of Convective, Air Flow Analyses

Mechanism A: Flow Through Underground Openings Only

Analysis No.	1	2	3	4	5	6
Shaft Fill Conductivity to Air						
3 m/min		X			X	
3×10^{-2} m/min	X			X		
3×10^{-5} m/min						
3×10^{-6} m/min			X			X
Drift Backfill						
drifts open	X	X	X			
drifts backfilled				X	X	X
Vertical Emplacement						
Total flow rate with 60 MPZ (m^3/min)	6.03×10^{-2}	6.01	1.46×10^{-4}	4.90×10^{-2}	4.89	2.35×10^{-5}
Total flow rate with 20 MPZ (m^3/min)	6.02×10^{-2}	6.01	5.28×10^{-5}	4.89×10^{-2}	4.89	1.75×10^{-5}
Horizontal Emplacement						
Total flow rate with 60 MPZ (m^3/min)	5.95×10^{-2}	5.94	1.45×10^{-4}	---	---	---
Total flow rate with 20 MPZ (m^3/min)	5.94×10^{-2}	5.94	5.21×10^{-5}	---	---	---

Table C-2. Results of Convective, Air Flow Analyses (Continued)

Mechanism B: Flow Through Underground Openings Plus Flow Through Rock Overlying the Repository

Analysis No.	7[a]	8[a]	9[a]	10[a]	11[a]	12[a]
Shaft Fill Conductivity to Air						
3 m/min						X
3×10^{-2} m/min					X	
3×10^{-3} m/min				X		
3×10^{-4} m/min			X			
3×10^{-5} m/min		X				
3×10^{-6} m/min	X					
Drift Backfill						
drifts open	X	X	X	X	X	X
drifts backfilled						
Vertical Emplacement						
Total flow rate with 60 MPZ (m^3/min)	5.88×10^{-4}	8.21×10^{-4}	3.00×10^{-3}	1.71×10^{-2}	8.07×10^{-2}	6.03
Total flow rate with 20 MPZ (m^3/min)	2.16×10^{-4}	4.53×10^{-4}	2.67×10^{-3}	1.69×10^{-2}	8.06×10^{-2}	6.03
Horizontal Emplacement						
Total flow rate with 60 MPZ (m^3/min)	5.53×10^{-4}	7.66×10^{-4}	2.67×10^{-3}	1.31×10^{-2}	6.99×10^{-2}	5.95
Total flow rate with 20 MPZ (m^3/min)	2.05×10^{-4}	4.26×10^{-4}	2.39×10^{-3}	1.30×10^{-2}	6.98×10^{-2}	5.95

(a) These runs are for the low rock conductivity combinations, i.e., the air conductivity is 3×10^{-7} m/min for both the welded and nonwelded tuff units.

Table C-2. Results of Convective, Air Flow Analyses (Continued)

Mechanism B: Flow Through Underground Openings Plus Flow Through Rock Overlying the Repository

Analysis No.	13[b]	14[b]	15[b]	16[b]	17[b]	18[b]
Shaft Fill Conductivity to Air						
3 m/min						X
3×10^{-2} m/min					X	
3×10^{-3} m/min				X		
3×10^{-4} m/min			X			
3×10^{-5} m/min		X				
3×10^{-6} m/min	X					
Drift Backfill						
drifts open	X	X	X	X	X	X
drifts backfilled						
Vertical Emplacement						
Total flow rate with 60 MPZ (m^3/min)	4.21×10^{-3}	4.45×10^{-3}	6.76×10^{-3}	2.80×10^{-2}	1.55×10^{-1}	6.18
Total flow rate with 20 MPZ (m^3/min)	1.44×10^{-3}	1.67×10^{-3}	4.04×10^{-3}	2.57×10^{-2}	1.54×10^{-1}	6.18
Horizontal Emplacement						
Total flow rate with 60 MPZ (m^3/min)	3.96×10^{-3}	4.18×10^{-3}	6.30×10^{-3}	2.46×10^{-2}	1.18×10^{-1}	6.02
Total flow rate with 20 MPZ (m^3/min)	1.36×10^{-3}	1.59×10^{-3}	3.79×10^{-3}	2.26×10^{-2}	1.17×10^{-1}	6.02

[b] These runs are for the intermediate, rock conductivity combination, i.e., the air conductivities are 3×10^{-4} m/min and 3×10^{-7} m/min for the welded and nonwelded tuff units, respectively.

Table C-2. Results of Convective, Air Flow Analyses (Concluded)

Mechanism B: Flow Through Underground Openings Plus Flow Through Rock Overlying the Repository (Concluded)

Analysis No.	19[c]	20[c]	21[c]	22[c]	23[c]	24[c]
Shaft Fill Conductivity to Air						
3 m/min						X
3×10^{-2} m/min					X	
3×10^{-3} m/min				X		
3×10^{-4} m/min			X			
3×10^{-5} m/min		X				
3×10^{-6} m/min	X					
Drift Backfill						
drifts open	X	X	X	X	X	X
drifts backfilled						
Vertical Emplacement						
Total flow rate with 60 MPZ (m^3/min)	2.56×10^{-1}	2.57×10^{-1}	2.59×10^{-1}	2.82×10^{-1}	5.12×10^{-1}	13.0
Total flow rate with 20 MPZ (m^3/min)	8.63×10^{-2}	8.66×10^{-2}	8.90×10^{-2}	1.13×10^{-1}	3.47×10^{-1}	12.9
Horizontal Emplacement						
Total flow rate with 60 MPZ (m^3/min)	2.41×10^{-1}	2.41×10^{-1}	2.44×10^{-1}	2.65×10^{-1}	4.75×10^{-1}	9.97
Total flow rate with 20 MPZ (m^3/min)	8.20×10^{-2}	8.22×10^{-2}	8.44×10^{-2}	1.07×10^{-1}	3.24×10^{-1}	9.93

[c] These runs are for the high, rock conductivity combination, i.e., the air conductivities are 3×10^{-4} m/min and 3×10^{-5} m/min for the welded and nonwelded tuff units, respectively.

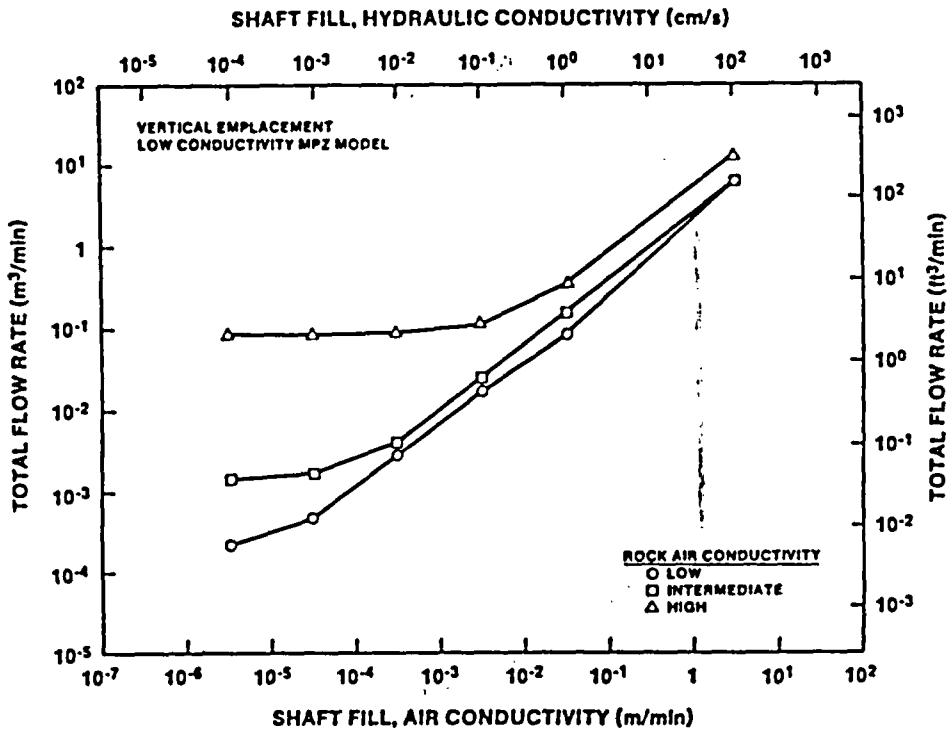


Figure C-3. Total Flow Rate as a Function of Shaft Fill, Air Conductivity (Vertical Emplacement and Low Conductivity, MPZ Model)

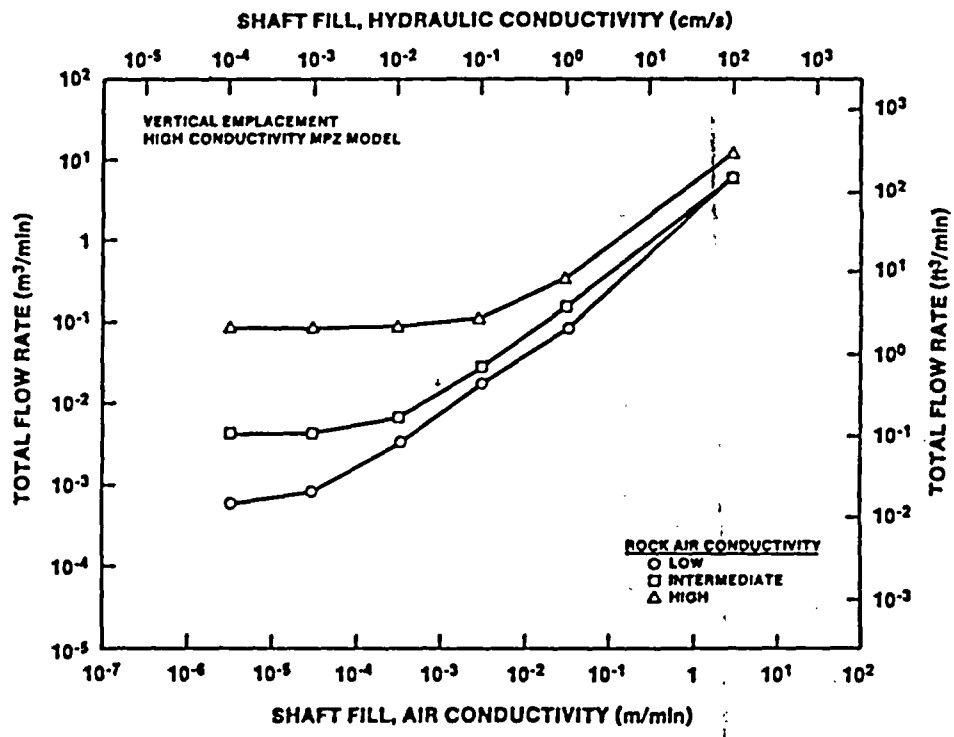


Figure C-4. Total Flow Rate as a Function of Shaft Fill, Air Conductivity (Vertical Emplacement and High Conductivity, MPZ Model)

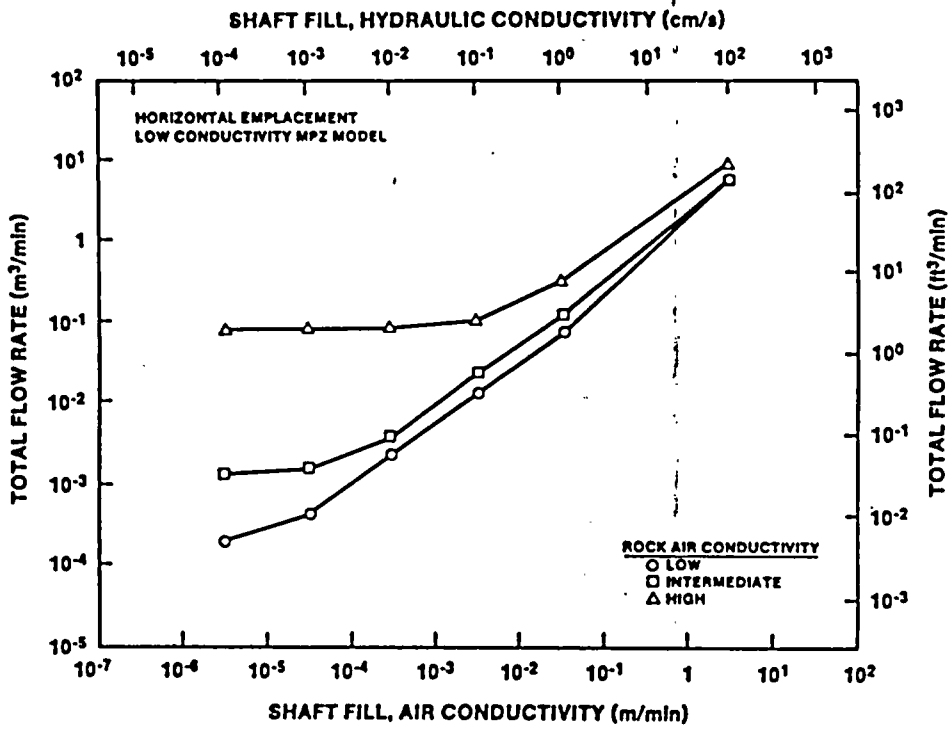


Figure C-5. Total Flow Rate as a Function of Shaft Fill, Air Conductivity (Horizontal Emplacement and Low Conductivity, MPZ Model)

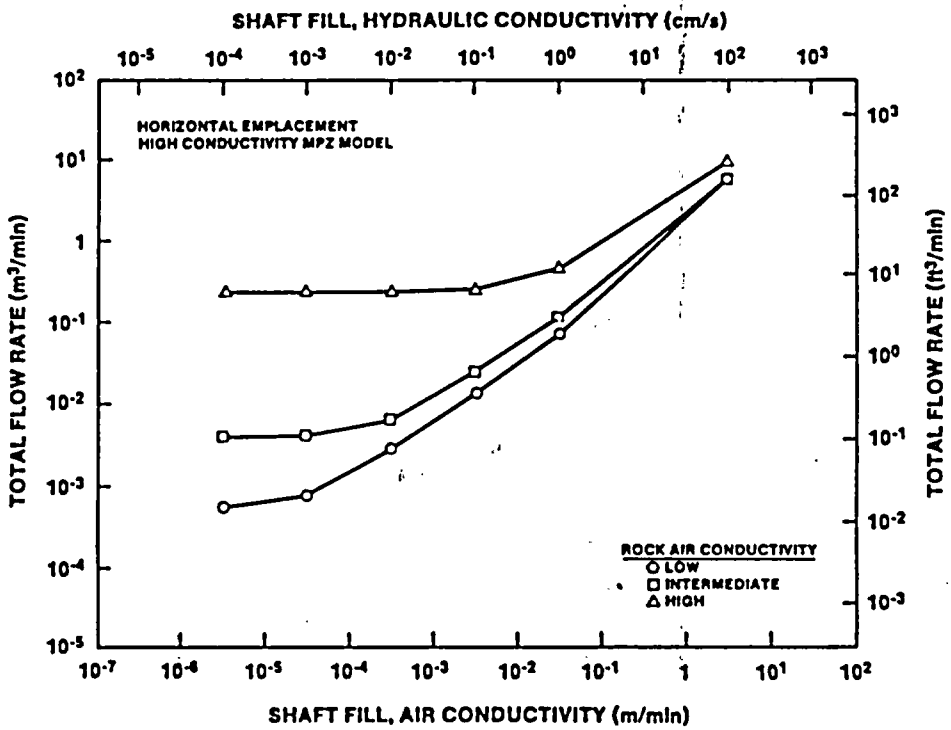


Figure C-6. Total Flow Rate as a Function of Shaft Fill, Air Conductivity (Horizontal Emplacement and High Conductivity, MPZ Model)

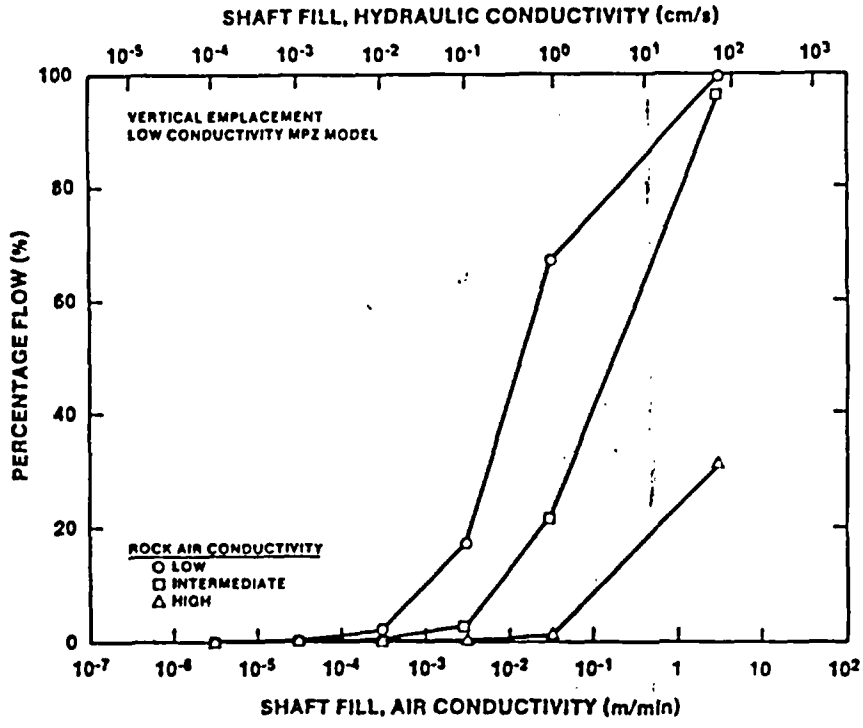


Figure C-7. Air Flow Through ES-1 and ES-2 (Shaft Fill and MPZ Flows Included) as a Percentage of Flow Through the Rock Over Repository Area (Vertical Emplacement and Low Conductivity, MPZ Model)

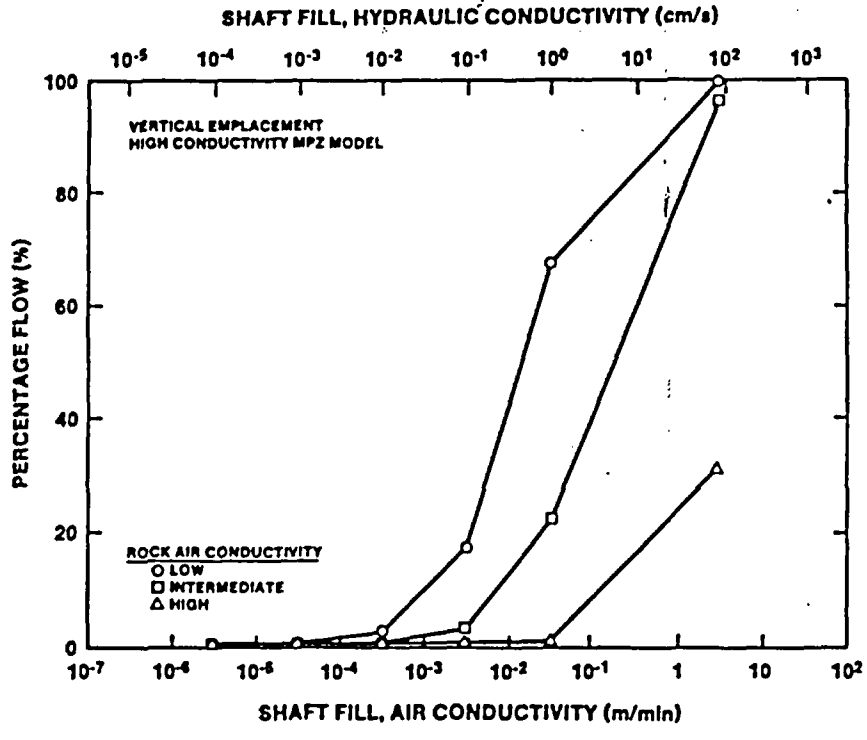


Figure C-8. Air Flow Through ES-1 and ES-2 (Shaft Fill and MPZ Flow Included) as a Percentage of Flow Through Rock Over Repository (Vertical Emplacement and High Conductivity, MPZ Model)

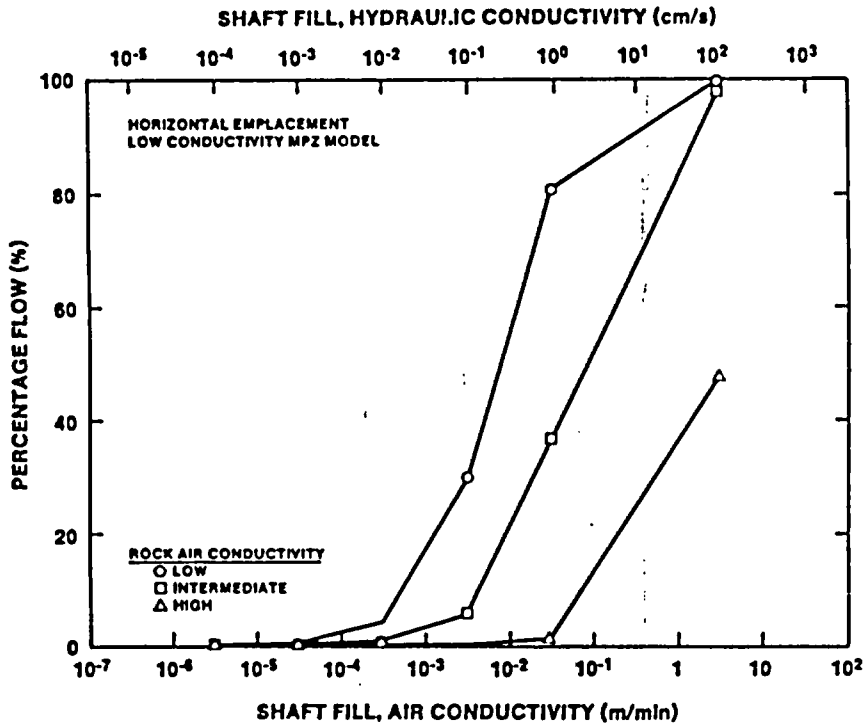


Figure C-9. Air Flow Through ES-1 and ES-2 (Shaft Fill and MPZ Flow Included) as a Percentage of Flow Through the Rock Over Repository (Horizontal Emplacement and Low Conductivity, MPZ Model)

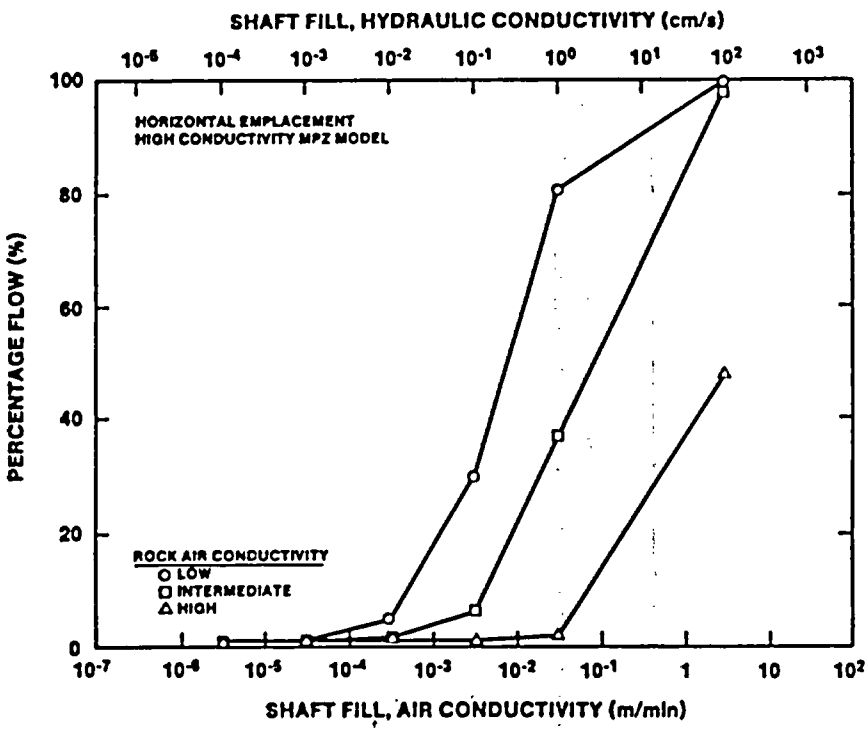


Figure C-10. Air Flow Through ES-1 and ES-2 (Shaft Fill and MPZ Flow Included) as a Percentage of Flow Through the Rock Over Repository (Horizontal Emplacement and High Conductivity, MPZ Model)

rock. With low, shaft fill conductivities (less than 10^{-5} m/min), flow into the repository is primarily through the MPZ, while flow out of the repository is dominantly through the tuff roof rock, and total flow rates are less than 10^{-1} m/min. The high conductivity, MPZ model results in somewhat higher flow rates than the low conductivity, MPZ model under these circumstances.

Effect of Backfill in the Drifts

The presence of backfill in the emplacement drifts in Mechanism A (Analyses 4-6) has a relatively minor effect on total air flow and air flow past waste containers over the range of backfill conductivities from 3×10^{-2} to 3 m/min. These results confirm that the main resistance to air flow in Mechanism A is in the shafts and ramps, with the network of drifts providing little resistance. Therefore, backfilling only the shafts and ramps is effective in reducing overall flow. It is postulated that the MPZ around the drifts (not included in these analyses) would have little effect on flow rate, provided high-air-conductivity, backfill materials were emplaced in the drifts. If low-conductivity materials were placed, much of the flow would occur in the MPZ, but the flow rate would be very low.

Effects of Host Rock, Air Conductivity

The effects of variations in the conductivity of the overlying rock on air flow into and out of the repository were evaluated for Mechanism B. It should be noted that the conductivity of the rock was determined by assuming flow in series through welded and nonwelded tuff units, as summarized in Table C-1.

The conductivity of the tuff units in series influences the total air flow through the repository. For the high-conductivity combination, the total flow begins to level off toward a constant value at a shaft fill, air conductivity of about 10^{-3} m/min. For the intermediate- and low-conductivity combinations, this stabilization of total flow rate occurs at shaft fill, air conductivity of approximately 10^{-5} m/min. At a low, air conductivity, the total flow rate varies over two orders of magnitude depending on the air conductivity of the rock.

In comparing total flow rate for the vertical and horizontal emplacement configurations, it is apparent that the results are very similar. This is because the geometry of the shafts and ramps accessing the repository are very similar. At a high, air conductivity, flow is dominantly through the shafts and ramps. At a low, air conductivity, resistance to flow is dominantly through the MPZ of the inlet shafts and ramps. In these analyses, no attempt has been made to distinguish temperature fields between the two emplacement configurations, although this may have some influence on calculated, upper-bound, convective, air-flow rates. It is reiterated that the assumption of the inlet shafts/ramps being at geothermal temperature is conservative for both emplacement configurations.

Effects of the Modified Permeability Zone

In the Mechanism A analyses, the MPZ affects the total flow only when the shaft fill conductivity is low (Analysis No. 3 or 6). In these cases, most of the flow occurs through the damaged zone, but the absolute flow is low (10^{-4} to 10^{-5} m^3/min) for vertical and horizontal emplacement.

In the Mechanism B analyses, the MPZ around the ESs is not a significant component of outflow. This occurs because the contrast in areas between the MPZs and undisturbed rock overlying the entire repository is much greater than the contrast in conductivities. With high, shaft fill, air conductivity, the outflow is dominantly through the shaft fill; with low, shaft fill, air conductivity, the outflow is dominantly through the undisturbed rock.

The MPZ has a more significant effect on inflow (with low, shaft fill, air conductivity) because inflow occurs only through the shaft and ramps and adjacent MPZs with no contribution from the undisturbed rock.

C-5 DISCUSSION

The results of the parametric analysis provide a basis for estimating flow rates through the underground network of shafts, ramps, and drifts. Total flow rates are dependent on the properties of the shaft fill, the MPZs surrounding the shafts and ramps, and the undamaged tuff. Total air flow rates from all of the analyses range from about 10^{-4} to about 10 m^3/min.

The distribution of flow through the shaft fill, the MPZ, and tuff roof rock was found to be dependent on the shaft fill, air conductivity. With high, shaft fill, air conductivity (3 m/min), the flow into and out of the repository is dominantly through the shaft fill with total flow approximately 6 to 13 m^3/min, depending on the conductivity of the roof rock. With low, shaft fill, air conductivities (less than 10^{-5} m/min), flow into the repository is primarily through the MPZ, while flow out of the repository is dominantly through the tuff, and total flow rates are less than 3×10^{-1} m^3/min.

The analysis indicates that the percentage of flow through ES-1 and ES-2 to total flow is also dependent on shaft fill, air conductivity. When the shaft fill conductivity is low, the percentage of flow through the shafts and ramps is also low, regardless of the existence of either a low- or high-conductivity MPZ. For example, for vertical emplacement with a shaft fill having an air conductivity less than 3×10^{-4} m/min (equivalent to a hydraulic conductivity of 10^{-2} cm/s), the contribution of ES-1 and ES-2 to total flow is about 2.5%. The percentage is somewhat higher for horizontal emplacement. This is attributable to a smaller roof area which tends to increase the percentage flow through ES-1 and ES-2. Nevertheless, for either emplacement configuration the percentage is smaller than 5.0% when the shaft fill, air conductivity is less than 3×10^{-4} m/min.

It is estimated that a significant percentage of the flow (60% to 100%) entering the repository would pass into the emplacement drifts. This is not only indicated by the results of the detailed network analysis, but also from the nature of thermal convection, in which the driving mechanism is hot air rising immediately above the heat source.

The analysis indicates that in order to reduce system flow it is most effective to fill shafts and ramps accessing the repository, and that little reduction in flow rate is achieved by backfilling drifts. If drifts are left open, the MPZ surrounding the drifts is not a significant flow regime.

APPENDIX D
HYDROLOGIC AND PHYSICAL PROPERTIES OF POTENTIAL BACKFILL MATERIALS

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APPENDIX D
HYDROLOGIC AND PHYSICAL PROPERTIES OF POTENTIAL BACKFILL MATERIALS

D.1 POROSITY OF CRUSHED TUFF

The porosity of cohesionless sands and gravels is a function of the grain size distribution and the degree of compaction. Generally, heterogeneous materials with a mixture of grain sizes will have lower porosities because the finer particles fill the pores between the larger particles.

For a given particle-size distribution, the porosity can be reduced by compactive effort. For example, for crushed salt, IT Corporation (1984, p. 7) found a porosity range of 40.3% to 24.8% corresponding to loose (poured into a container without compaction) and dense (standard Proctor) compaction. The particle size distribution in this case ranged from 20 mm to about 0.03 mm with an average (D_{50}) of 2 to 3 mm.

A similar relationship of porosity to compaction was found by Taylor et al. (1980, p. 143) for a coarsely crushed sample of basalt. The sample in this case was a fine gravel with the smallest size about 2.5 mm. The range in porosity between loose and maximum density was 52% to 43%. The smaller overall range (compared with the crushed salt) may be indicative of a more uniform gradation and the greater competency of the material (resulting in less breakdown during compaction).

Porosities of natural granular materials and artificial materials, as indicated by standard texts (e.g., Winterkorn and Fang, 1979, p. 257; Davis and DeWeist, 1966, p. 375), range typically from about 25% to 50%. Assuming that a backfill in a repository will contain a wide range of grain sizes, and that it will be moderately to well compacted, a reasonable estimated range of porosity is 30% to 35%.

D.2 SATURATED HYDRAULIC CONDUCTIVITY OF POTENTIAL BACKFILL MATERIALS

The permeability or hydraulic conductivity of a soil or a potential backfill material is, in general, influenced by the following major factors:

- grain size (magnitude and distribution),
- mineralogy (especially clay type and clay content), and
- density (degree of compaction).

Other factors, such as the degree of saturation, the nature of the permeant, fabric, temperature, and the hydraulic gradient, also influence permeability, but generally to a lesser degree. The relative importance of each of the factors listed above is dependent on the type of backfill; in particular, whether or not the material contains appreciable (greater than 5% to 10%) clay or silt. Two general classes of backfill are discussed below: crushed tuff and crushed tuff-clay mixtures. For these two classes of materials, the dominant

factors controlling permeability are discussed, and generalized relationships established between those factors and permeability are presented.

D.2.1 Hydraulic Conductivity of Cohesionless Backfill Materials

The hydraulic conductivity of sands and gravels is affected by both the predominant grain size and by the grain-size distribution. Generally, lower hydraulic conductivities are associated with finer grain size and lower void ratios. For example, hydraulic conductivity varies from greater than 10 cm/s for coarse gravel to 10^{-2} to 10^{-4} cm/s for fine sands (Terzaghi and Peck, 1967). Well-graded materials generally have lower permeabilities than well-sorted (poorly-graded) materials, particularly if the well-graded material contains a significant proportion of fine material such as silt and clay filling the voids between the larger grains. In other words, the fines in a material may have more influence on hydraulic conductivity than the predominant or average grain size.

The Hazen Equation (Terzaghi and Peck, 1967, p. 50) is sometimes used to estimate the hydraulic conductivity of sands:

$$K = C_1 D_{10}^2$$

where

K = hydraulic conductivity (cm/s),
C_1 = constant set to 100 in units of 1/(cm - s), and
D_{10} = the maximum grain size of the finest 10% (by weight) of the soil obtained by particle sieve analysis (cm).

This equation states that hydraulic conductivity increases with the square of the representative, grain size (D_{10}). The relation was originally developed for clean, relatively uniform sands and may be limited in its applicability to other cohesionless materials. It does not directly account for the effects of gradation and density.

The Hazen Equation is plotted in Figure D-1 together with data points for natural soils obtained from a literature search. The scatter in the data points in Figure D-1 represents, at least in part, the range of gradations and densities of the materials. For values of D_{10} that fall in the gravel or sand range, there is fairly good correlation between D_{10} and hydraulic conductivity. The general trend of Figure D-1 suggests that it is possible to estimate hydraulic conductivity within approximately one to two orders of magnitude for cohesionless materials with a known D_{10} greater than about 0.1 mm.

Density, which can also be represented by porosity or void ratio, also influences hydraulic conductivity, although the range of hydraulic

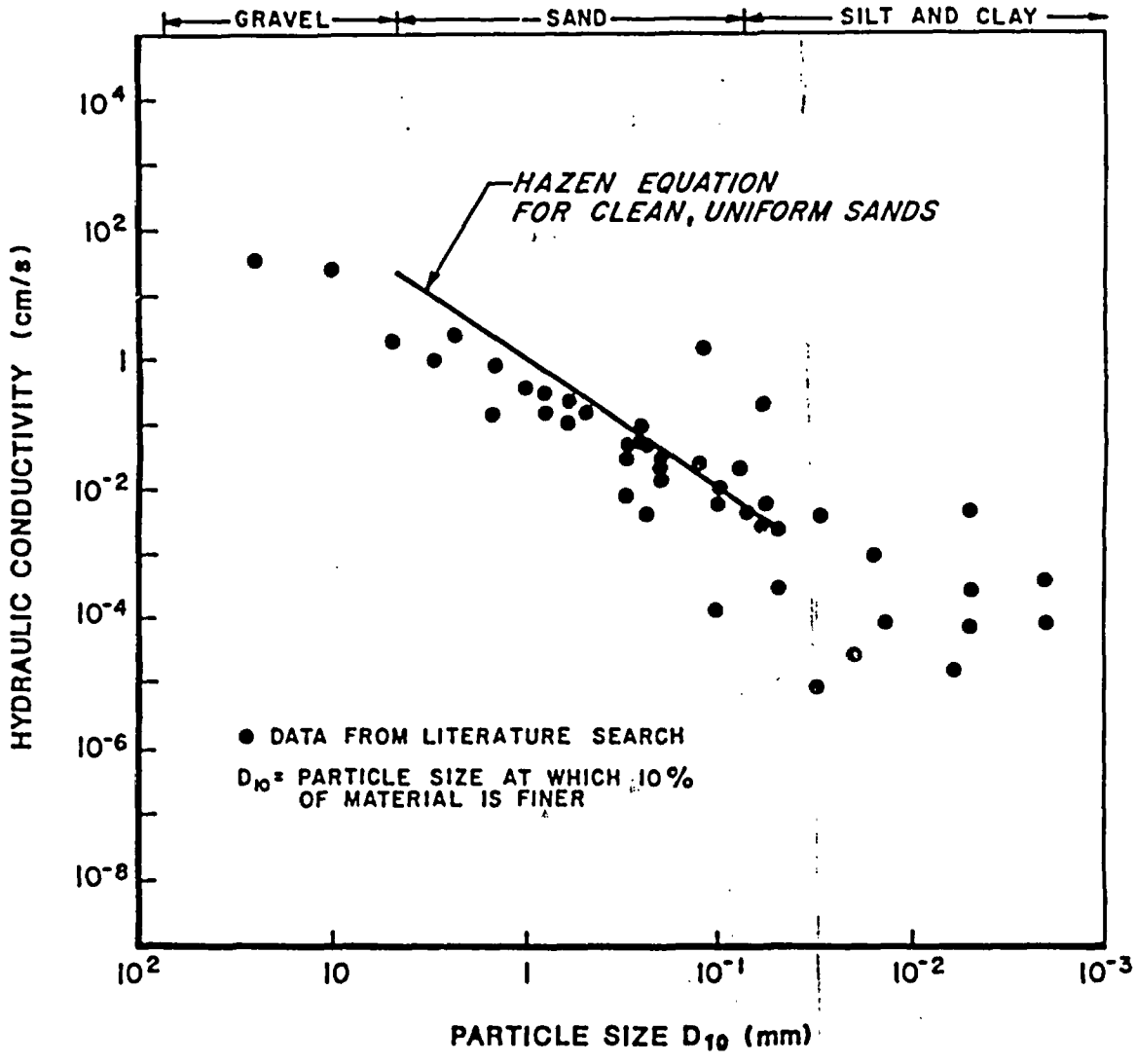


Figure D-1. Relationship Between Hydraulic Conductivity and D_{10} Particle Size

conductivity between loose- and dense-cohesionless materials may be only one order of magnitude or less. In cohesive materials, such as clays or materials containing about 30% or more (by weight) clay minerals, increased density, and reduced porosity result in decreased hydraulic conductivity of the material.

There is only limited information available regarding the hydraulic conductivity of crushed tuff. Particle-size distributions have been obtained for tuff crushed in the laboratory, and these can be used to estimate hydraulic conductivity by comparing the particle-size distribution against the grain-size distributions of materials with known hydraulic conductivity. This comparison may be made on the basis of the D_{10}-size parameter.

Figure D-2 shows particle-size distributions obtained from crushing tuff in a jaw crusher in four stages with a progressively smaller jaw opening. All of the materials are relatively coarse and might be classed as sandy gravels. The range of hydraulic conductivity estimated for these materials from the Hazen Equation is 1 to 10^2 cm/s. The costs of crushing to produce finer particle sizes increase disproportionately, and it would be more economical to reduce permeability by adding silt or clay to the crushed rock.

D.2.2 Hydraulic Conductivity of Crushed Tuff-Clay Mixtures

For tuff/clay mixtures, and in general for all fine-grained materials, the primary factors controlling hydraulic conductivity are the type and percentage of clay and the degree of compaction. Adding clay in an appreciable amount (greater than 5% to 10%) to a cohesionless material results in large reduction in hydraulic conductivity. The effect of adding clay may be expected to be proportionately less for clay contents greater than 30% to 40% when the clay effectively fills the voids between the larger grains.

The effect of clay percentage for a specific type of clay is illustrated in Figure D-3, which presents hydraulic conductivity for various sand-bentonite mixtures as a function of percentage of bentonite. Hydraulic conductivity ranges from approximately 10^{-4} cm/s for 5% bentonite to approximately 10^{-12} cm/s for 100% bentonite. The scatter in the data may be related to different densities, although all the materials tested were highly compacted. An important point to note from these data is that small amounts of bentonite can significantly reduce the hydraulic conductivity of cohesionless materials.

Information available in the literature concerning permeability versus clay content for other clay types was examined to determine whether relationships similar to that shown in Figure D-3 have been determined for other clay types. Limited published data exists for mixtures of kaolinite and illite clays. Figure D-4 presents this information and shows that bentonite (predominantly smectite) mixtures have a lower hydraulic conductivity than mixtures of other clays. The effect of clay content on hydraulic conductivity is seen to vary significantly with the type of clay. For example, mixtures using kaolinite clay have conductivities as low as 10^{-8} cm/s, while mixtures with bentonite

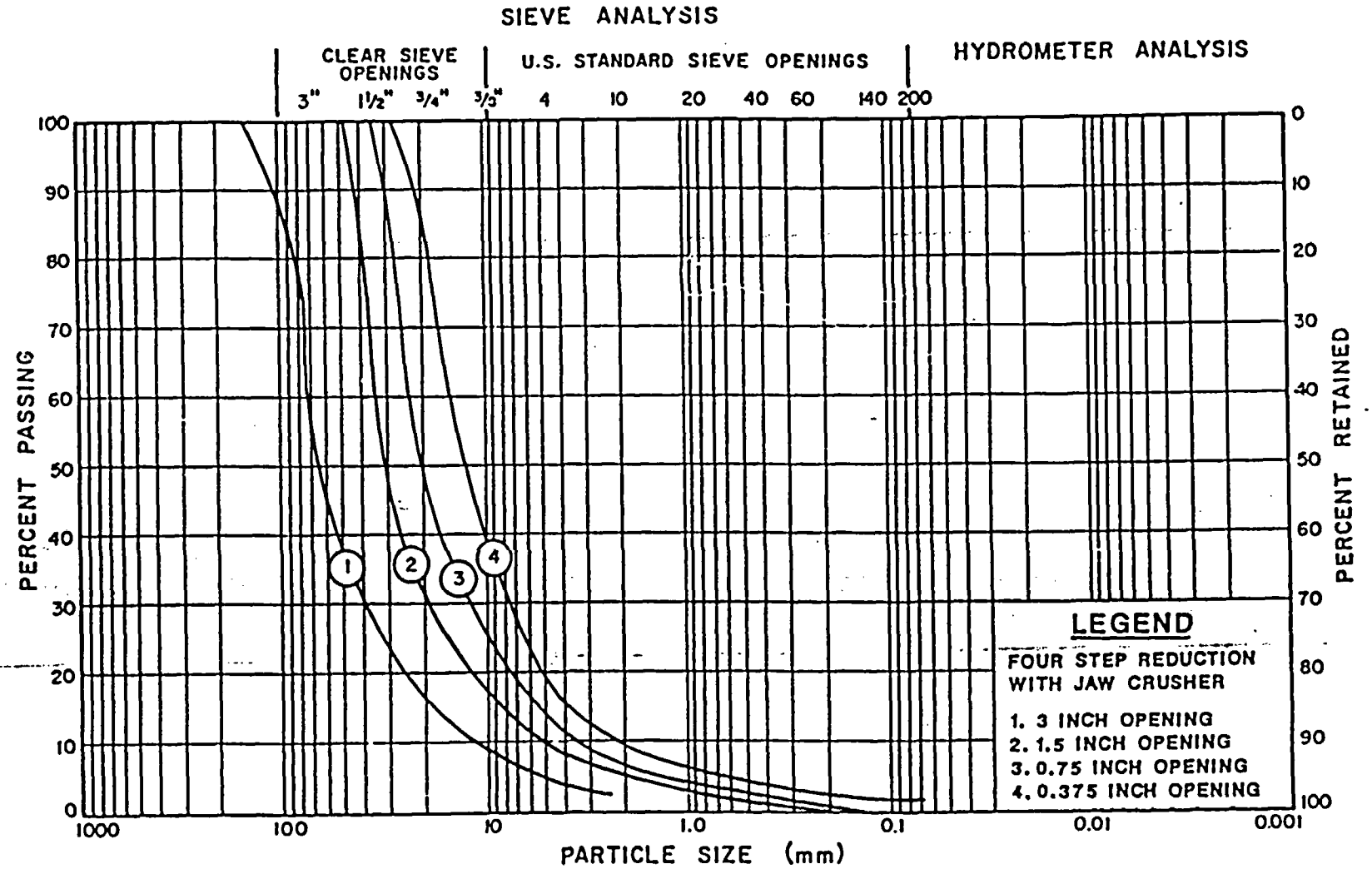
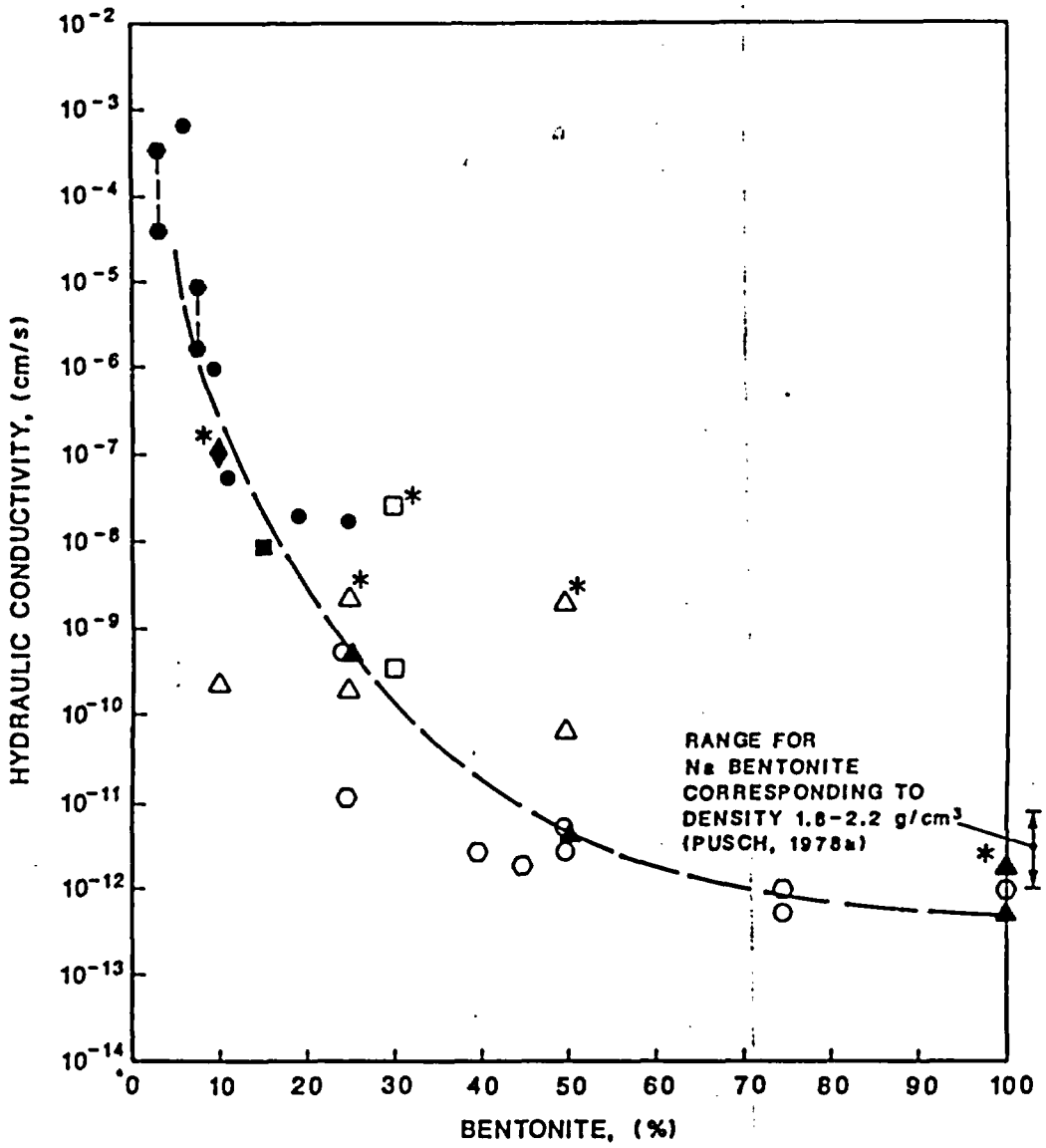


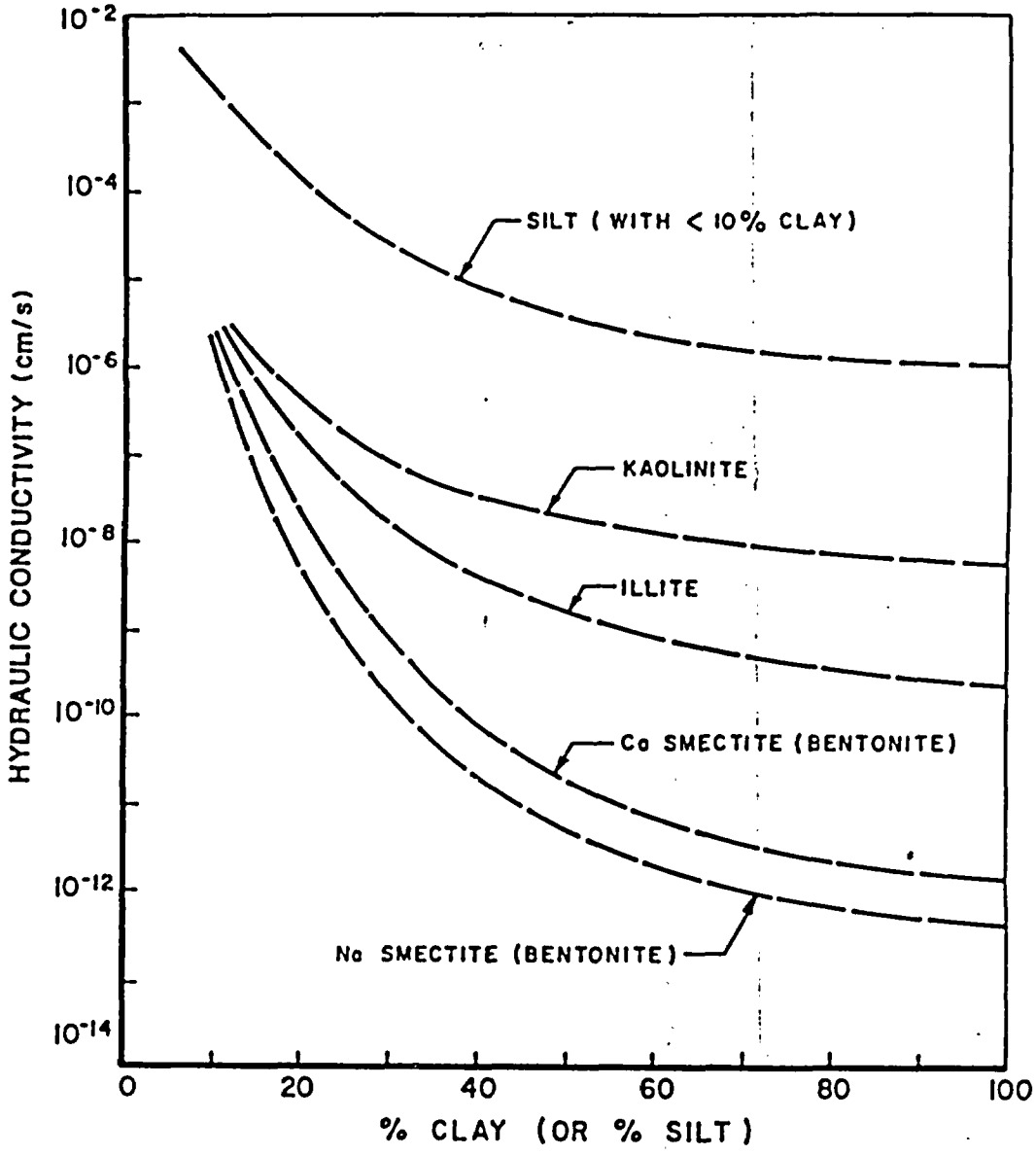
Figure D-2. Particle Size Distributions for Crushed Tuff (After Fernandez and Wong, in preparation)



LEGEND

- ♦ 10% OREGON BENTONITE, 90% GLACIOFLUVIAL SAND, MODEL PLUG, DENSITY-2.1 g/cm³ (TAYLOR ET AL, 1980)
- ▲ BENTONITE/SAND, DENSITY 2.0-2.2g/cm³ (WHEELWRIGHT ET AL, 1981)
- ● BENTONITE/SAND, DENSITY 1.8g/cm³ (PUSCH, 1979)
- ■ 10-20% BENTONITE, 10% SILT, 60-70% SAND, 10% GRAVEL, DENSITY 1.9-2.0g/cm³ (FAGERSTROM AND LUNDAHL, 1977)
- * NOTE: Ca BENTONITE, OTHER VALUES ARE Na BENTONITE
- ● BENTONITE/CRUSHED GRANITE, DRY DENSITY AT MAXIMUM MODIFIED PROCTOR (YONG, ET AL, 1986)
- □ BENTONITE/SAND (GRIM, 1962)
- ○ BENTONITE/SAND, DENSITY 2.09-2.18 g/cm³ (WESTIK, ET AL, 1983)
- △ BENTONITE/SAND AND BENTONITE/CRUSHED GRANITE, DENSITY 1.59-1.69 g/cm³ (RADHAKRISHNA, ET AL, 1982)
- ○ BENTONITE/SAND, (DIXON, ET AL, 1985)

Figure D-3. Hydraulic Conductivity of Compacted Bentonite/Sand Mixtures as a Function of Bentonite Content



NOTE: THESE CURVES ARE GENERALIZED FROM SPARSE DATA AND REPRESENT LOWER-BOUND ESTIMATES OF HYDRAULIC CONDUCTIVITY FOR HIGHLY COMPACTED MATERIALS. HYDRAULIC CONDUCTIVITIES FOR MATERIALS COMPACTED IN THE FIELD MAY BE 1-2 ORDERS OF MAGNITUDE HIGHER.

Figure D-4. Hydraulic Conductivity of Well-Compacted, Sand-Clay or Sand-Silt Mixtures

can be as low as 10^{-12} cm/s. The curves do not indicate the influence of other factors such as the type and degree of compaction or the moisture content. The curves were developed from data points representing well-compacted materials and thus represent approximate lower bounds to hydraulic conductivity for materials compacted at or close to the optimum moisture content. Materials that are not highly compacted at optimum moisture content will have hydraulic conductivities possibly several orders of magnitude higher than the values indicated by Figure D-4 for the type and proportion of clay.

Several points from Figure D-4 are worth noting.

- Significant reductions in hydraulic conductivity are possible with low percentages of clay content.
- The curves for all clay types tend to converge at approximately 10%, suggesting that at 10% clay, the type of clay is not highly significant.
- The effectiveness of using clay to decrease conductivity decreases with increasing clay content.
- Hydraulic conductivity values of the order of 10^{-7} cm/s can be achieved with all clays with low to moderate clay contents.

The curves of Figure D-4 do not indicate how characteristics such as gradation of the cohesionless fraction of the mixture may affect the hydraulic conductivity of the mixture. The apparent implication from the data used to derive the figure is that approximately the same permeabilities may be achieved if clay is added to coarse gravel as opposed to sand. Additional data are required to support this tentative conclusion. It should also be noted that the values shown in Figure D-4 apply only to homogeneous materials. Cracks could significantly increase permeability. Also, the permeability at the interface between a backfill and the rock could be much higher than the internal permeability of the backfill. In this regard, bentonite is advantageous in that it swells appreciably on contact with water and will tend to heal internal or interface cracks (Pusch, 1980; Seed et al., 1962; Sherif et al., 1982).

D.3 PROPERTIES OF UNSATURATED CRUSHED TUFF

A recent study by Abeele (1984) included data on the hydraulic conductivity of crushed Bandelier Tuff (from New Mexico) under unsaturated conditions. Some results from Abeele (1984) are presented in Table D-1. The hydraulic conductivity values range from approximately 10^{-8} m/s to about 5×10^{-11} m/s for varying moisture contents.

Table D-1. Hydraulic Characteristics of Crushed Bandelier Tuff as a Function of Matrix Potential[a]

Matrix Potential, -kPa	Outflow, g	Water Ratio by Volume	Specific Water Capacity, m^{-1}	Water Diffusivity, m^2/s	Hydraulic Conductivity, m/s
0	0.000	.400			
10	20.082	.356			
11	24.320	.347	8.98×10^{-2}	1.02×10^{-7}	9.18×10^{-9}
30	108.128	.166			
33	110.261	.162	1.58×10^{-2}	2.15×10^{-7}	3.38×10^{-9}
42	117.407	.147			
46	120.241	.140	1.50×10^{-2}	1.38×10^{-7}	2.07×10^{-9}
60	126.241	.127			
66	127.841	.124	5.64×10^{-3}	1.33×10^{-7}	7.51×10^{-10}
80	131.589	.116			
88	132.749	.113	3.07×10^{-3}	1.40×10^{-7}	4.30×10^{-10}
96	134.437	.109	2.98×10^{-3}	1.21×10^{-7}	3.60×10^{-10}
120	136.949	.104			
132	137.949	.102	1.76×10^{-3}	9.30×10^{-8}	1.64×10^{-10}
146	139.315	.099	2.06×10^{-3}	4.80×10^{-8}	9.91×10^{-11}
161	140.543	.096	1.73×10^{-3}	4.62×10^{-8}	8.01×10^{-11}
177	142.643	.092	2.78×10^{-3}	1.88×10^{-8}	5.22×10^{-11}
195	144.057	.089	1.66×10^{-3}	3.08×10^{-8}	5.13×10^{-11}

[a]Specific water capacity, water diffusivity, and hydraulic conductivity were measured only at pressure steps <10%. From Abeele (1984).

APPENDIX E
INTERSECTION OF REPOSITORY DRIFTS AND BOREHOLES WITH THE GHOST DANCE FAULT

FIGURES

PAGE

E-1 Number of Emplacement Rooms Intersected by the Ghost Dance Fault - Vertical Emplacement.....E-3

E-2 Number of Emplacement Boreholes Intersected by Ghost Dance Fault - Horizontal Emplacement.....E-4

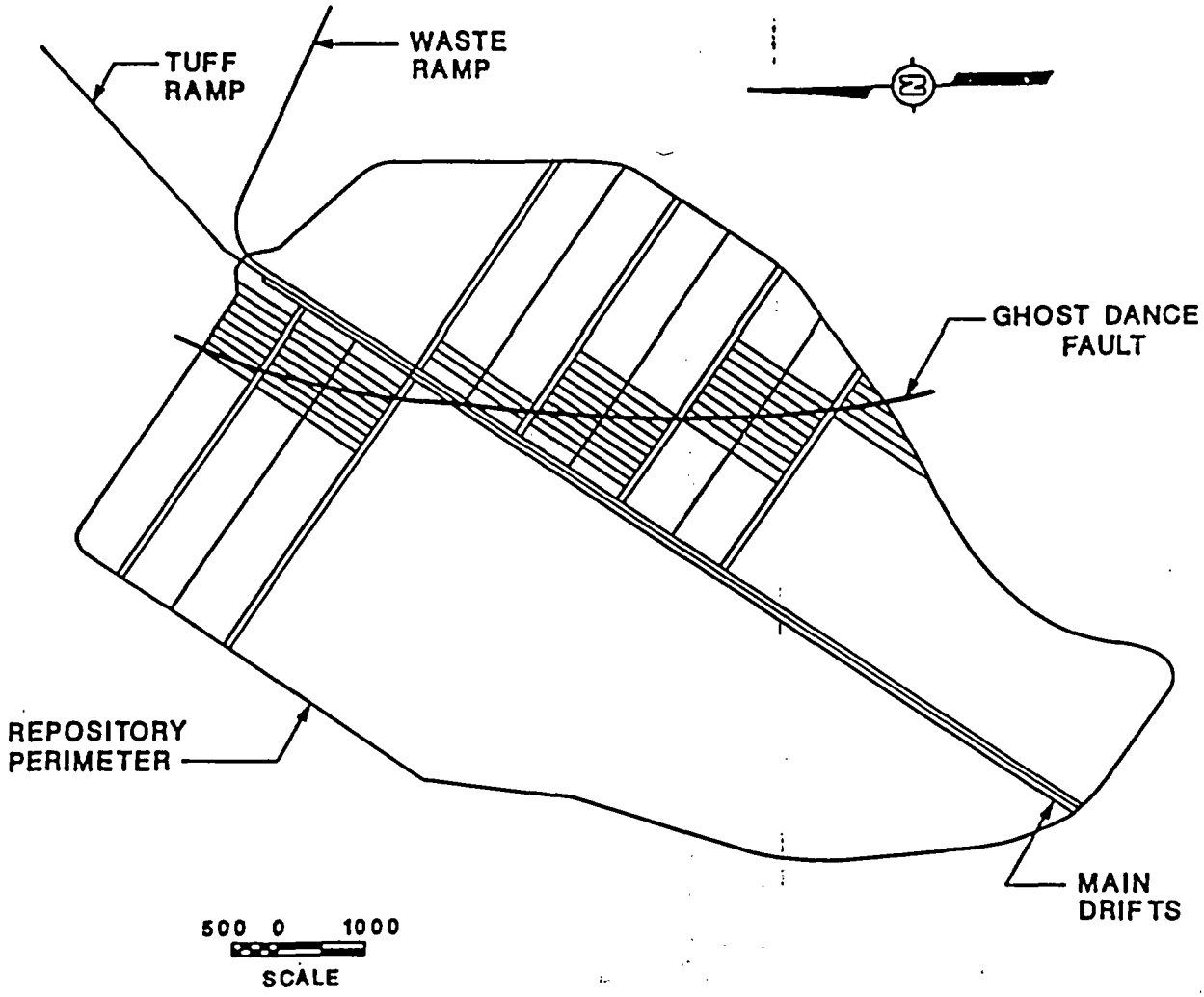
E-3 Surface Washes Crossing the Ghost Dance Fault.....E-5

APPENDIX E

INTERSECTION OF REPOSITORY DRIFTS AND BOREHOLES WITH THE GHOST DANCE FAULT

For the vertical emplacement configuration, about 24 rooms would be intersected by the Ghost Dance Fault (Figure E-1). If no waste were emplaced in any of these rooms, the disposal capacity lost would be 9.4% of the total capacity of the present repository layout. For the horizontal emplacement configuration, about 97 holes would be intersected, and the lost disposal capacity would be about 3.6% (Figure E-2).

Recharge to the Ghost Dance Fault is more likely to occur where the fault is crossed by washes at the ground surface. Figure E-3 shows that nine washes cross the fault within the repository perimeter.



NOTE: Only Emplacement Rooms Intersected by the Fault Are Shown.

Figure E-1. Number of Emplacement Rooms Intersected by the Ghost Dance Fault - Vertical Emplacement

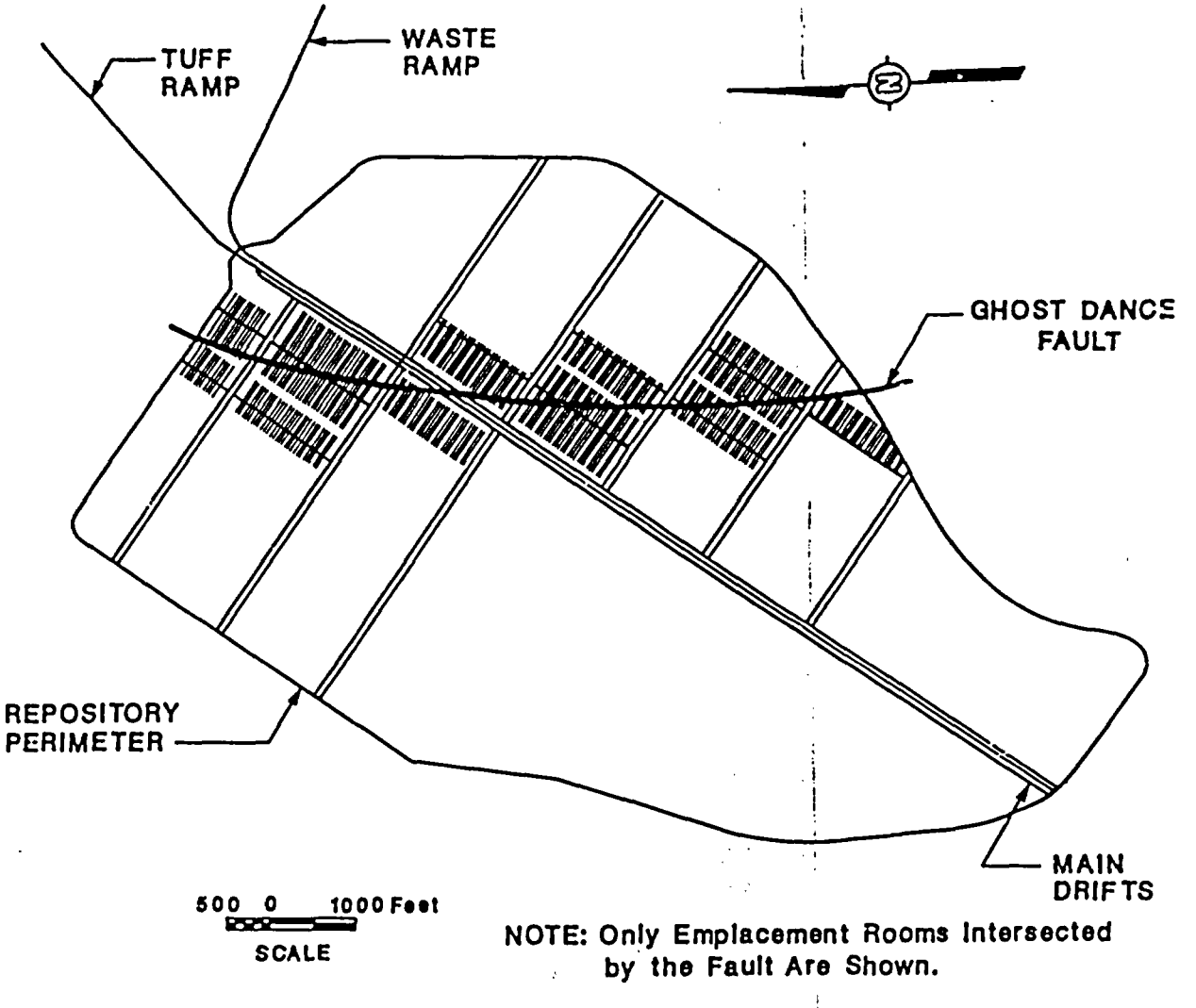


Figure E-2. Number of Emplacement Boreholes Intersected by Ghost Dance Fault - Horizontal Emplacement

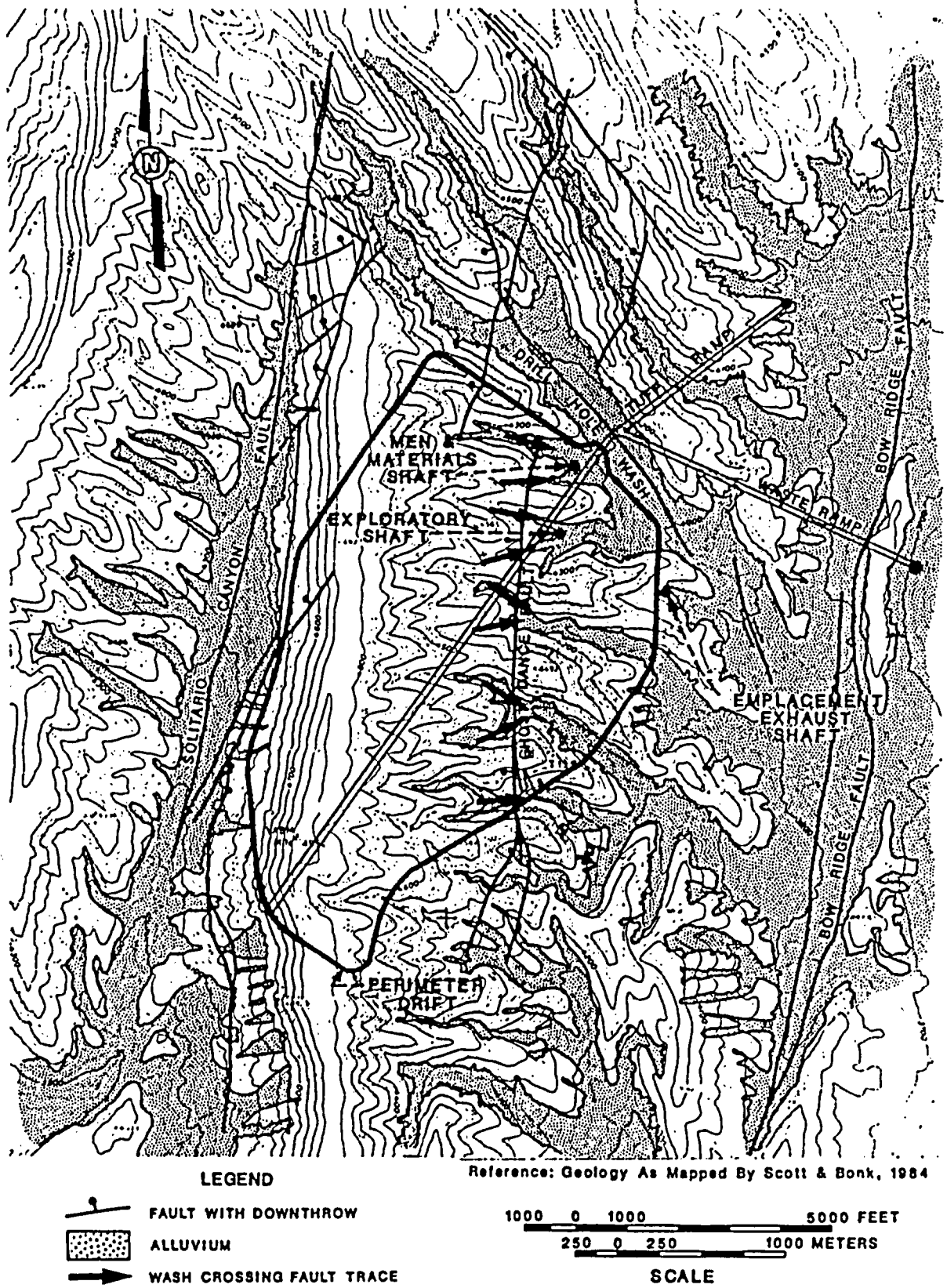


Figure E-3. Surface Washes Crossing the Ghost Dance Fault

APPENDIX F
DESIGN CHARTS USED TO DEVELOP HYDROLOGIC DESIGN REQUIREMENTS

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APPENDIX F
DESIGN CHARTS USED TO DEVELOP HYDROLOGIC DESIGN REQUIREMENTS

This appendix contains a series of design charts that support the selection of design requirements for sealing components placed in shafts, ramps, underground drifts, and horizontal emplacement holes. The models for flow under a pressure head are presented in Figure F-1. Each model corresponds to a specific design chart (Figures F-3 through F-7). The unconfined flow models are presented in Figure F-2 with each model corresponding to design charts, Figures F-8 through F-10.

The hydrological design goals as developed from the performance allocation and other considerations (Section 6.2) are presented on each chart. These goals are based upon the performance allocation to the shafts/ramps and the underground facility and the estimated number of seal components (see Tables 6-2 and 6-3 for specific calculations). Note that design goals are presented after repository closure and at the end of the sealing period (500 years for underground facility seals and 1,000 years for shaft/ramp seals). Note that in each case, the higher goals correspond to the end of the sealing period.

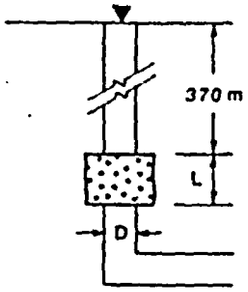
The parameters used to develop each design chart are given below:

Figure F-3 Plug at base of shaft
- Plug length, L = 9.1 m
- Shaft diameter, D = 6.9 m
- Water-column height above plug = 370 m
- Host rock, hydraulic conductivity = 10^{-2} cm/s

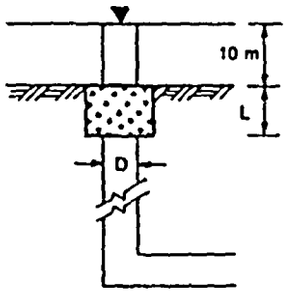
Figure F-4 Plug near top of shaft
- Plug length, L = 9.1 m
- Shaft diameter, D = 6.9 m
- Water-column height = 10 m
- Host rock, hydraulic conductivity = 10^{-2} cm/s

Figure F-5 Plug near base of ramp
- Plug length, L = 9.1 m
- Ramp diameter, D = 7.6 m
- Water-column height (vertical) above plug = 250 m
- Host rock, hydraulic conductivity = 10^{-2} cm/s

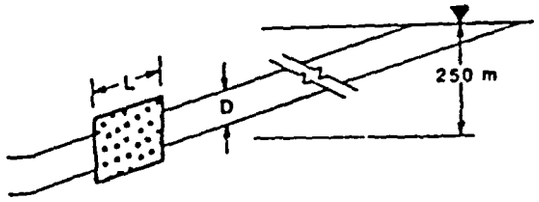
Figure F-6 Emplacement Drift Bulkhead
- Plug length, L = 12.8 m
- Drift height, H = 6.3 m
- Water-column height = 300 m
- Host rock, hydraulic conductivity = 10^{-2} cm/s



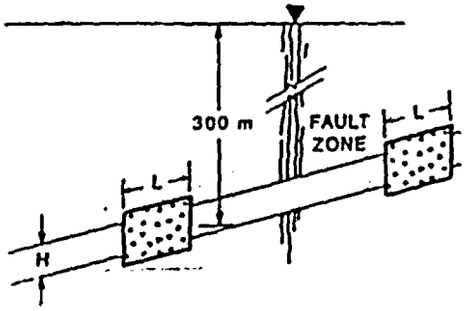
PLUG AT BASE OF SHAFT
(Figure F-3)



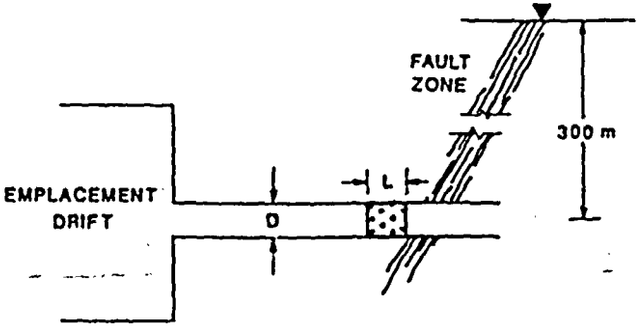
PLUG NEAR TOP OF SHAFT
(Figure F-4)



PLUG NEAR BASE OF RAMP
(Figure F-5)



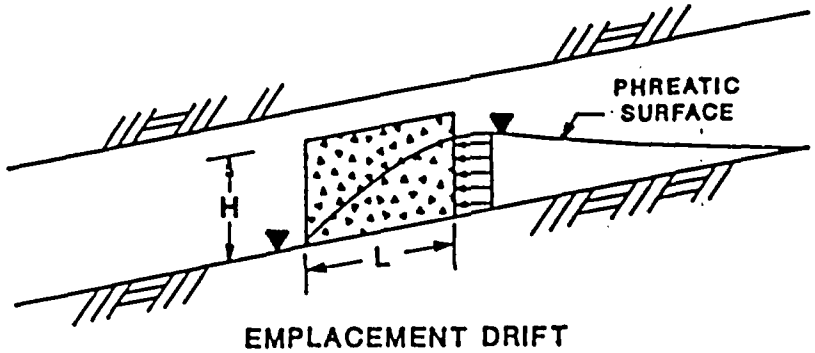
EMPLACEMENT BULKHEAD
(Figure F-6)



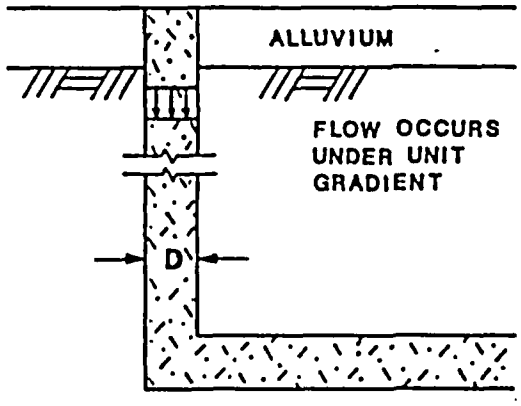
BOREHOLE PLUG IN A HORIZONTAL EMPLACEMENT HOLE
(Figure F-7)

NOTE: Each model references the figure with the corresponding design chart.
▼ indicates assumed water elevation

Figure F-1. Confined Flow Models



EMPLACEMENT DRIFT
OR PERIMETER DRIFT
(Figures F-8 and F-9)



SHAFT FILL
(Figure F-10)

NOTE: Each model references the figure with the corresponding design chart.
▼ Indicates assumed water elevation

Figure F-2. Unconfined Flow Models

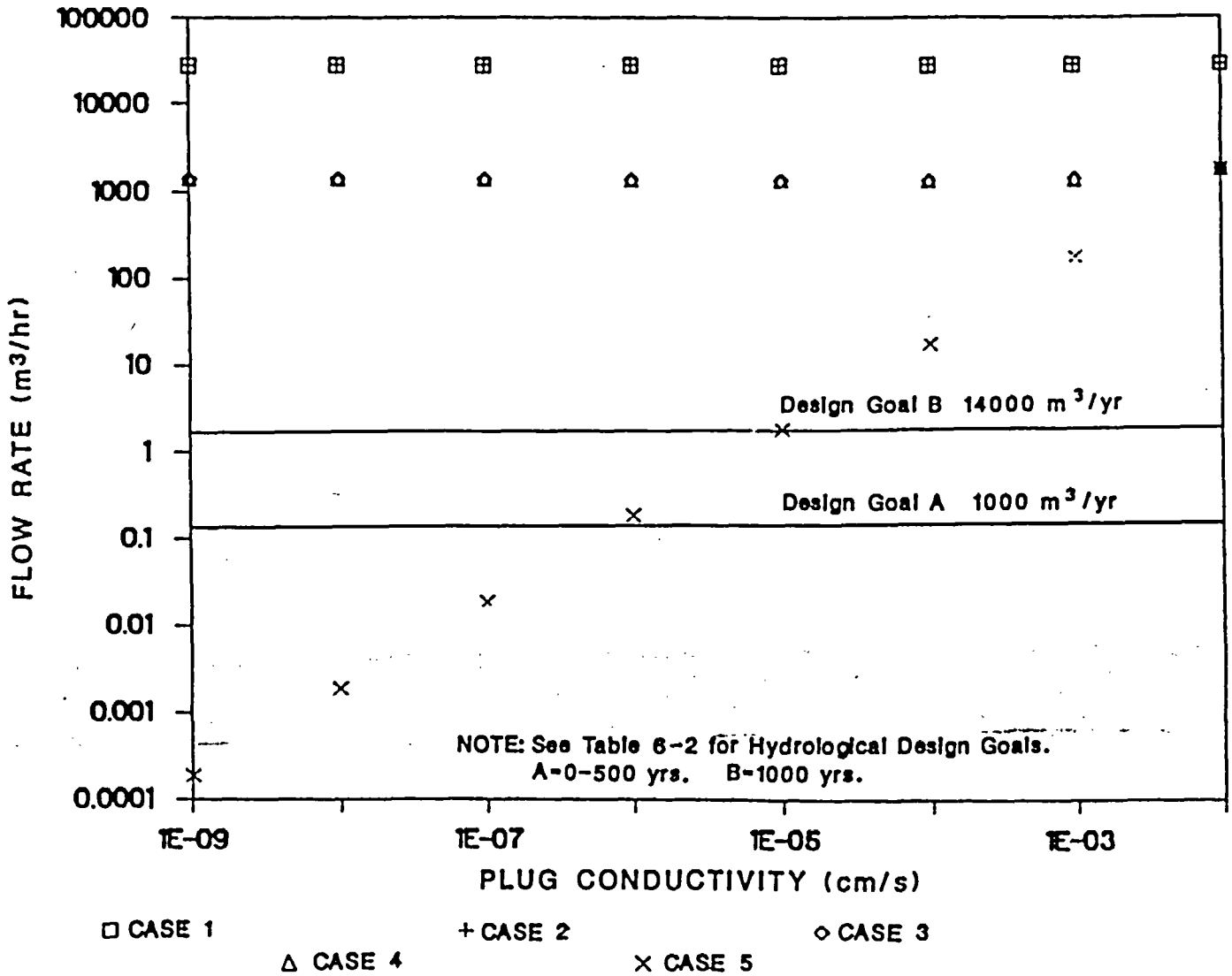


Figure F-3. Design Chart for a Seal/Plug Placed at the Base of a Shaft

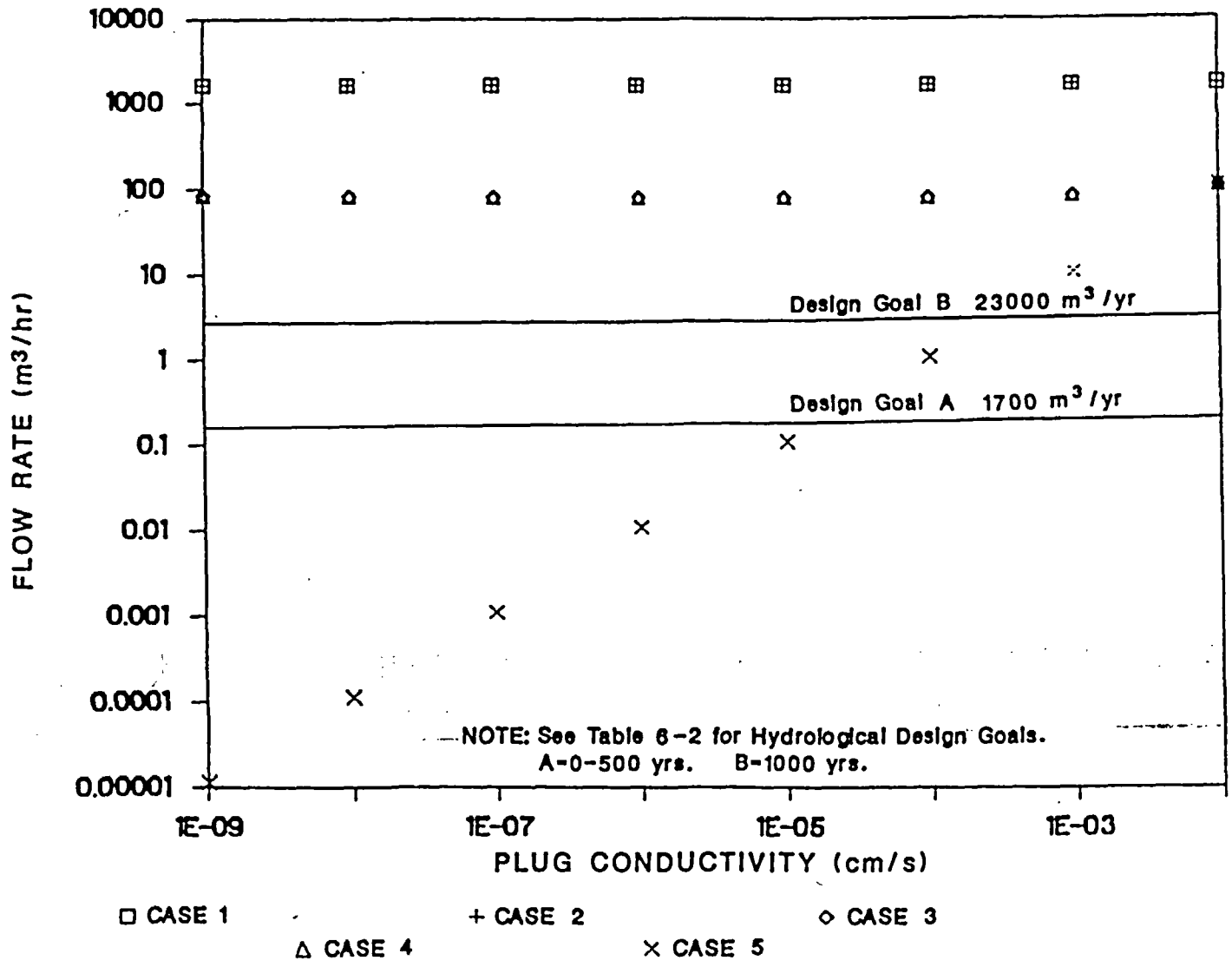


Figure F-4. Design Chart for a Seal/Plug Placed Near the Top of a Shaft

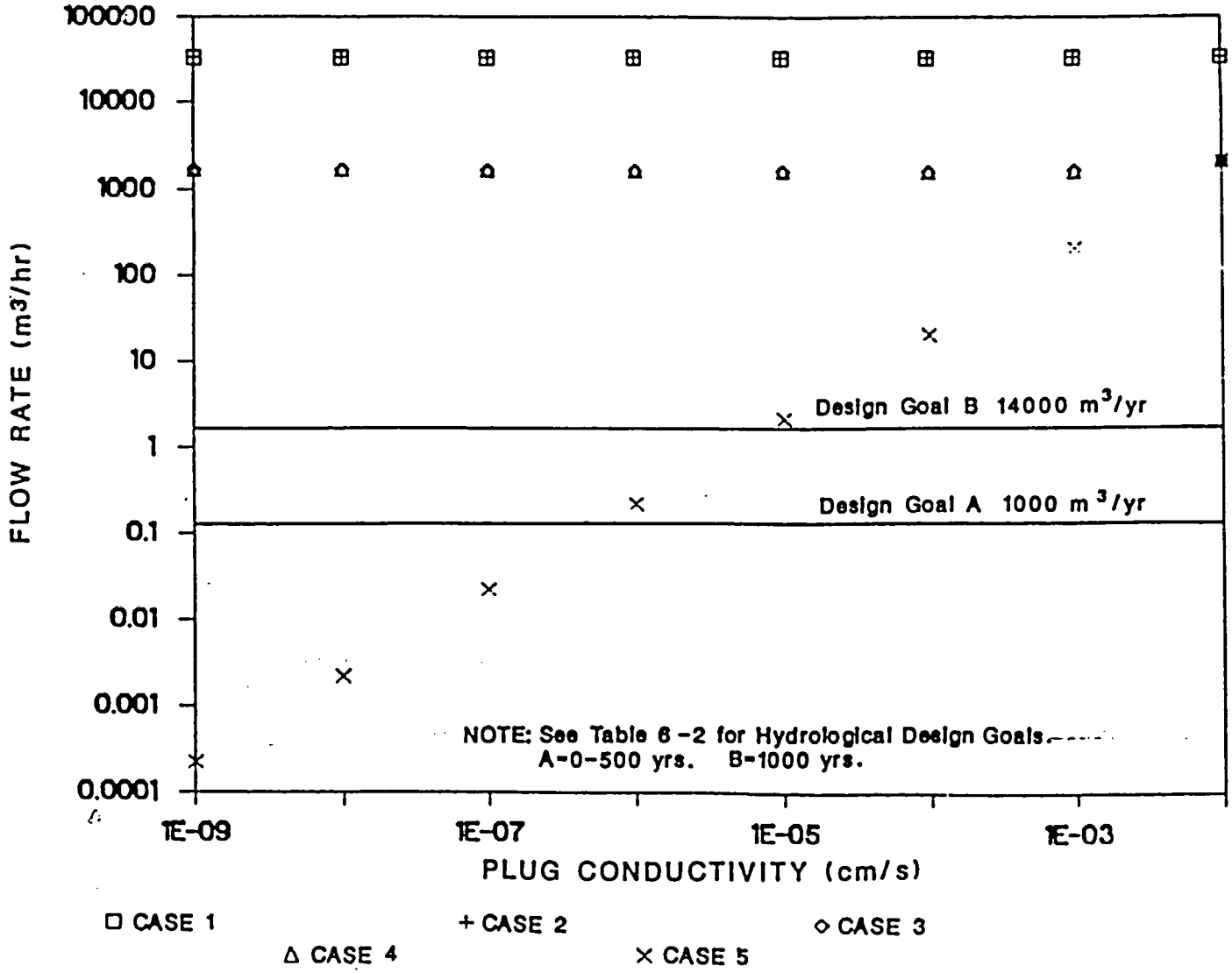


Figure F-5. Design Chart for a Seal/Plug Placed at the Base of a Ramp

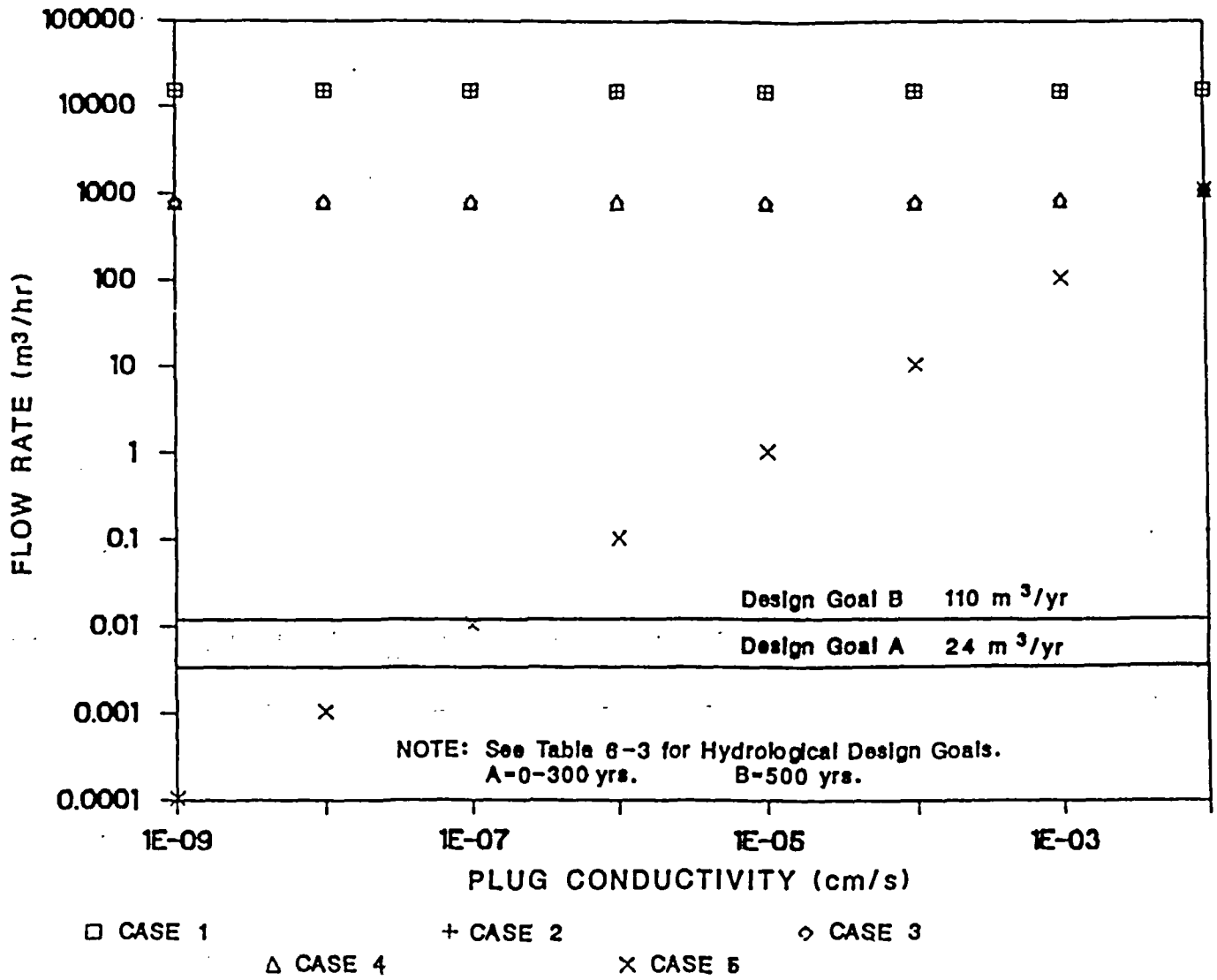


Figure F-6. Design Chart for a Double Bulkhead Placed in an Emplacement Drift

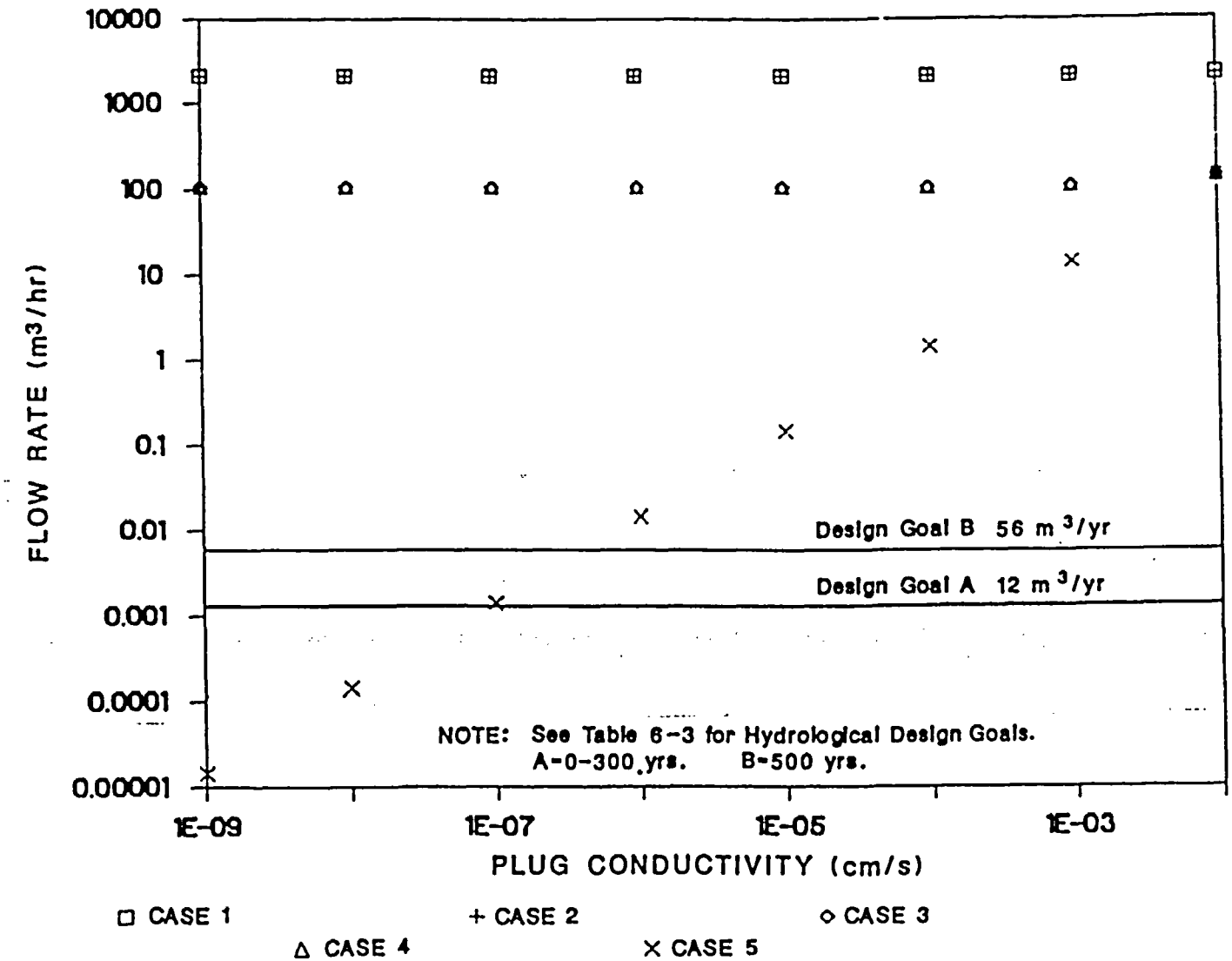


Figure F-7. Design Chart for a Plug Placed in an Emplacement Borehole

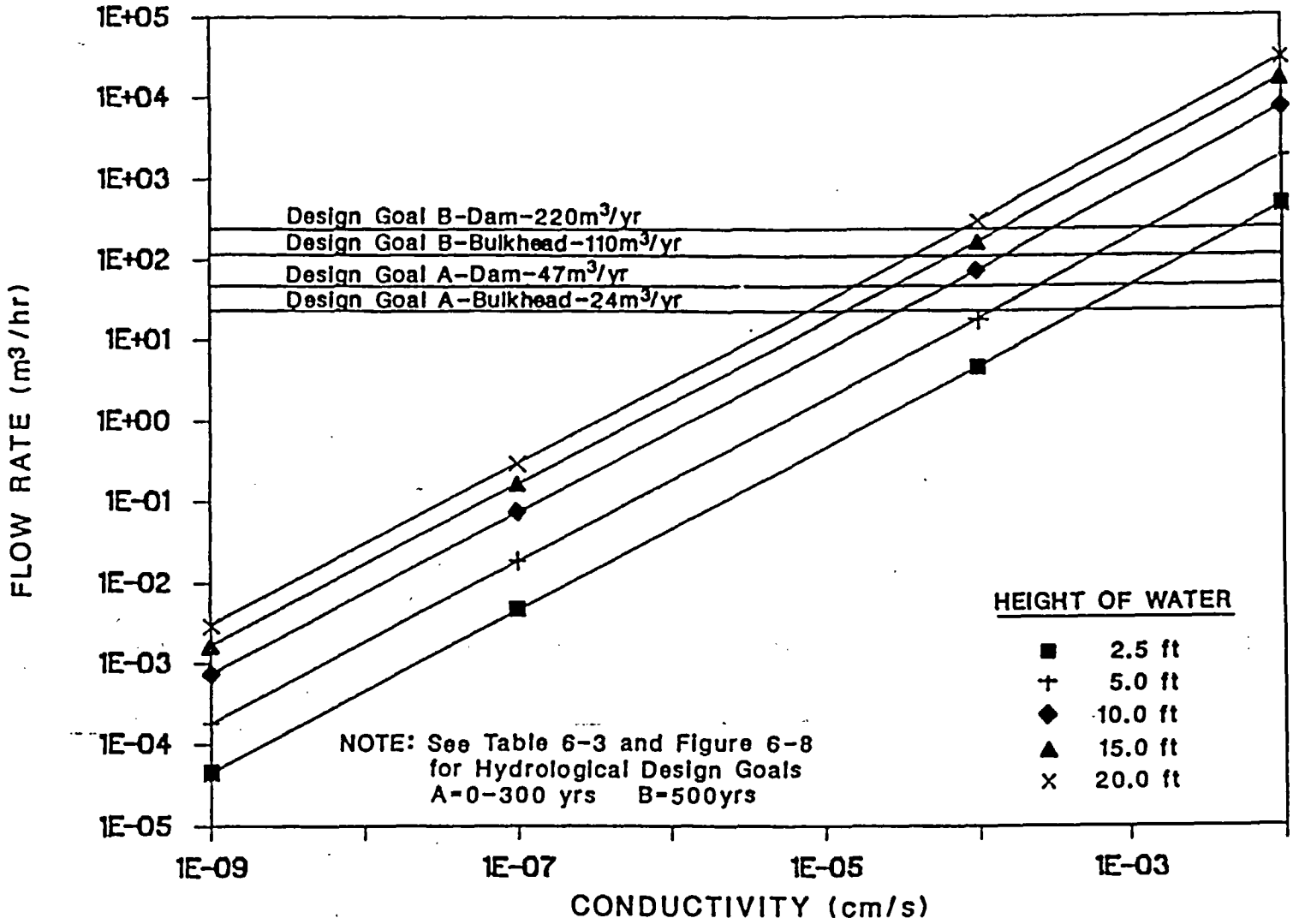


Figure F-8. Design Chart for a Dam or a Single Bulkhead with Settlement Placed in an Emplacement Drift

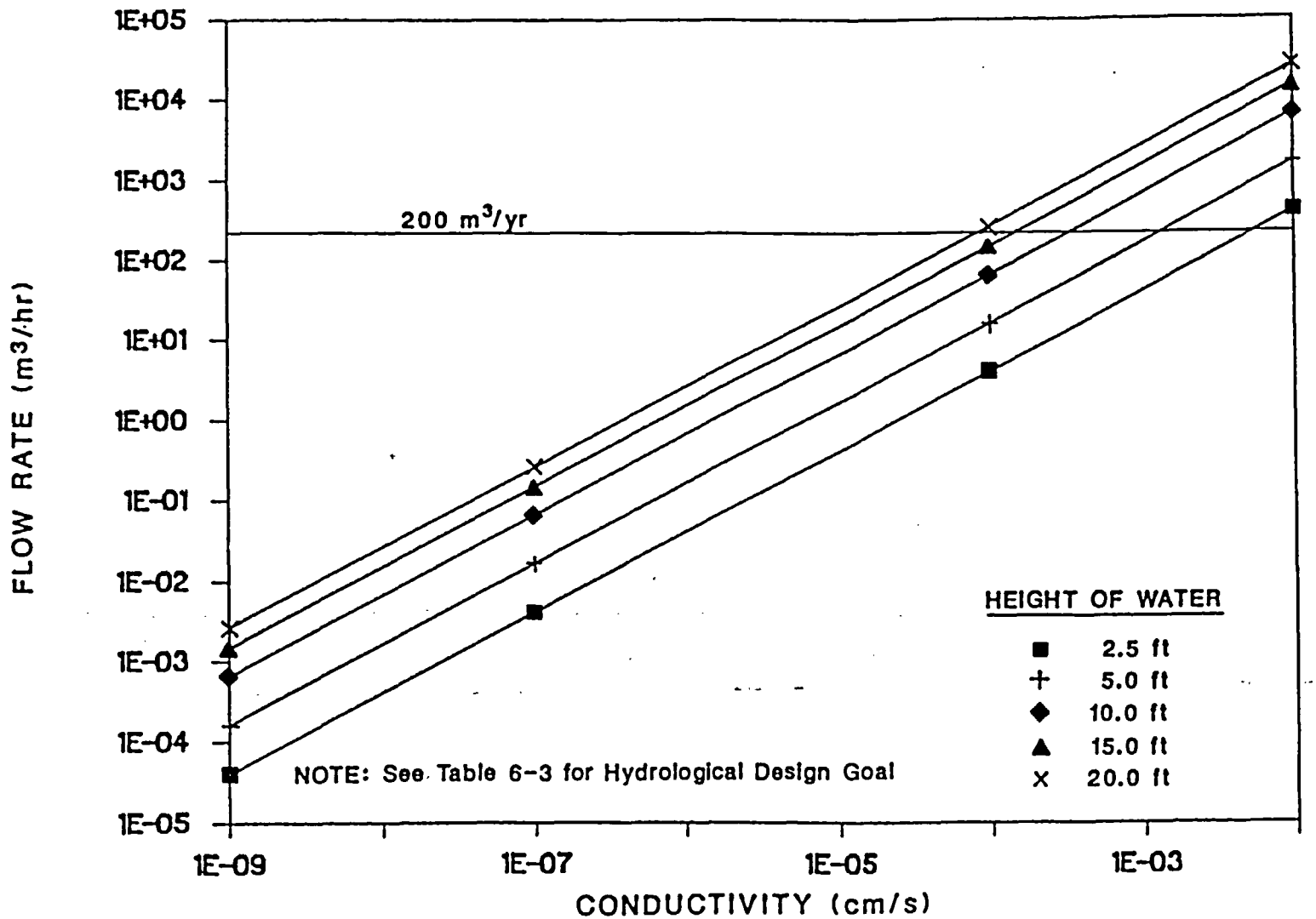


Figure F-9. Design Chart for a Dam Placed in the Perimeter and Main Drift

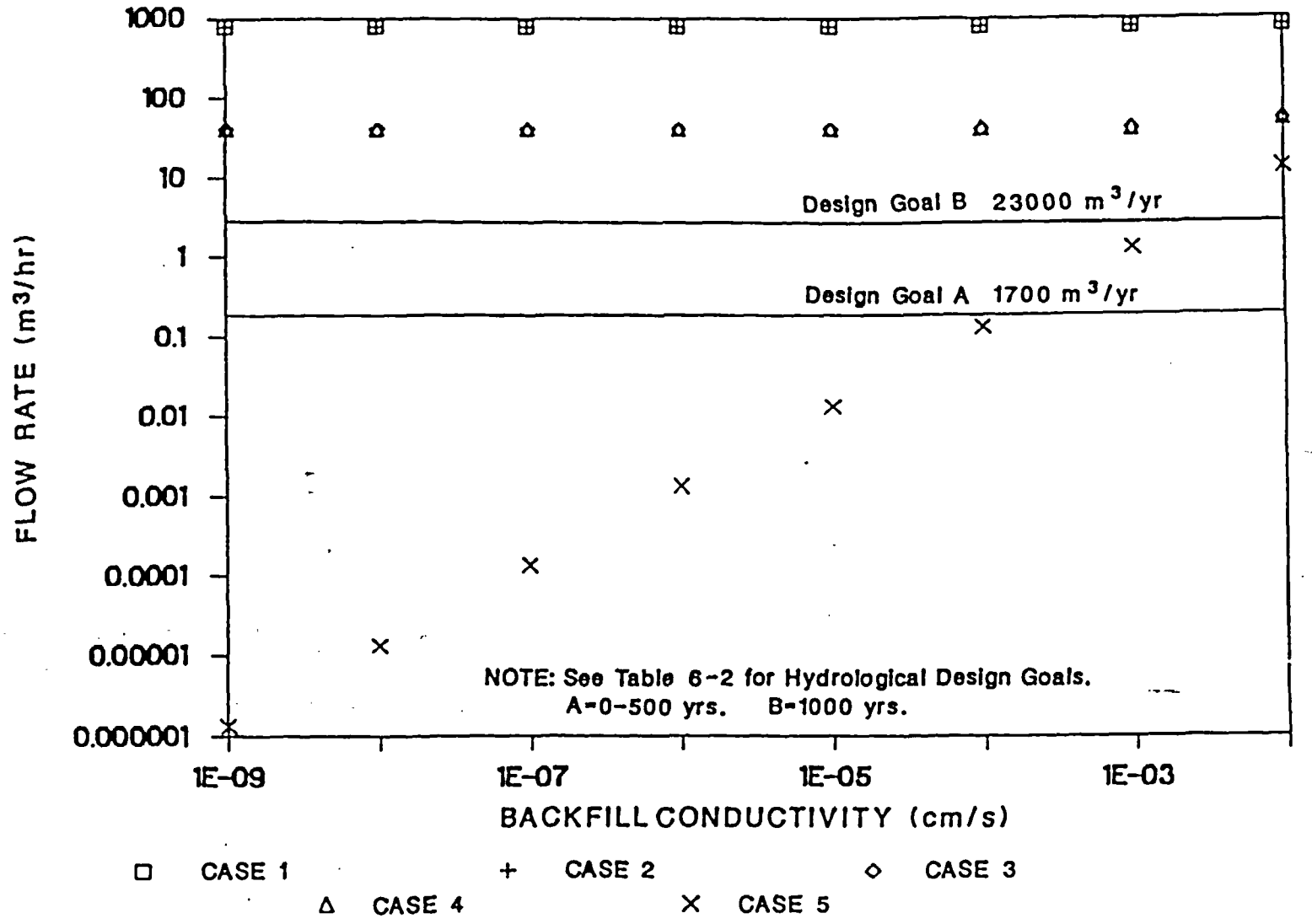


Figure F-10. Design Chart for Shaft Fill

Figure F-7 Emplacement Borehole Plug
Plug length, L = 1.7 m
Plug diameter, D = 1 m
Water-column height = 300 m
Host rock, hydraulic conductivity = 10^{-2} cm/s

Figure F-8 Emplacement Drift Dam
Dam length, L = 9.8 m
Dam height, H = 6.1 m
Water depth = 0.18 - 6.1 m
Host rock, hydraulic conductivity = Impermeable

Figure F-9 Perimeter Drift Dam
Plug length, L = 14.0 m
Dam height, H = 6.1 m
Water depth = 0.8 - 6.1 m
Host rock, hydraulic conductivity = Impermeable

Figure F-10 Shaft Fill
Backfill length, L = Entire length of the shaft
Shaft diameter, D = 6.9 m
Host rock, hydraulic conductivity = 10^{-2} cm/s

For the confined flow models, flow is assumed to occur through the plug, through the interface zone between the plug and the host rock, and through an annulus around the shaft one radius wide. This annulus represents the zone in which the host rock permeability might be modified by the excavation process or, alternatively, a zone in which the host rock might be replaced by a seal material with lower permeability. Such a zone of low-permeability material would have the effect of forcing flow out into the host rock away from the seal. In these analyses, the flow does not reenter the penetration downstream from the seal. This is considered to be a reasonable assumption given the rock adjacent to the seal is unsaturated.

Five cases are presented for each design chart (Figures F-3 through F-7 and Figure F-10) representing different modified permeability zones (MPZs) and interface conductivities:

Case 1. MPZ conductivity 20 times the host rock (expected condition for welded tuff obtained by Case and Kelsall, 1987) combined with interface conductivity from laboratory test on borehole seals conducted by Terra Tek (Lingle and Bush, 1982).

Case 2. MPZ conductivity 20 times the host rock, combined with interface conductivity 10 times the value from the Terra Tek test.

Case 3. MPZ conductivity equal to the host rock conductivity, with interface conductivity from the Terra Tek test.

Case 4. MPZ conductivity equal to the host rock conductivity, with interface conductivity 10 times the value from the Terra Tek test.

Case 5. MPZ replaced by seal material, no interface flow.

From an examination of Figures F-3 through F-7, it can be seen that the interface has essentially no effect on the results relative to the effect of the MPZ. Moreover, the results from the same cases show that it is necessary to reduce the hydraulic conductivity of the MPZ to achieve the design goals superimposed on the charts (obtained from Chapter 6). In Case 5, the effective hydraulic conductivity of the modeled zone is reduced by replacing the rock within the MPZ with a low-permeability, seal material. It is acknowledged that construction of a keyway of this type will probably result in propagation of the MPZ away from the shaft, ramp, and drift as a result of additional stress relief. Nonetheless, for the case of a shaft, the keyway would serve to divert water away from the shaft. It is also noted that the effect of the keyway might also be achieved by grouting.

The unconfined flow models are simple dams that impound water (Figure F-2) or vertical infiltration under saturated conditions and unit gradient (Figure F-2). In the case of dams, it is assumed that a phreatic surface develops through the dam and that flow may be predicted by the Dupuit equation (Todd, 1980, p. 113). No leakage is assumed to occur through the MPZ or underlying rock. In the case of shaft fill, the same cases (with the exception of Case 5) are evaluated with flow occurring vertically downward under unit gradient. Instead of Case 5, Case 6 is evaluated, which assumes that an MPZ does not develop or that the host rock is relatively impermeable compared to the shaft fill. The design curves in Figure F-10 indicate that the use of shaft fill would not be effective if flow through a seal system (including an MPZ and interface zone) exceeds the design goal.

APPENDIX G

SATURATED AND UNSATURATED, HYDRAULIC PROPERTIES OF SELECTED CEMENTITIOUS-BASED MATERIALS (GROUT AND CONCRETE)

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Saturation and Relative, Hydraulic Conductivity Curves for

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Permeability Versus Pressure-Head Curve for

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APPENDIX G
SATURATED AND UNSATURATED, HYDRAULIC PROPERTIES OF SELECTED CEMENTITIOUS-BASED MATERIALS (GROUT AND CONCRETE)

This appendix includes the saturated and unsaturated, hydraulic properties of two grouts (82-022 and 82-030) and one concrete (CL-40-CON). The data contained in this appendix are intended to be used to determine the hydrologic flow through cementitious-based materials in the unsaturated-tuff environment at Yucca Mountain. Further, these grouts and concretes represent cementitious-based materials that potentially could be used as sealing components. Both 82-022 and 82-030 were selected for further testing following an initial screening of other expansive and nonexpansive grouts and mortars (Licastro et al., in preparation). Both of these samples are expansive to permit a more adequate seal between the grout/mortar and the host rock. The concrete selected for testing included tuff as a coarse aggregate.

The constituents of the grouts and concrete are defined in Tables G-1 and G-2. The primary difference between these two grouts is the addition of sand (ASTM C 109) in sample 82-022. This addition would classify sample 82-022 as a mortar and the lack of this sand in sample 82-030 as a neat cement. Another difference is the manner in which expansion is achieved in both samples. In sample 82-030, hemihydrate (Gypseal) was added to the mixture. In sample 82-022, expansion was achieved by the development of ettringite through the addition of a type K, shrinkage-compensating cement (ChemComp). The mixture of CL-40-CON contained an expansive cement to achieve expansion of the sample. Samples 82-022 and 82-030 were cured at 38°C for 482 and 502 days, respectively. CL-40-CON was cured at 21°C for 646 days.

To determine the unsaturated, hydraulic properties of these materials, Sandia National Laboratories requested Pacific Northwest Laboratories to perform laboratory tests to determine the water-retention characteristics and the saturated hydraulic conductivity of samples 82-022, 82-030, and CL-40-CON. A number of subsamples were cut from the large core to enable multiple testing of each sample. Three subsamples were taken from each grout sample, and six subsamples were taken from the concrete sample. All tested subsamples were 1.5 cm x 1.2 cm (diameter times height). The hydraulic conductivity was determined using a Ruska Liquid Permeameter. The water-retention characteristics were determined through the use of vapor pressure, equilibrium techniques. The details of these techniques are provided by Peters et al. (1984).

The laboratory data associated with the water-retention characteristics of each subsample are plotted as saturation versus pressure-head curves on Figures G-1 to G-16. The identification (ID) number for each data set analyzed is given in Table G-3. The laboratory-derived data for the saturated hydraulic conductivity are contained within Table G-4. The permeability of the subsamples (Table G-4) was computed using the equation (Freeze and Cherry, 1979, p. 27):

Table G-1. Composition of 82-022 and 82-030 (From Licastro et al., in preparation)

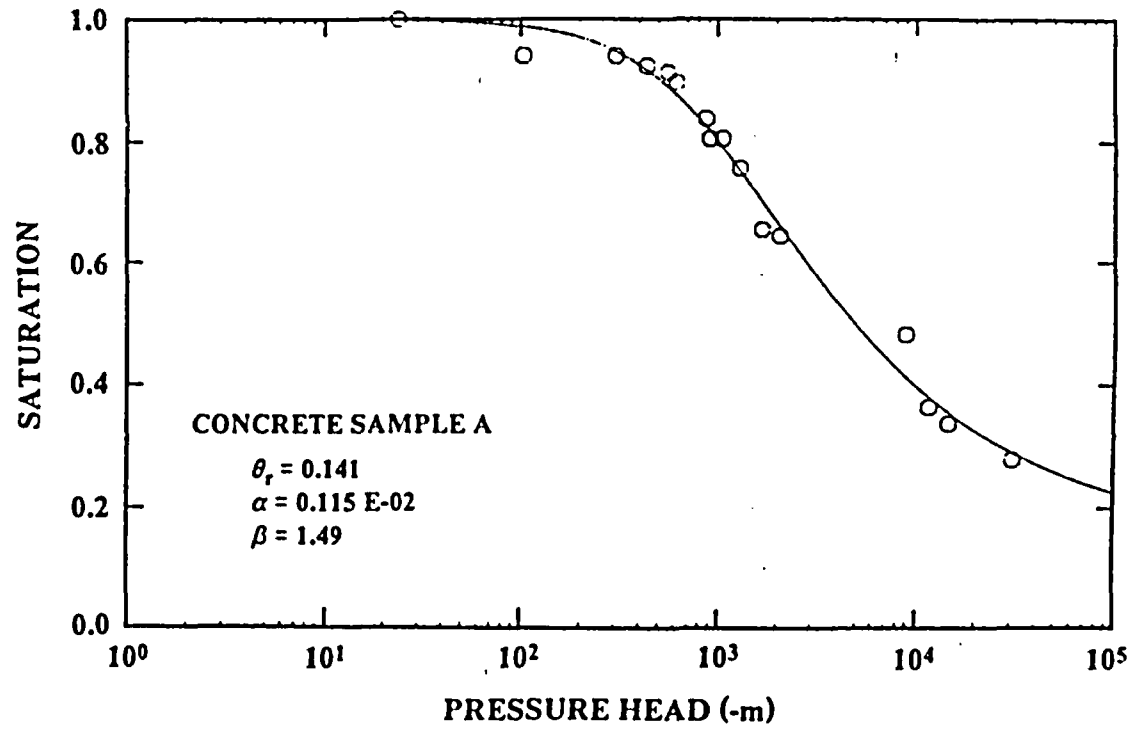
	Component[a]	Expansive Mixtures 82-022	82-030
Basic Mixture	Type K cement (ChemComp)	33.83	--
	Class H cement	--	47.29
	Simulated, tuff ground water	--	20.32
	Deionized water	15.87	--
	Slag	--	--
	Silica fume	7.29	4.75
	Fly ash (low lime)	8.18	--
	Fly ash (high lime)	--	15.20
	Silica flour (5 micrometers)	--	4.82
	Sand (ASTM C 109)	33.83	--
	Granite sand	--	--
Admixtures	Hemihydrate (Gypseal)	--	6.48
	Dispersant - Mighty 150	1.00	--
	Dispersant - Dowell D65	--	1.12
	Defoamer - Dowell D47	0.005	0.02
Design Information	Water to cement ratio	0.47	0.43
	Water to cementitious materials ratio	0.32	0.28

[a] Values for each component are reported as weight percentage of the total mixture.

Table G-2. Composition of CL-40-CON (From Fernandez and Wong, in preparation)

	Materials	Concrete Trial Mixture No. 5[a]
Basic Mixture	Cement, kg (lb)	222 (490)
	Fly ash, kg (lb)	83.9 (185)
	Fine aggregate from the Nevada Test Site concrete sand, kg (lb)	684.5 (1,509)
	Coarse aggregate - tuff, kg (lb)	569.7 (1,256)
	Water, kg (lb)	122.5 (270.1)
Admixture	High-range, water reducer, kg (lb)	3.2 (7.0)
	Retarding densifier - plastifier, l (fl. oz.)	0.8818 (29.79)
Design Information	Water to cementitious materials ratio	0.4
	Sand to total aggregate ratio, % by volume	50

[a]Proportions are for 0.76 m^3 (1 yd^3). The original volume of the concrete, trial mixtures was about 0.014 m^3 (0.5 ft^3).



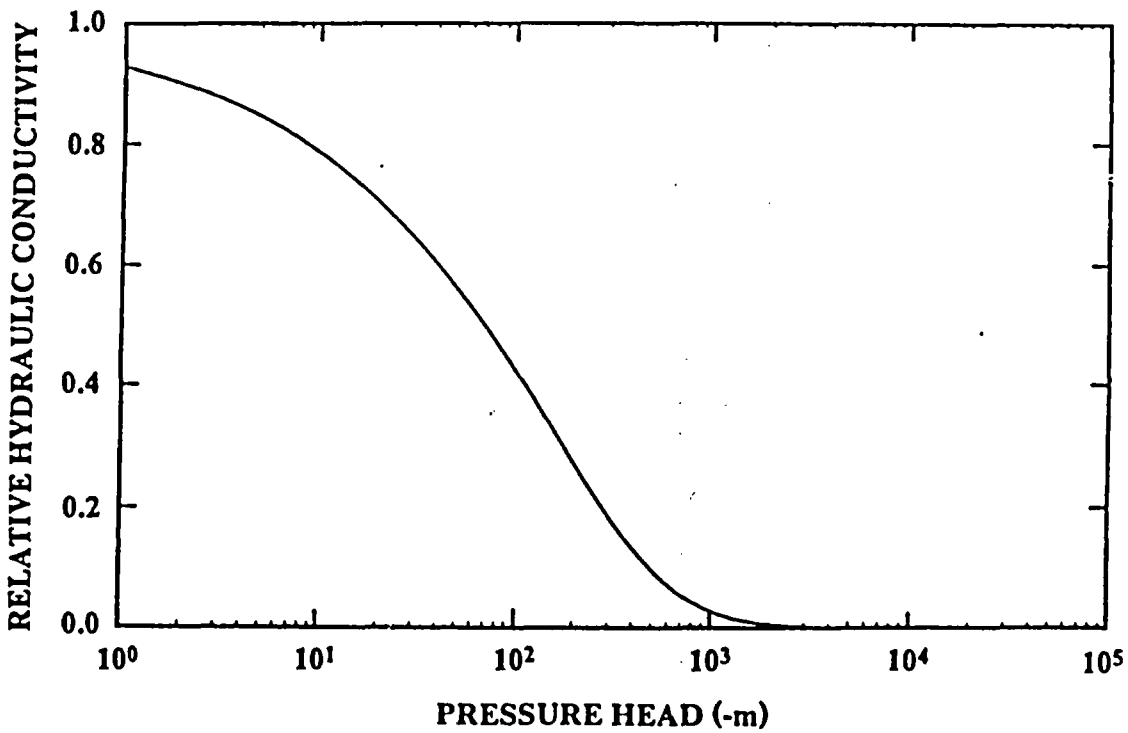
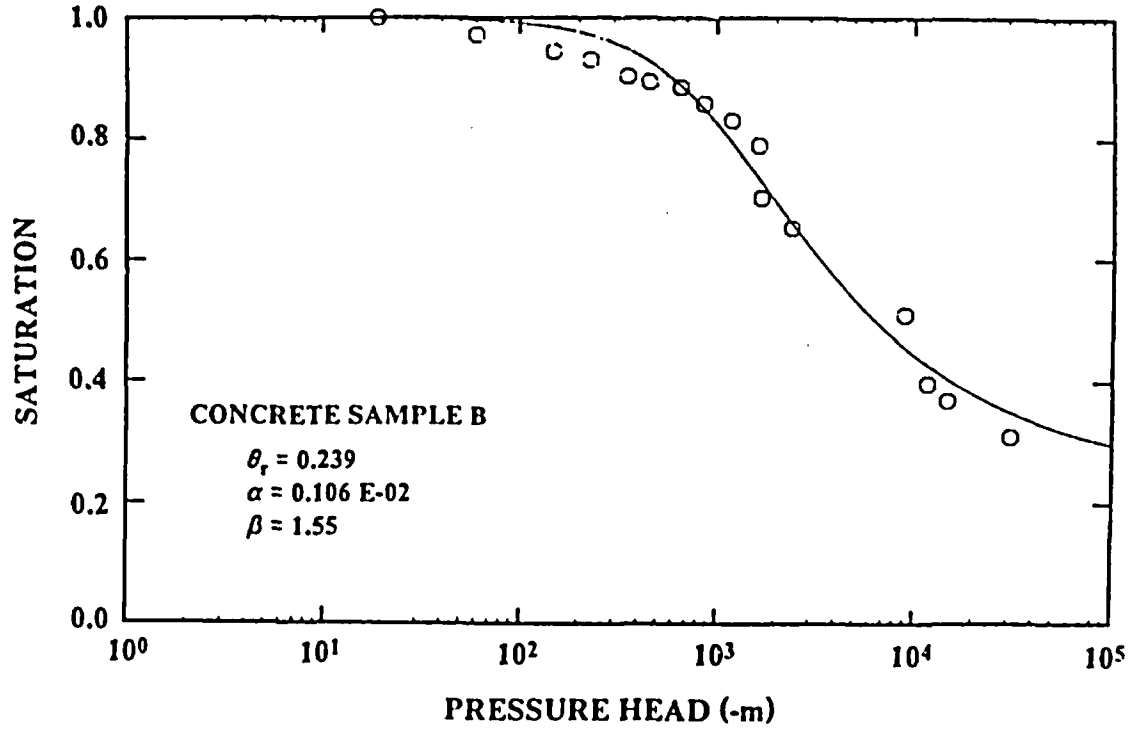


Figure G-1. Saturation and Relative, Hydraulic Conductivity Curves for Concrete Sample A



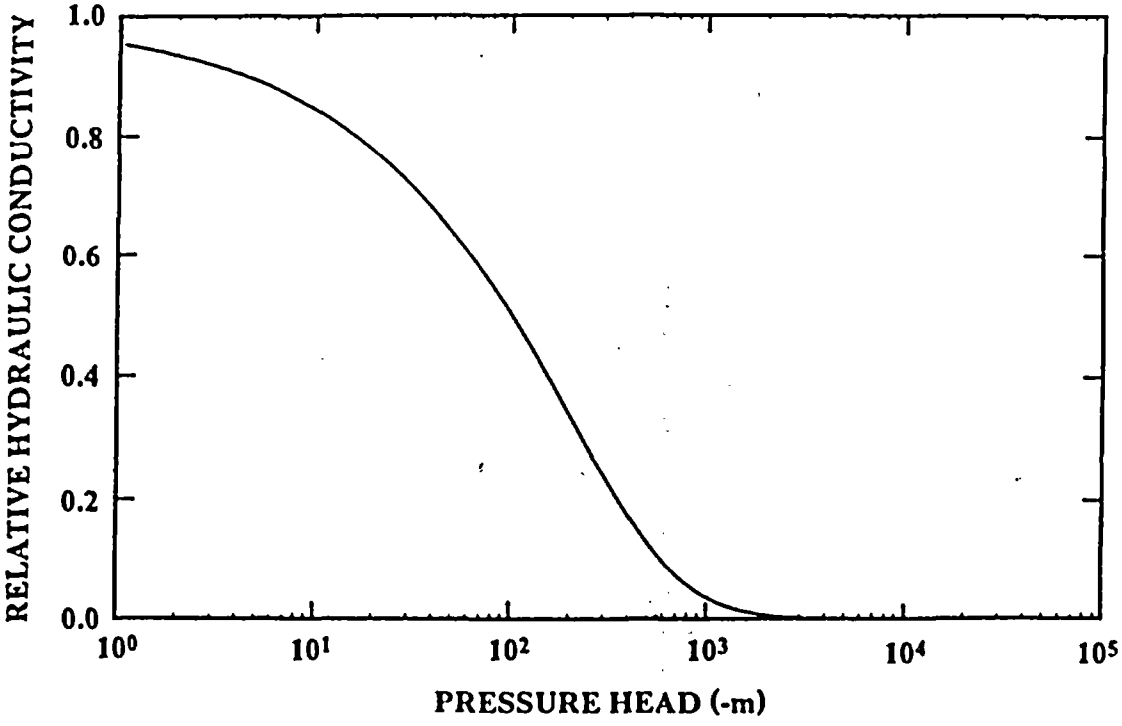
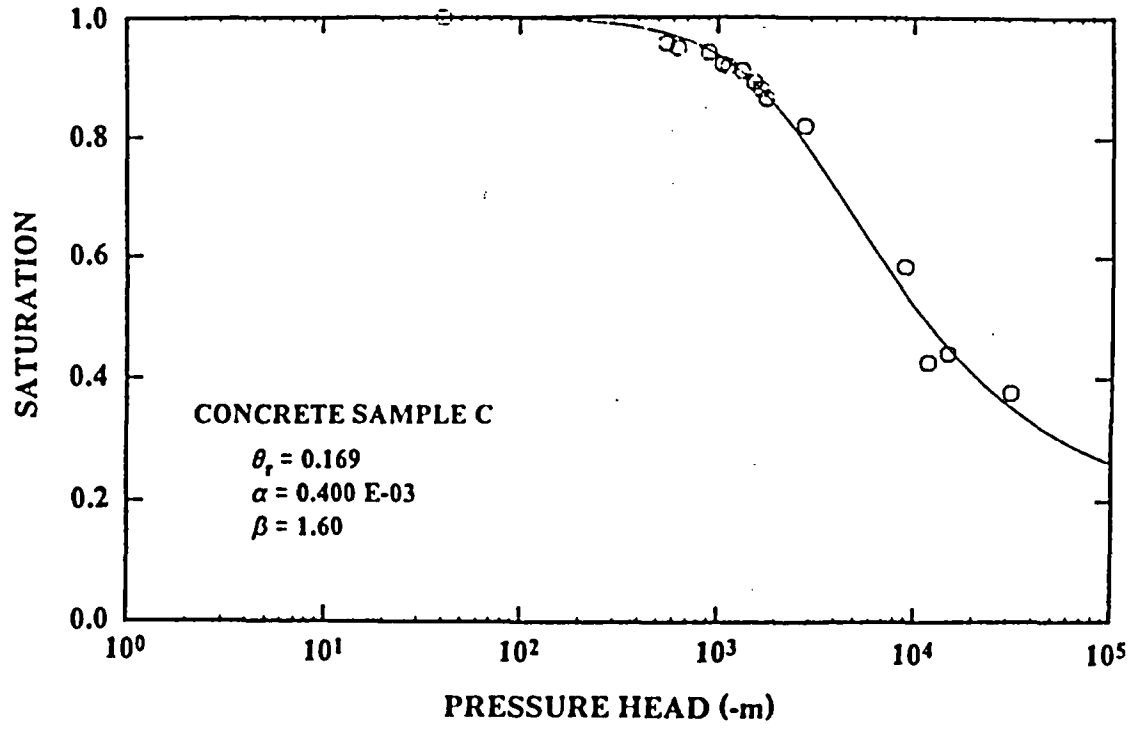


Figure G-2. Saturation and Relative, Hydraulic Conductivity Curves for Concrete Sample B



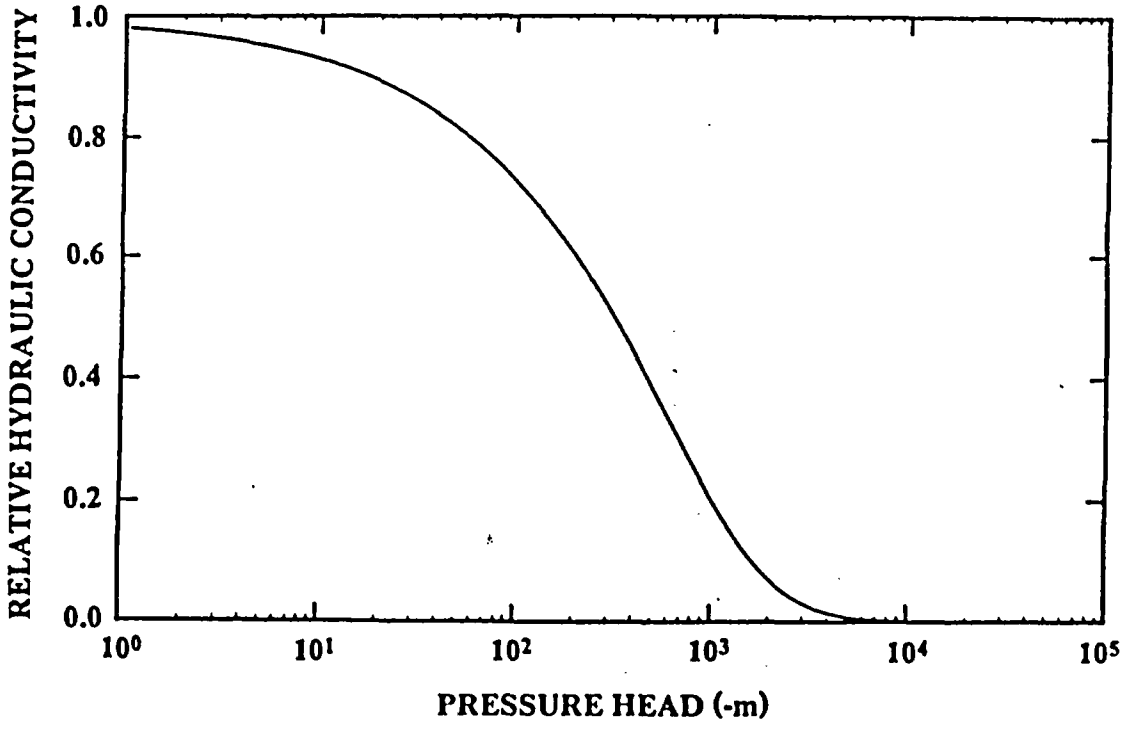
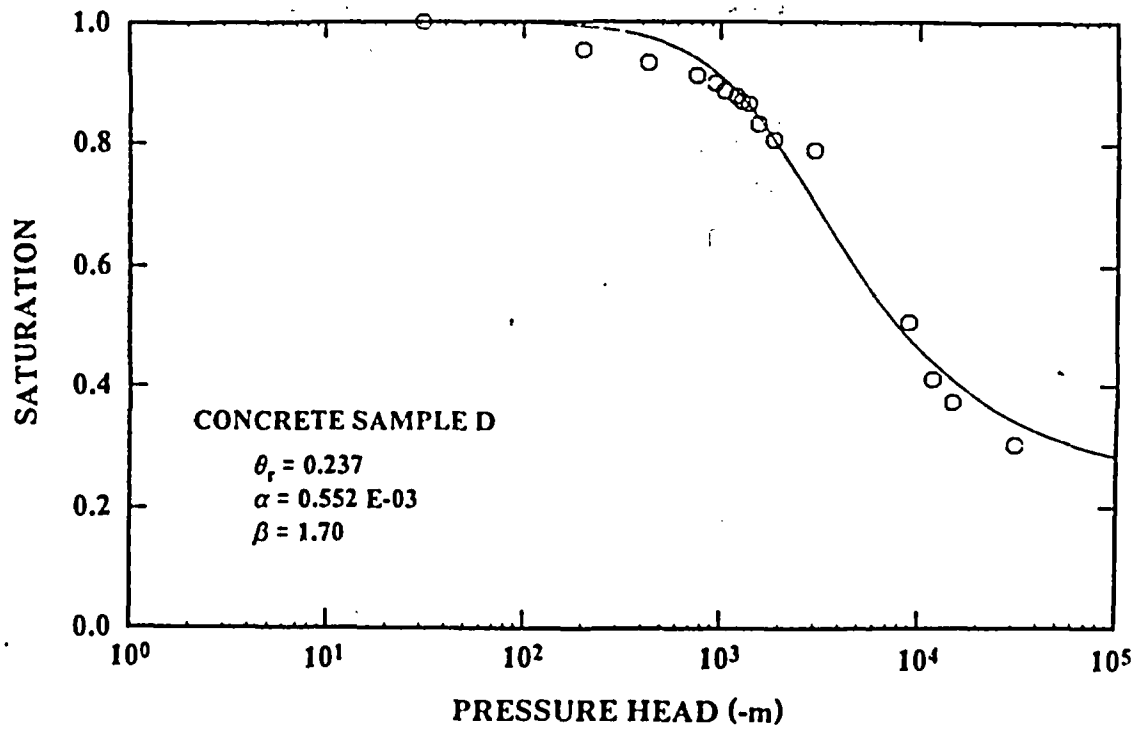


Figure G-3. Saturation and Relative, Hydraulic Conductivity Curves for Concrete Sample C



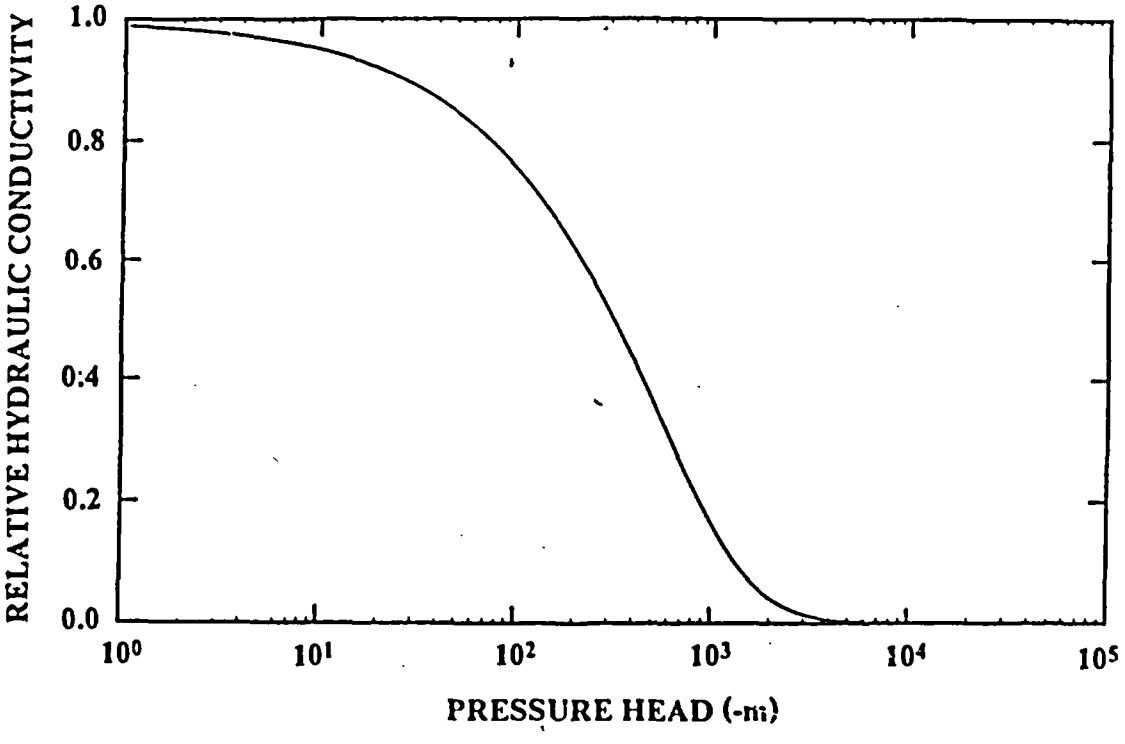
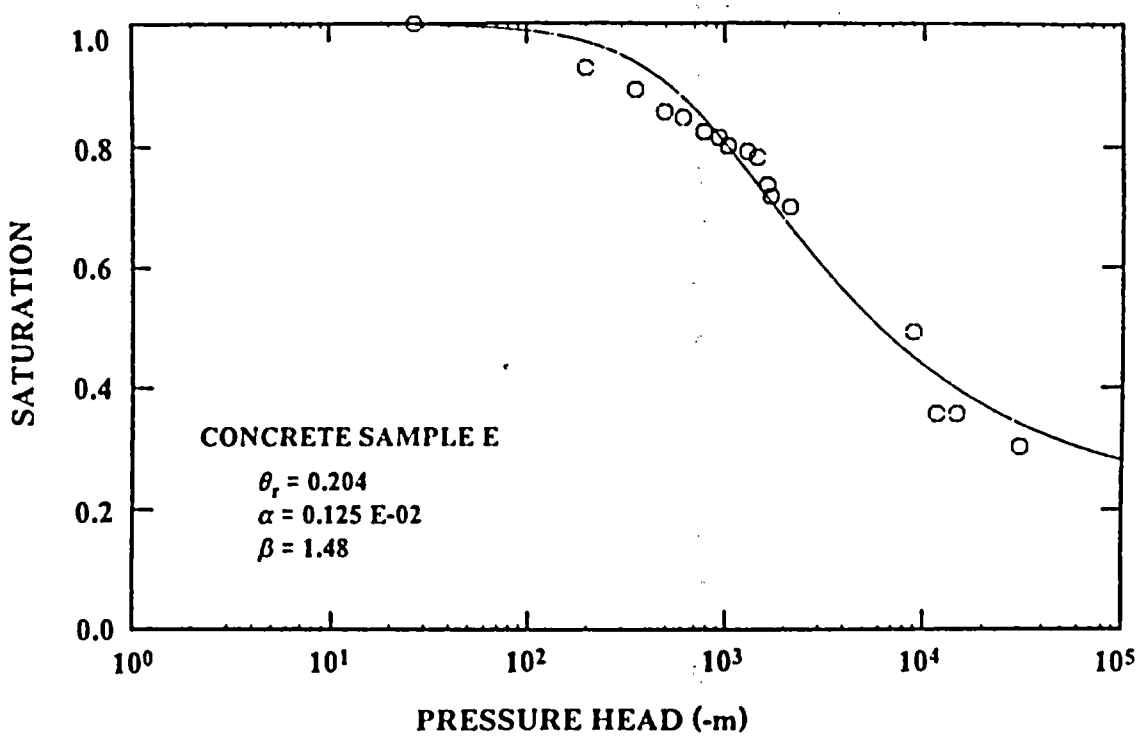


Figure G-4. Saturation and Relative, Hydraulic Conductivity Curves for Concrete Sample D



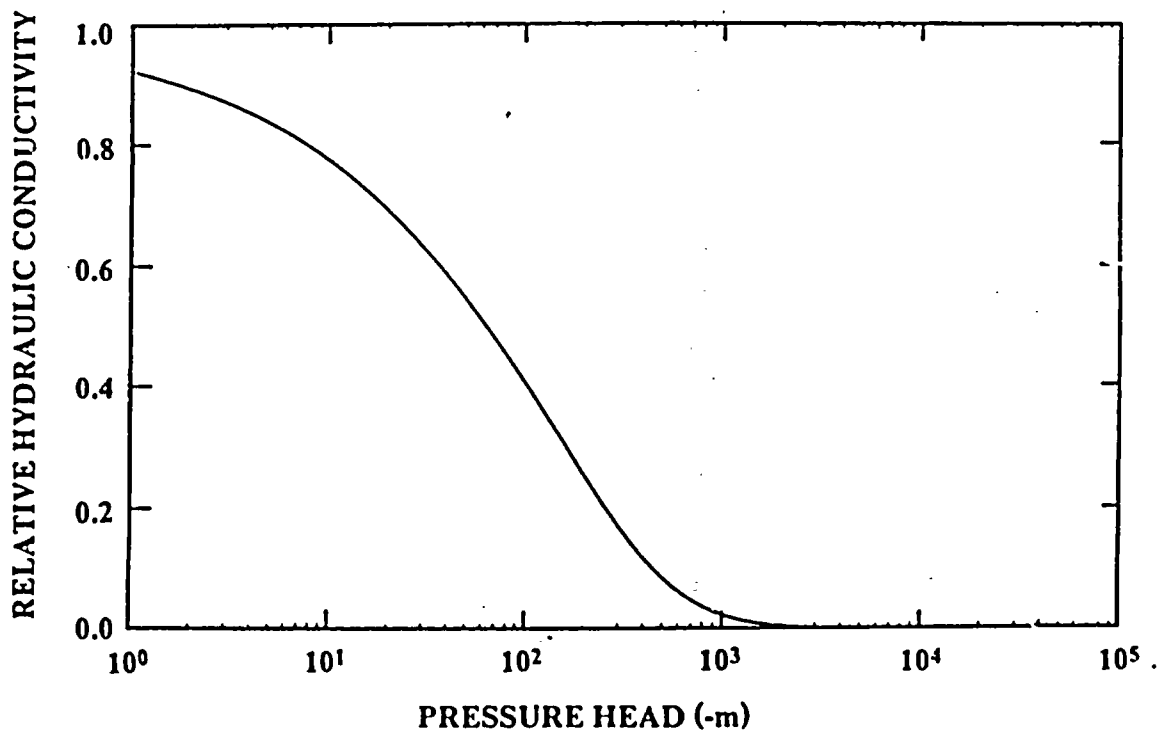
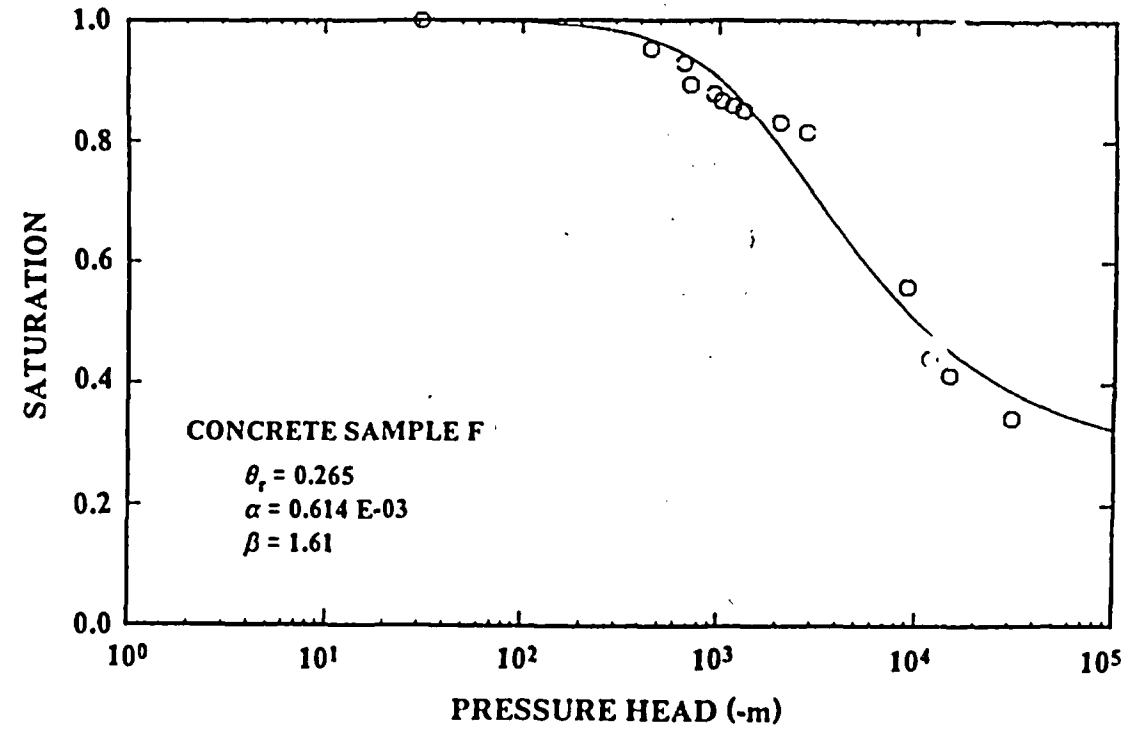


Figure G-5. Saturation and Relative, Hydraulic Conductivity Curves for Concrete Sample E



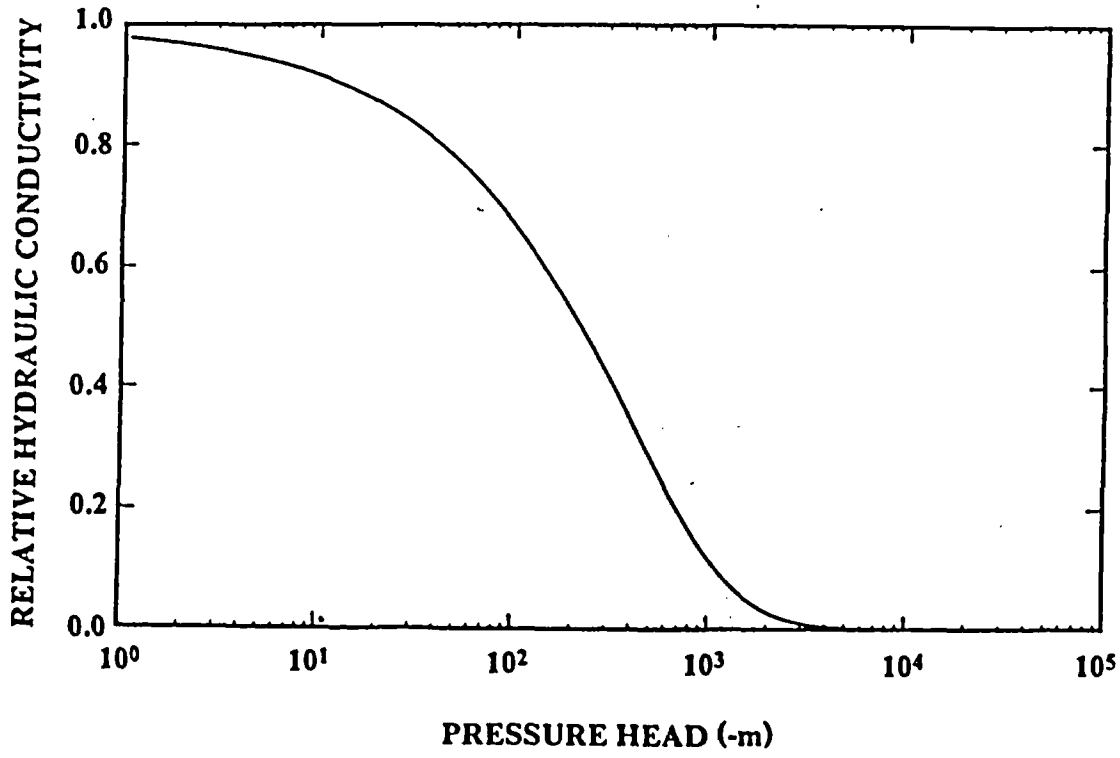
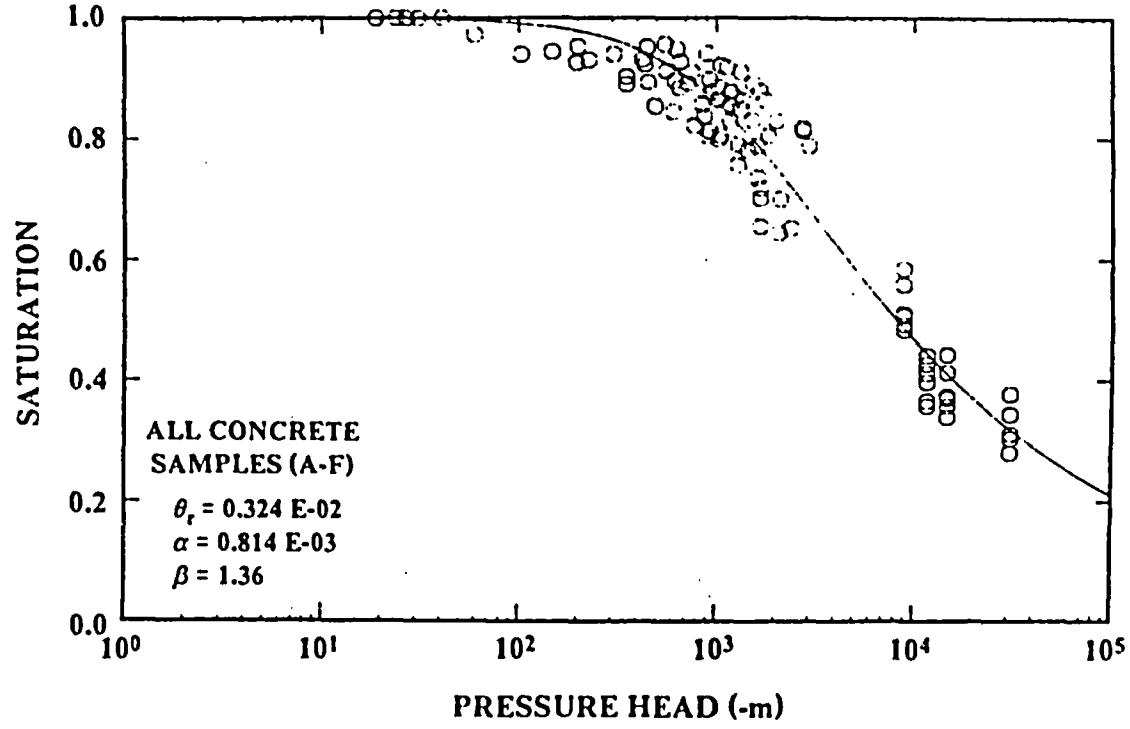


Figure G-6. Saturation and Relative, Hydraulic Conductivity Curves for Concrete Sample F



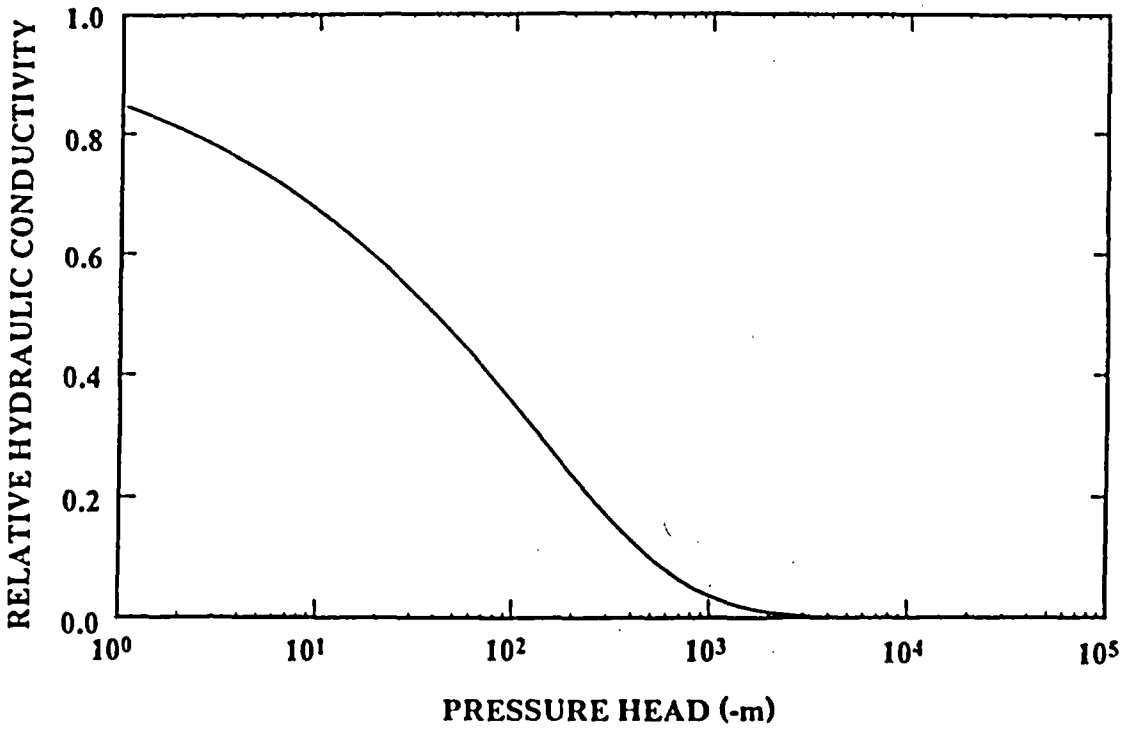
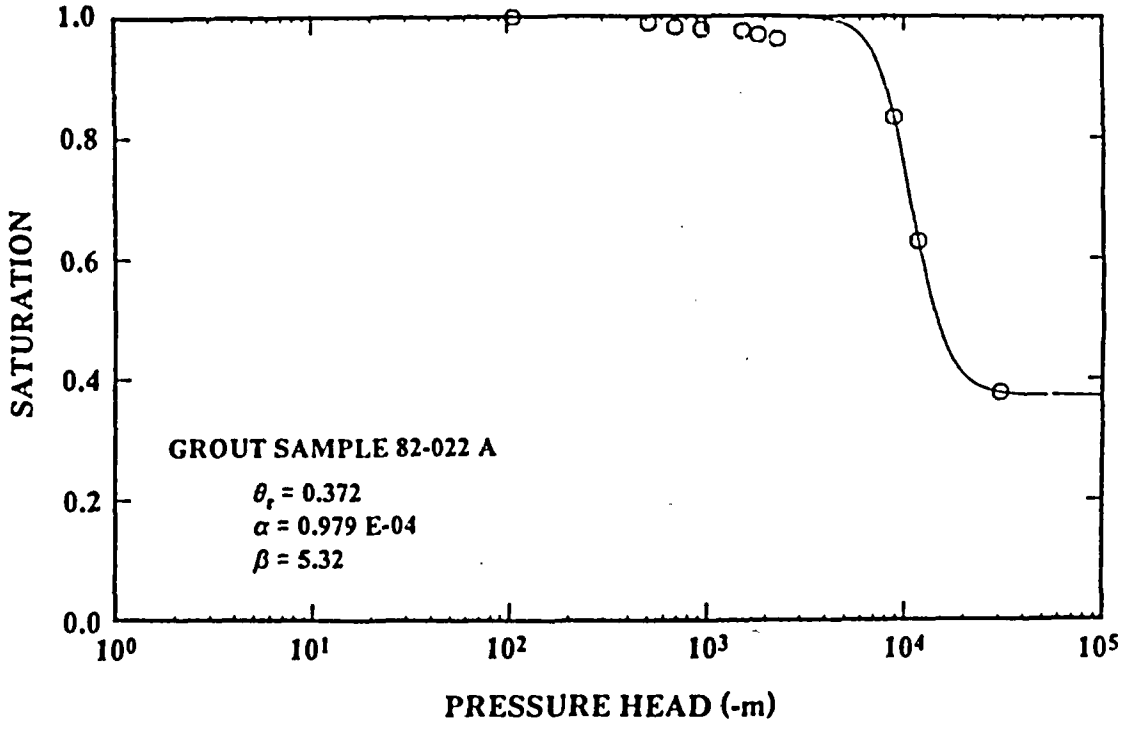


Figure G-7. Saturation and Relative, Hydraulic Conductivity Curves for All Concrete Samples



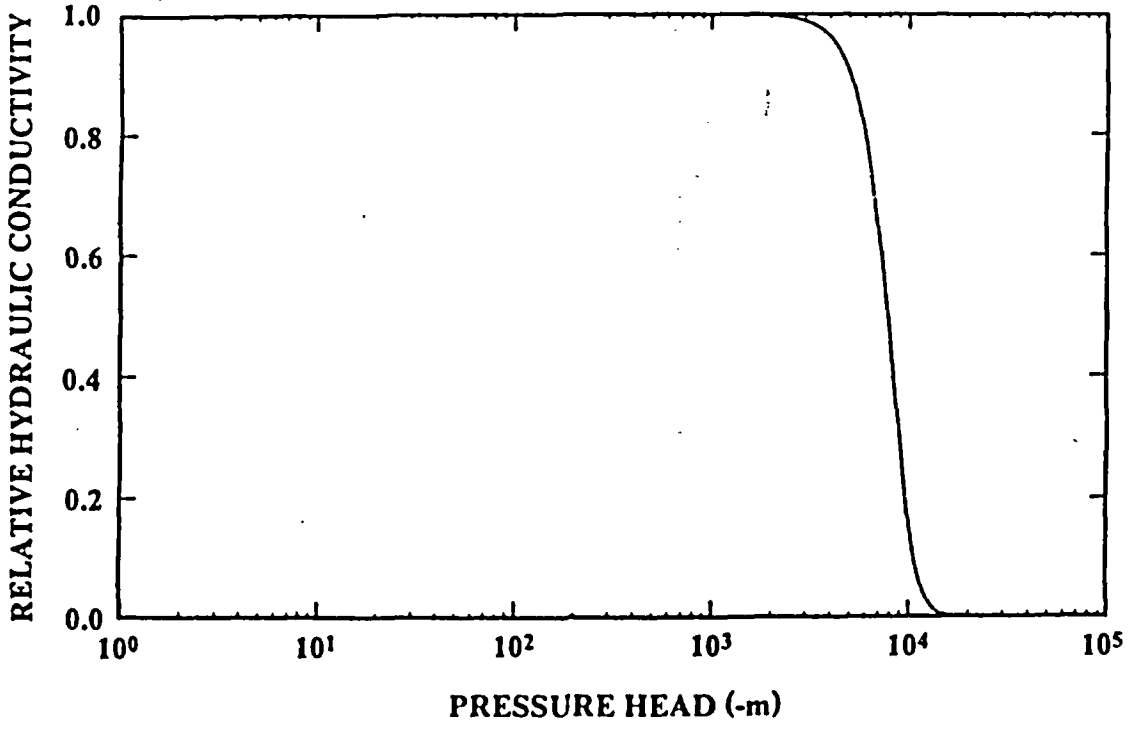
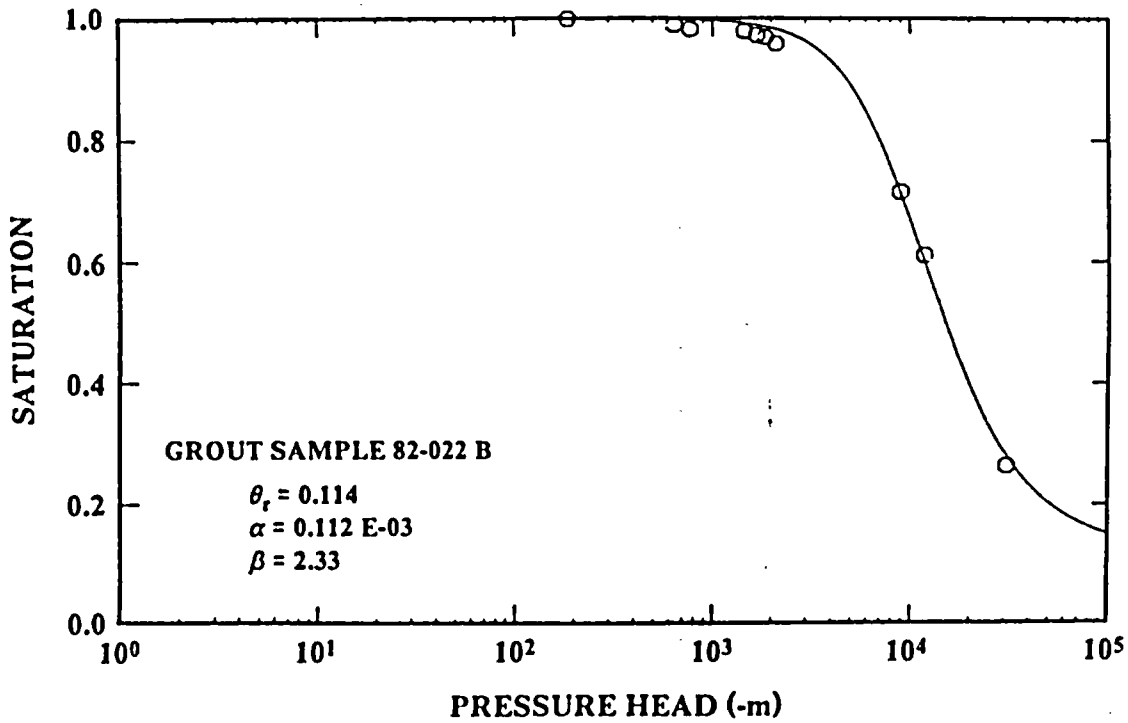


Figure G-8. Saturation and Relative, Hydraulic Conductivity Curves for Grout Sample 82-022A



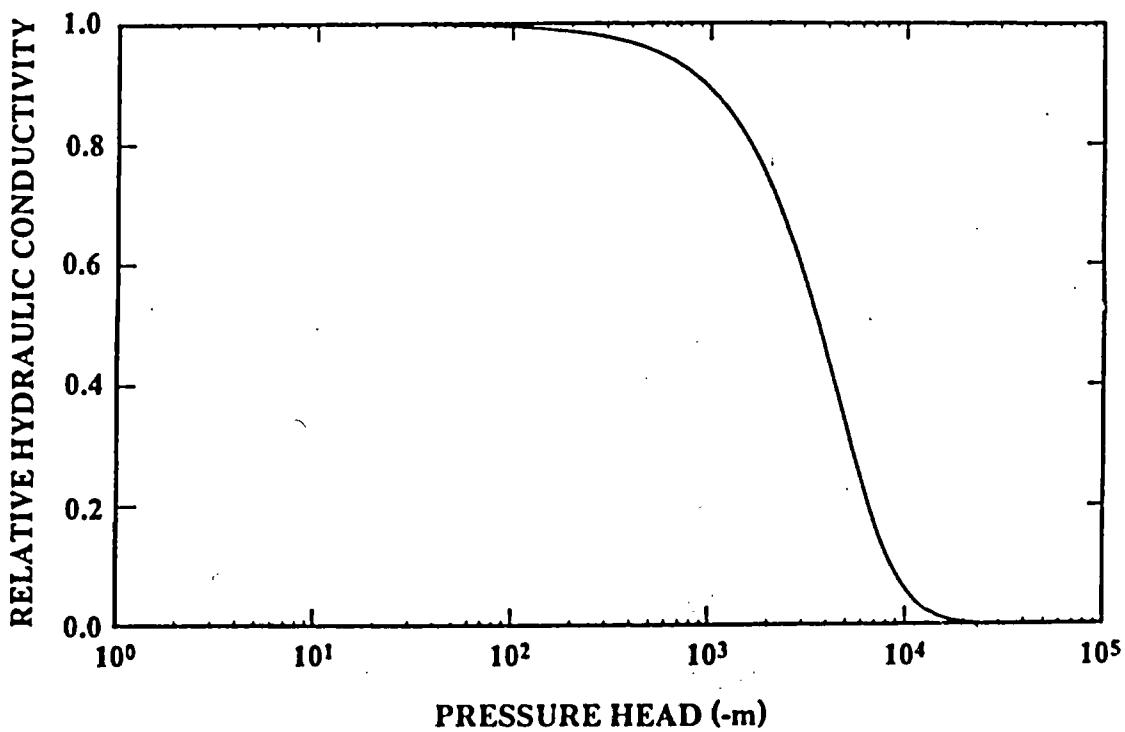
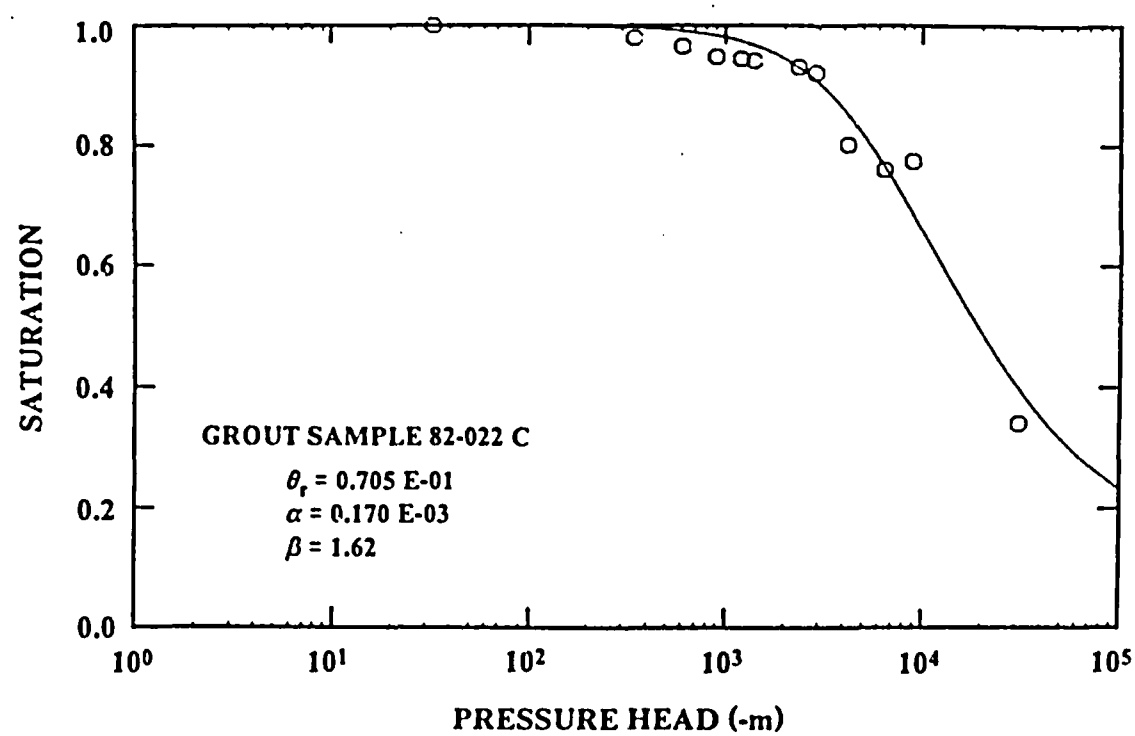


Figure G-9. Saturation and Relative, Hydraulic Conductivity Curves for Grout Sample 82-022B



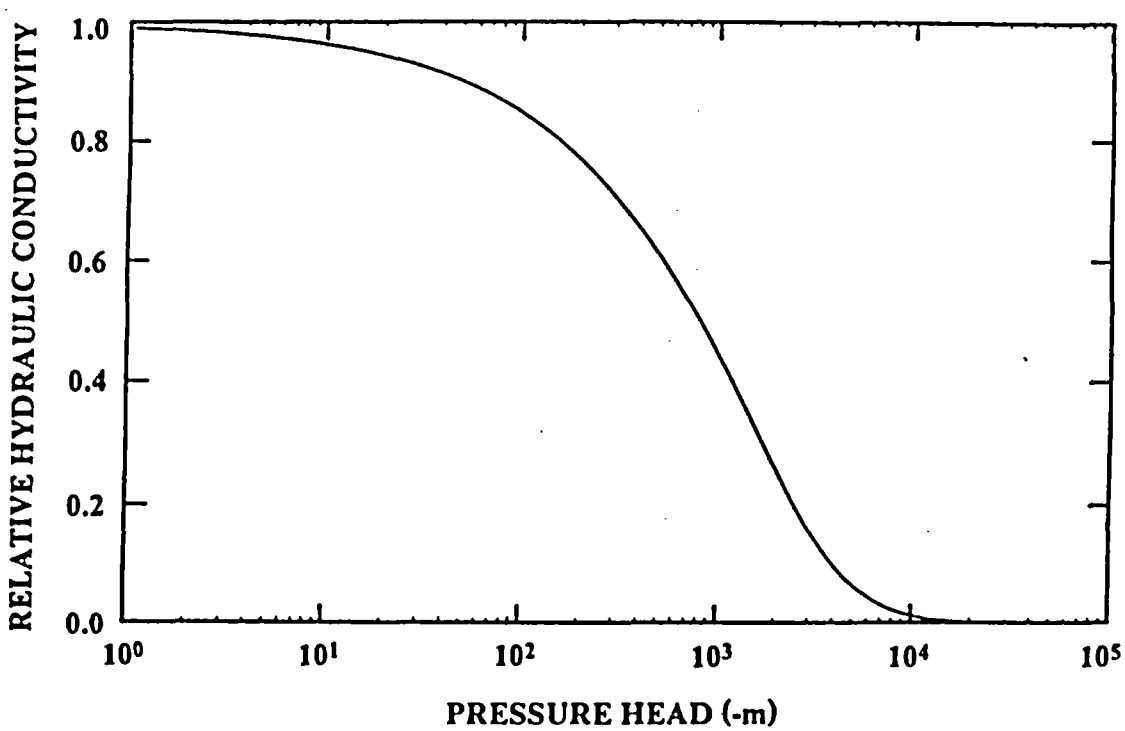
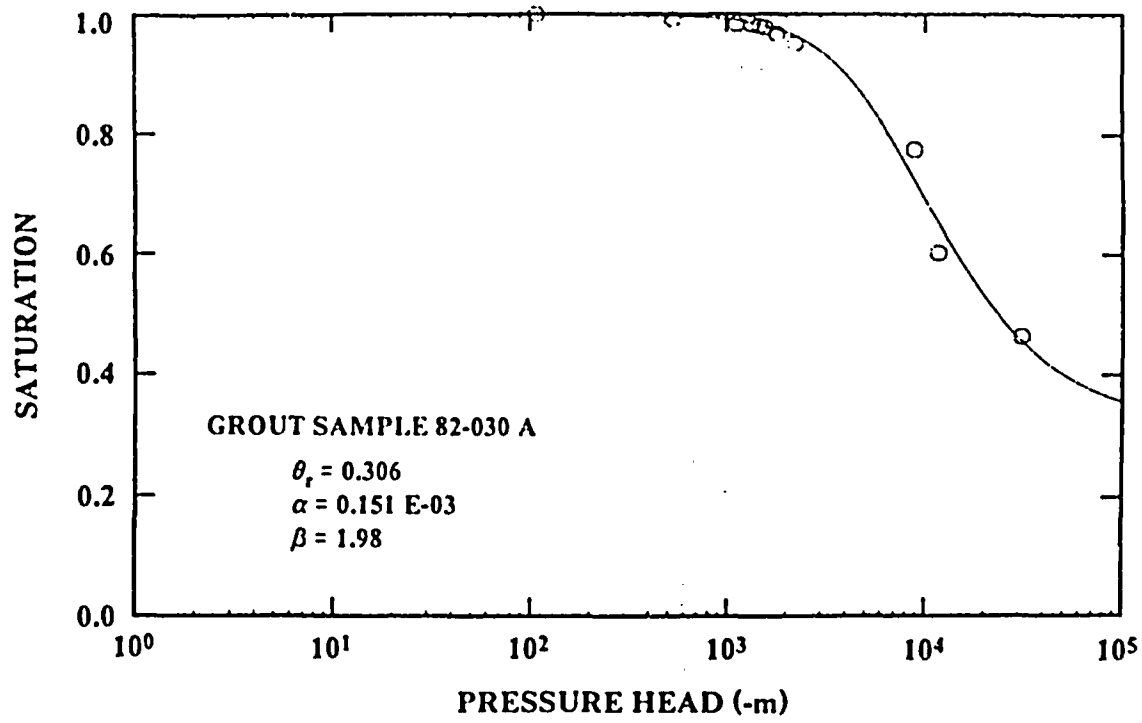


Figure G-10. Saturation and Relative, Hydraulic Conductivity Curves for Grout Sample 82-022C



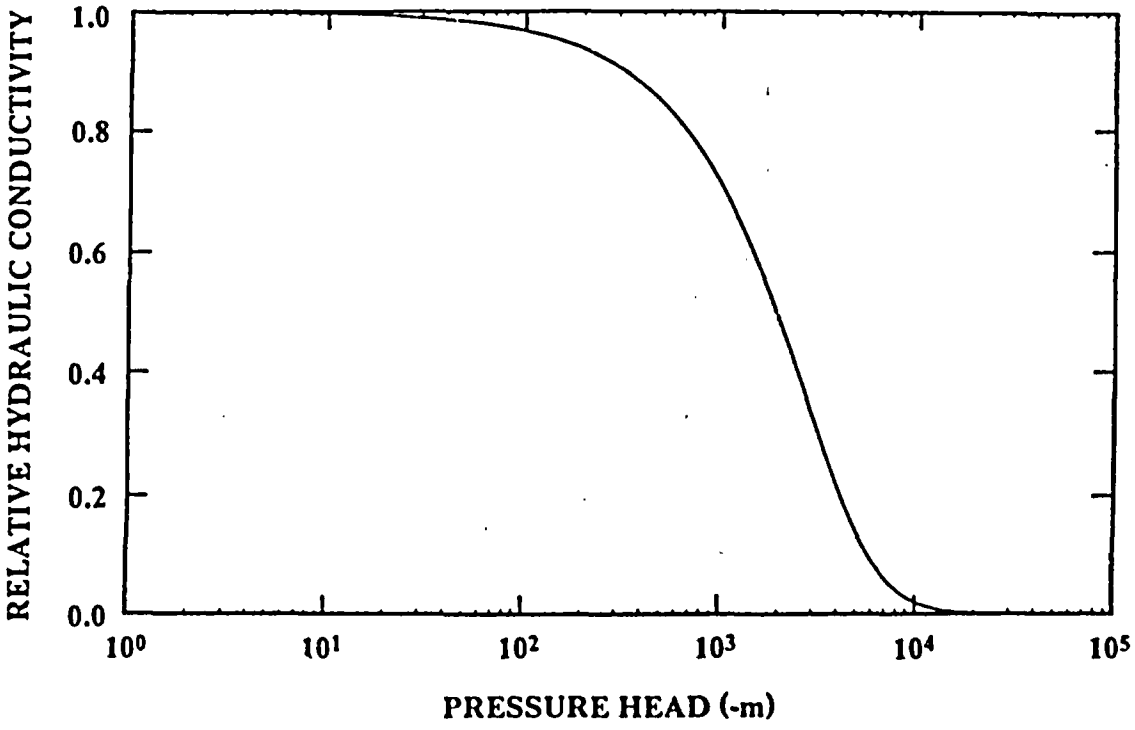
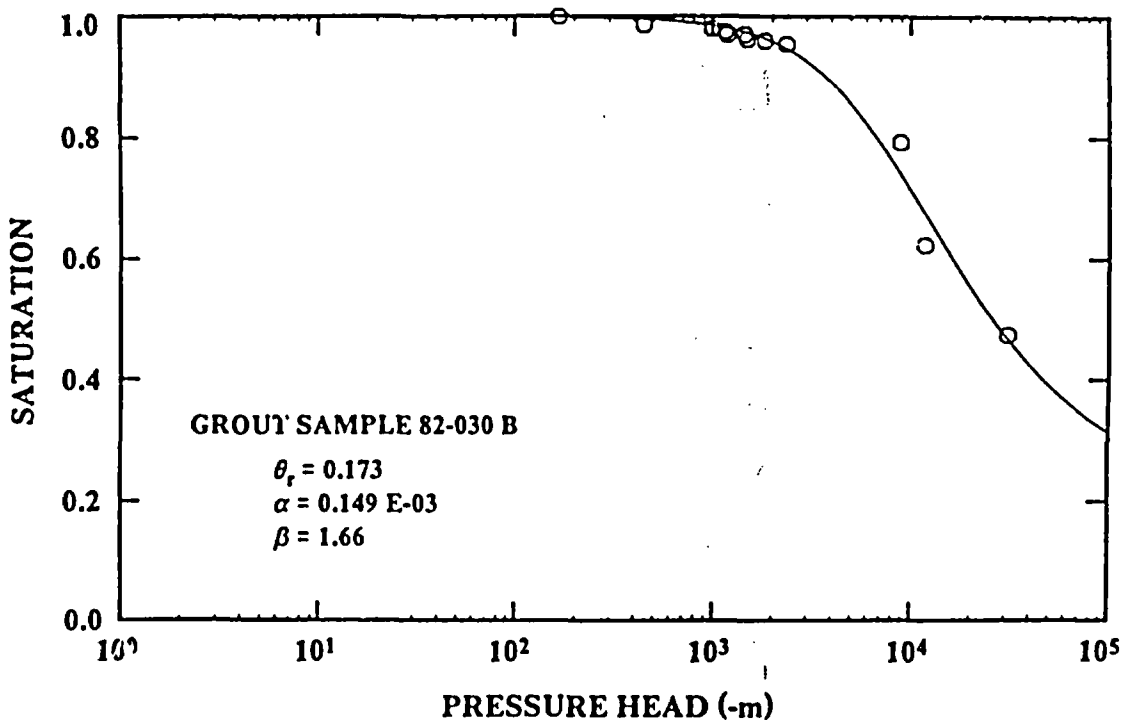


Figure G-11. Saturation and Relative, Hydraulic Conductivity Curves for Grout Sample 82-030A



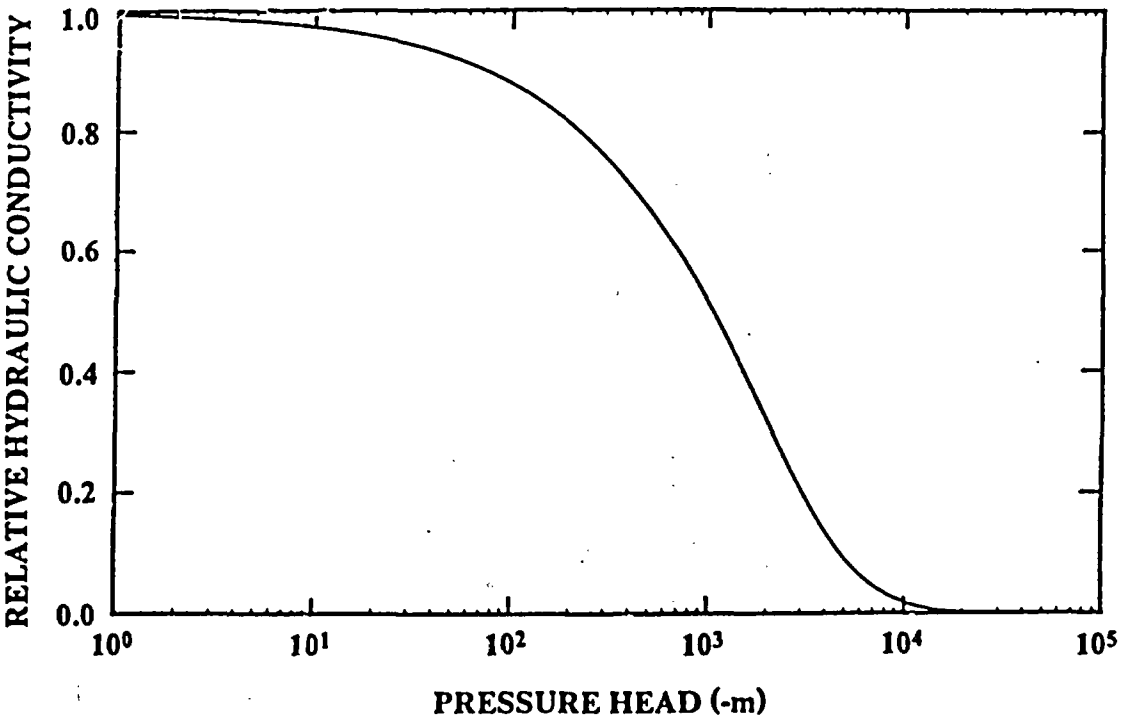
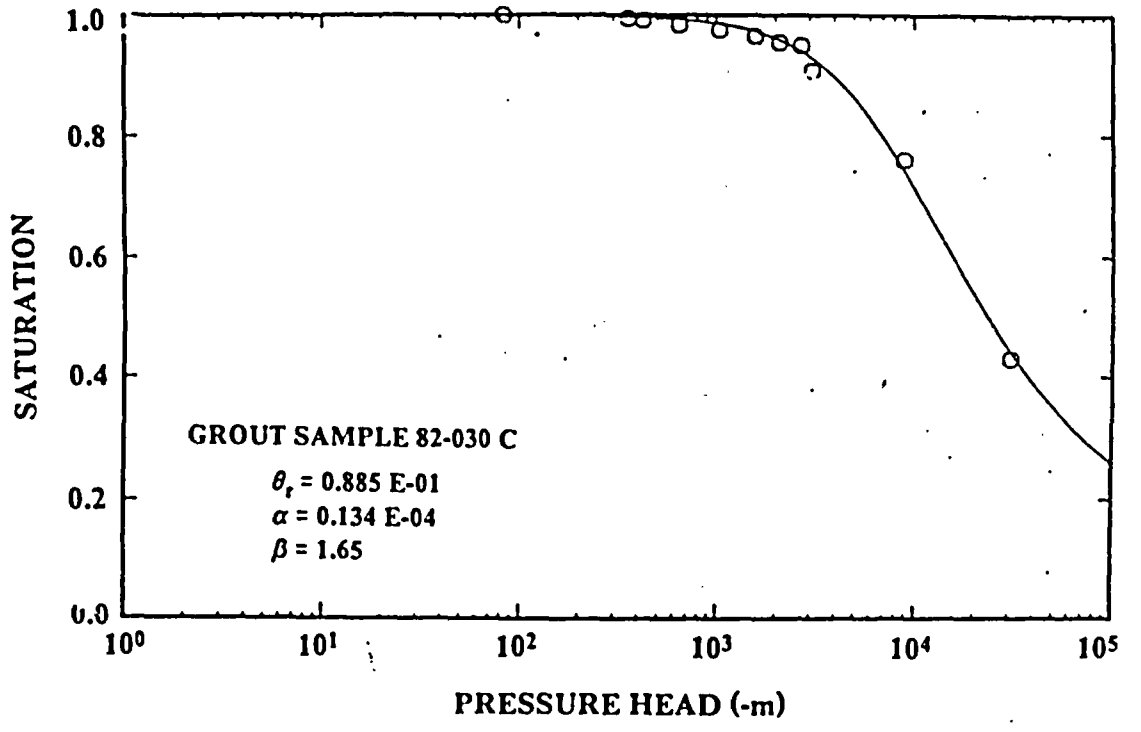


Figure G-12. Saturation and Relative, Hydraulic Conductivity Curves for Grout Sample 82-030B



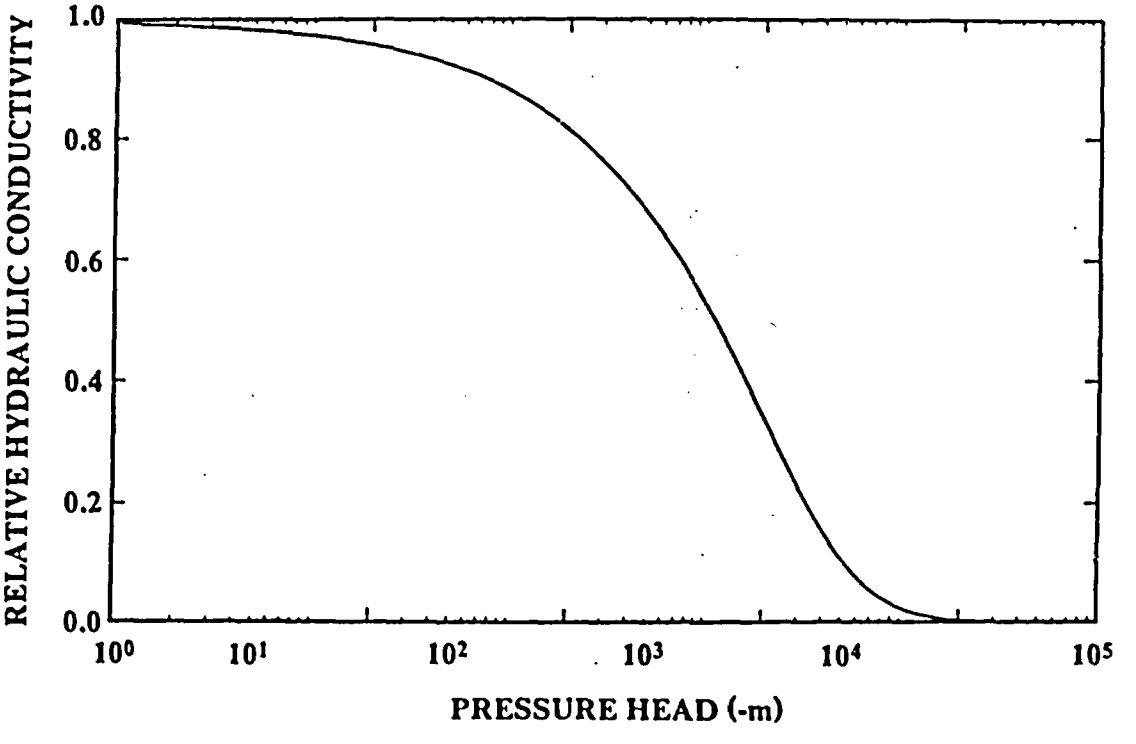
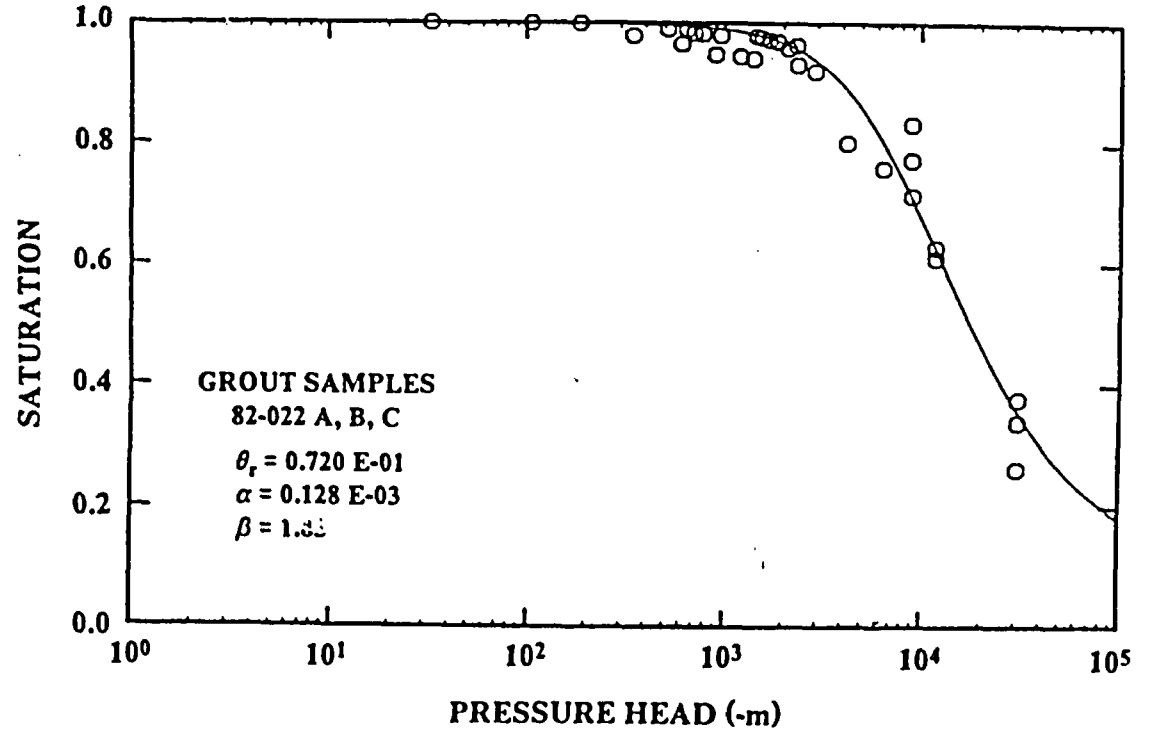


Figure G-13. Saturation and Relative, Hydraulic Conductivity Curves for Grout Sample 82-030C



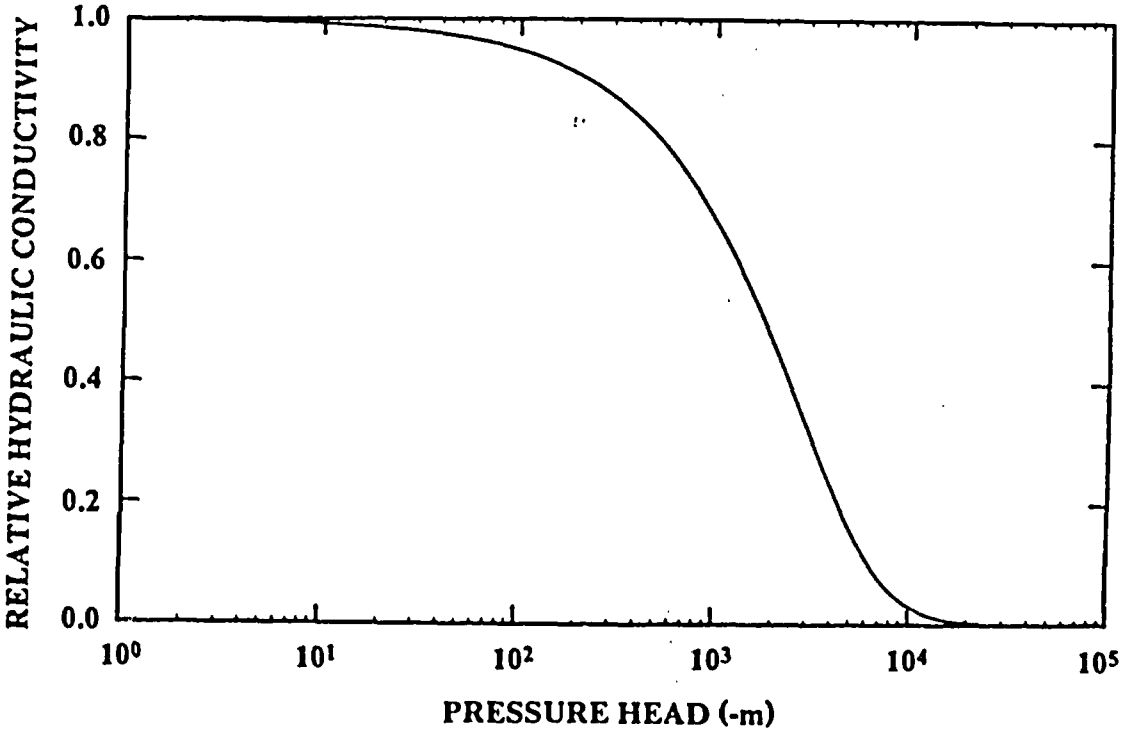
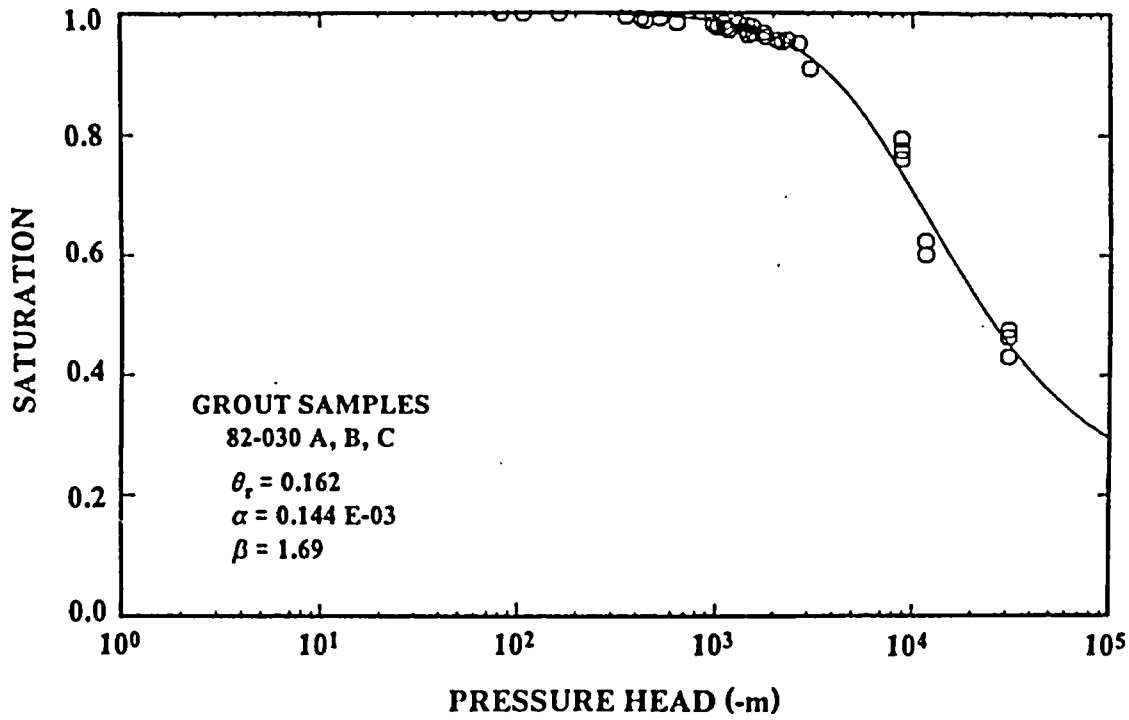


Figure G-14. Saturation and Relative, Hydraulic Conductivity Curves for Grout Samples 82-022A, 82-022B, 82-022C



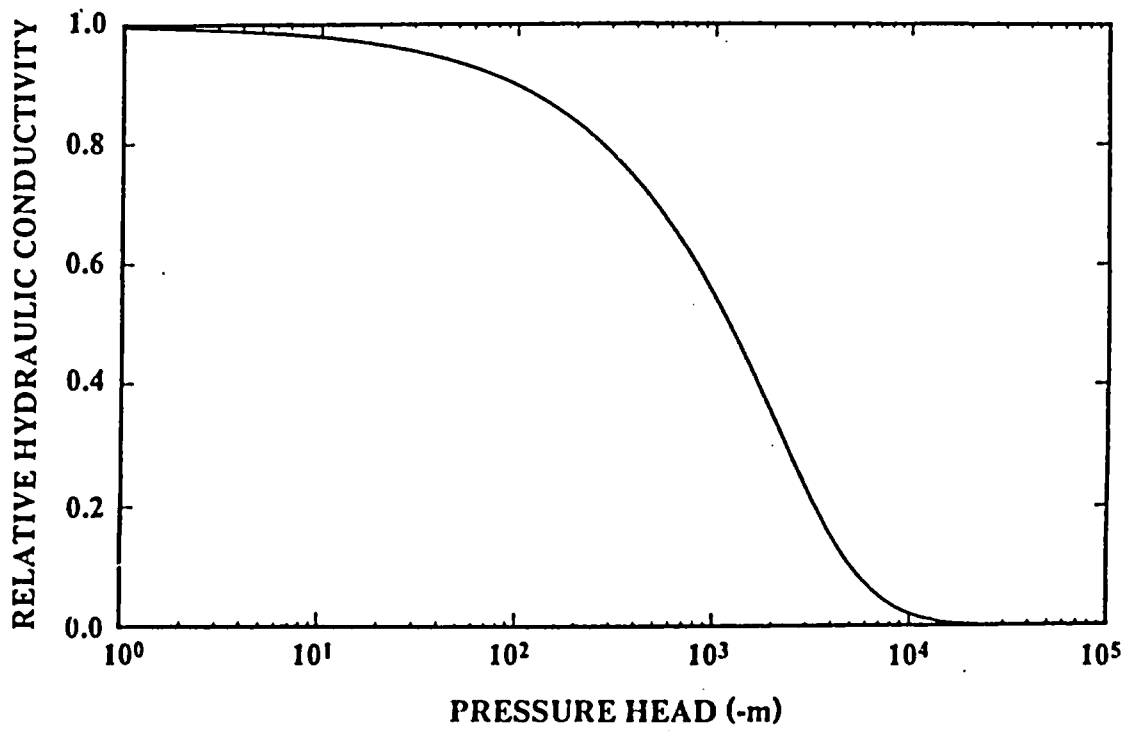
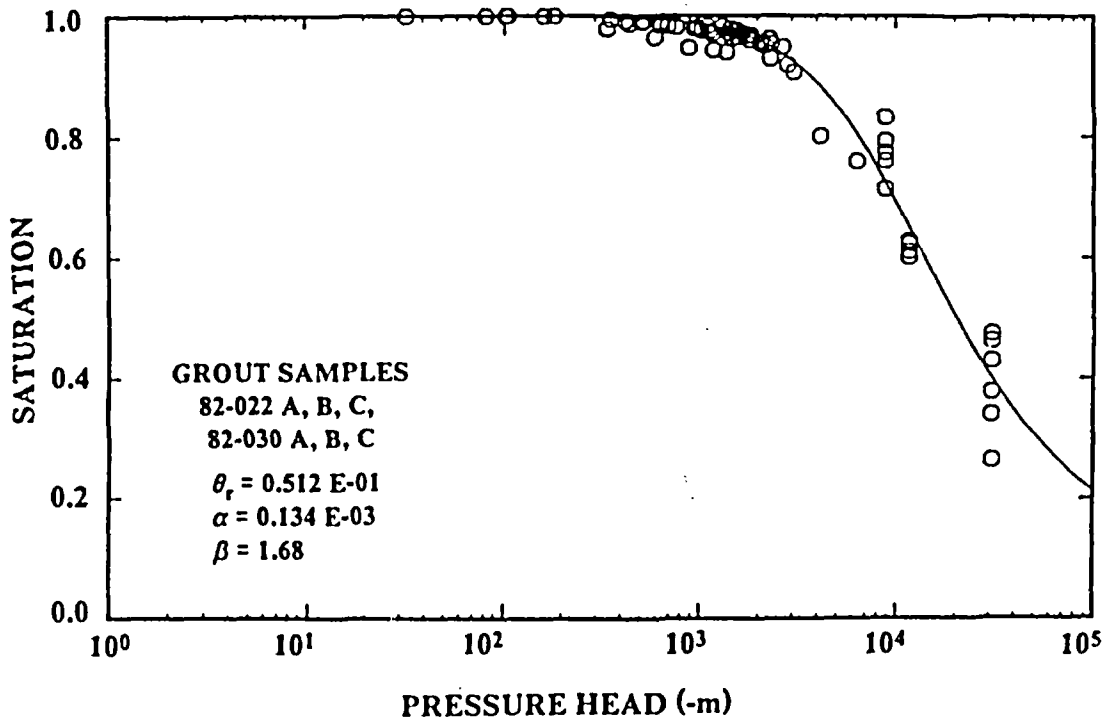


Figure G-15. Saturation and Relative, Hydraulic Conductivity Curves for Grout Samples 82-030A, 82-030B, 82-030C



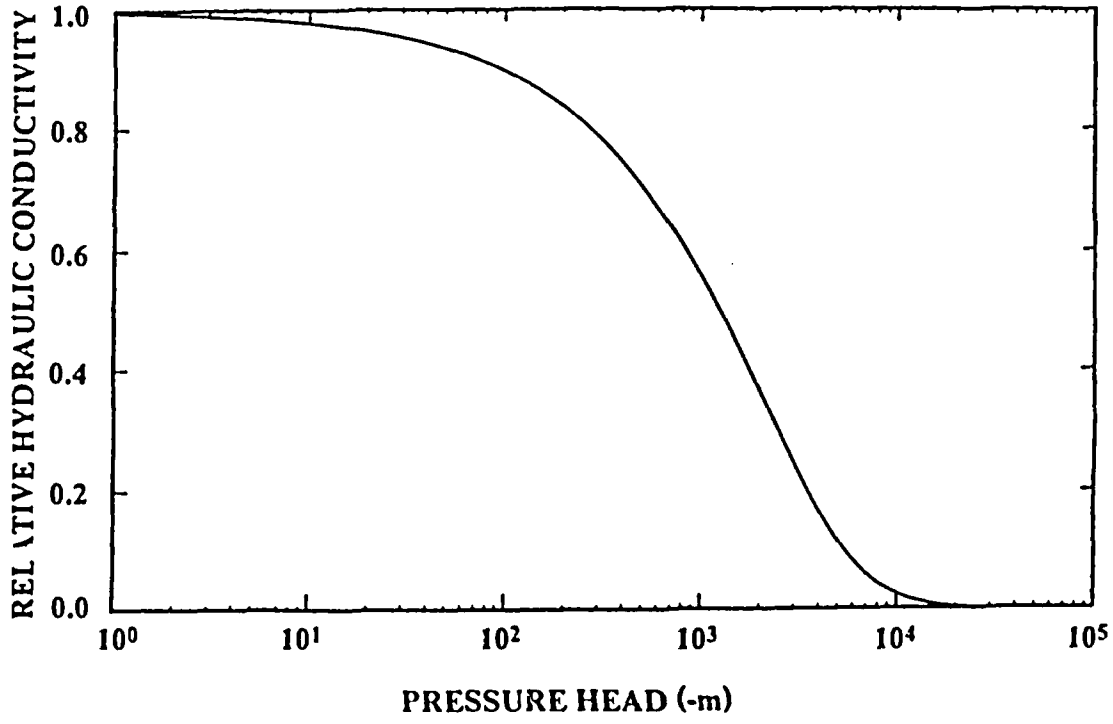


Figure G-16. Saturation and Relative, Hydraulic Conductivity Curves for All Grout Samples

Table G-3. ID Number for Each Data Set Analyzed

Curve ID No.[a]	Data Set Analyzed
1	Sample CL-40-CON A
2	Sample CL-40-CON B
3	Sample CL-40-CON C
4	Sample CL-40-CON D
5	Sample CL-40-CON E
6	Sample CL-40-CON F
7	Samples CL-40-CON A to CL-40-CON F
8	Sample 82-022A
9	Sample 82-022B
10	Sample 82-022C
11	Sample 82-030A
12	Sample 82-030B
13	Sample 82-030C
14	Samples 82-022A, 82-022B, 82-022C
15	Samples 82-030A, 82-030B, 82-030C
16	Samples 82-022A, 82-022B, 82-022C, and Samples 82-030A, 82-030B, 82-030C

[a]Curve ID No. is used to identify data sets.

Table G-4. Correlation Between Saturated Hydraulic Conductivity and Saturated Permeability

Sample	Hydraulic Conductivity, cm/s	Permeability, m^2
82-022A	2.89×10^{-10}[a]	2.89×10^{-19}
82-022B	4.21×10^{-10}	4.21×10^{-19}
82-022C	1.58×10^{-10}	1.58×10^{-19}
82-030A	5.61×10^{-10}	5.61×10^{-19}
82-030B	9.51×10^{-10}	9.51×10^{-19}
82-030C	2.80×10^{-10}	2.80×10^{-19}
82-022 (average)	2.89×10^{-10}	2.89×10^{-19}
82-030 (average)	5.13×10^{-10}	5.13×10^{-19}
Grouts (average)	4.74×10^{-10}	4.74×10^{-19}
CL-40-CON A	3.55×10^{-10}	3.55×10^{-19}
CL-40-CON B	2.19×10^{-10}	2.19×10^{-19}
CL-40-CON C	4.69×10^{-10}	4.69×10^{-19}
CL-40-CON D	4.97×10^{-10}	4.97×10^{-19}
CL-40-CON E	4.43×10^{-10}	4.43×10^{-19}
CL-40-CON F	3.46×10^{-10}	3.46×10^{-19}
Concrete (average)	3.88×10^{-10}	3.88×10^{-19}

[a] Average of 82-022 samples used.

$$K_s = k_s \rho g / \mu$$

where

K_s = saturated hydraulic conductivity,
k_s = saturated permeability,
ρ = density of water,
g = acceleration due to gravity, and
μ = viscosity of water.

The samples were run at 21°C. Therefore, the values for density and viscosity of water were 998 kg/m^3 and 84.49 kg/(day•m), respectively. The value for the gravitational constant was 7.3148×10^{10} m/day^2. Based on the conversion formula and the above values, the intrinsic permeability and hydraulic conductivity values are given in Table G-4.

The first step in the determination of unsaturated properties is the data reduction for the curve-fit of the saturation versus pressure-head data. When the shape of the saturation versus pressure-head curve is well defined by the data points, the VanGenuchten and Haverkamp curve fits are nearly identical (Peters et al., 1984). Further, the VanGenuchten curve fit was chosen as the standard method to fit saturation data (Peters et al., 1984, p. D-1) for the reduction of data associated with tuff samples. This same curve-fitting procedure was used to reduce the data obtained for the concrete and grout samples.

The equation defining the VanGenuchten, curve fit is:

$$S = (S_s - S_r) \left(\frac{1}{1 + |\alpha h|^{\beta}} \right)^{\lambda} + S_r$$

where

S = saturation as a function of pressure head,
S_s = maximum saturation,
S_r = residual saturation,
$\lambda = 1 - 1/\beta$,
h = pressure head, and

α and β are VanGenuchten, curve-fit parameters.

Some difficulty was experienced in curve-fitting the laboratory data because the residual saturation was not well defined for the concrete and grout samples. To obtain better definition of the residual saturation, samples were placed in controlled atmospheres of 53%, 35% and 11% relative humidity and allowed to reach equilibrium. Water contents were then calculated. These relative-humidity values correspond to high, negative pressure heads, and once

the data were obtained from this extended testing, the residual saturation could be better defined.

The primary parameters obtained as part of the VanGenuchten, curve fit are:

θ_r = residual saturation
α = curve-fit parameter, and
β = curve-fit parameter.

The results of these statistical analysis are contained on Figures G-1 through G-16. Sixteen data sets were fitted. The legend for these curve fits is given in Table G-3. Using the curve-fit parameters and the equation for the VanGenuchten fit, a continuous curve for saturation versus pressure head can be developed (Figures G-1 through G-16).

Using these same, curve-fit parameters, the equation for the unsaturated hydraulic conductivity, and the method of Mualem (Peters et al., 1984, p. 31), the hydraulic conductivity as a function of pressure head can be computed:

$$K(h) = K_s \frac{[1 - |\alpha h|^{\beta-1} \cdot (1 + |\alpha h|^{\beta})^{-\lambda}]^2}{(1 + |\alpha h|^{\beta})^{\lambda/2}}$$

where

K(h) = hydraulic conductivity as a function of pressure head (h),
K_s = saturated hydraulic conductivity, and the other parameters are as defined earlier.

If 'K'_s is input as the value of 1.0, a relative hydraulic conductivity versus pressure-head curve can be developed (Figures G-1 through G-16). If the saturated, hydraulic conductivity value is input, a plot of actual hydraulic conductivity (or permeability) versus pressure head can be developed (Figures G-17 through G-37).

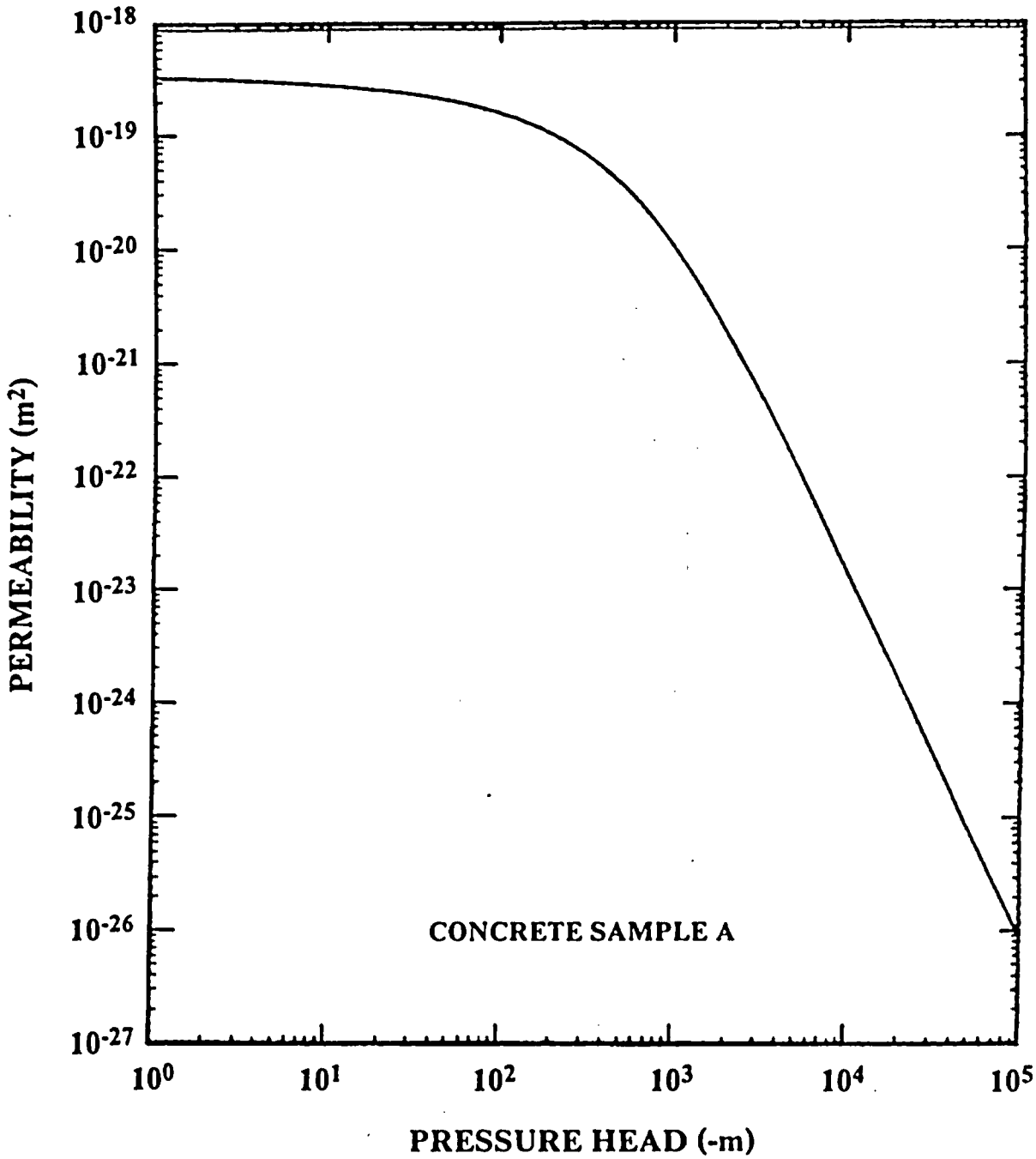


Figure G-17. Permeability Versus Pressure-Head Curve for Concrete Sample A

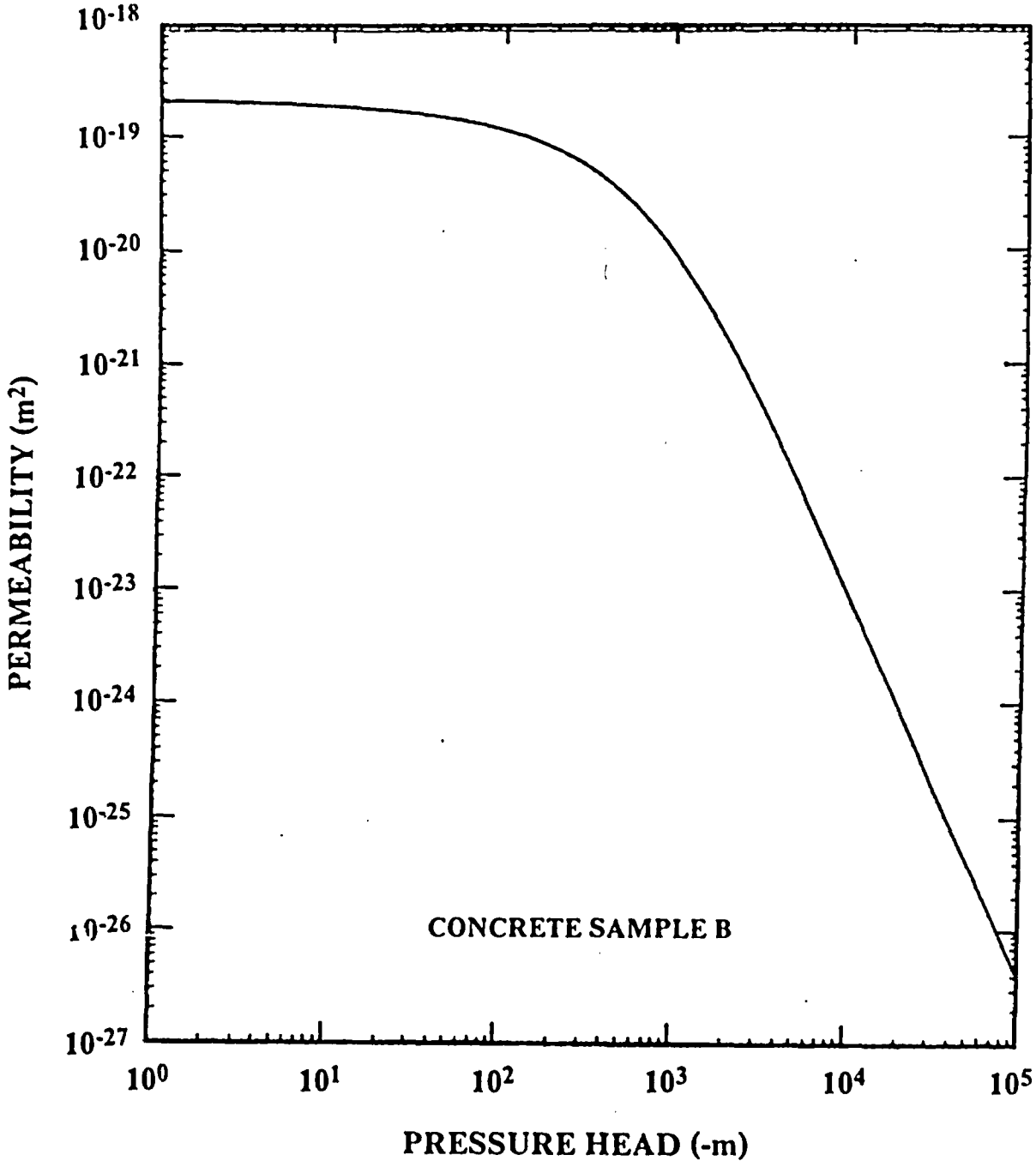


Figure G-18. Permeability Versus Pressure-Head Curve for Concrete Sample B

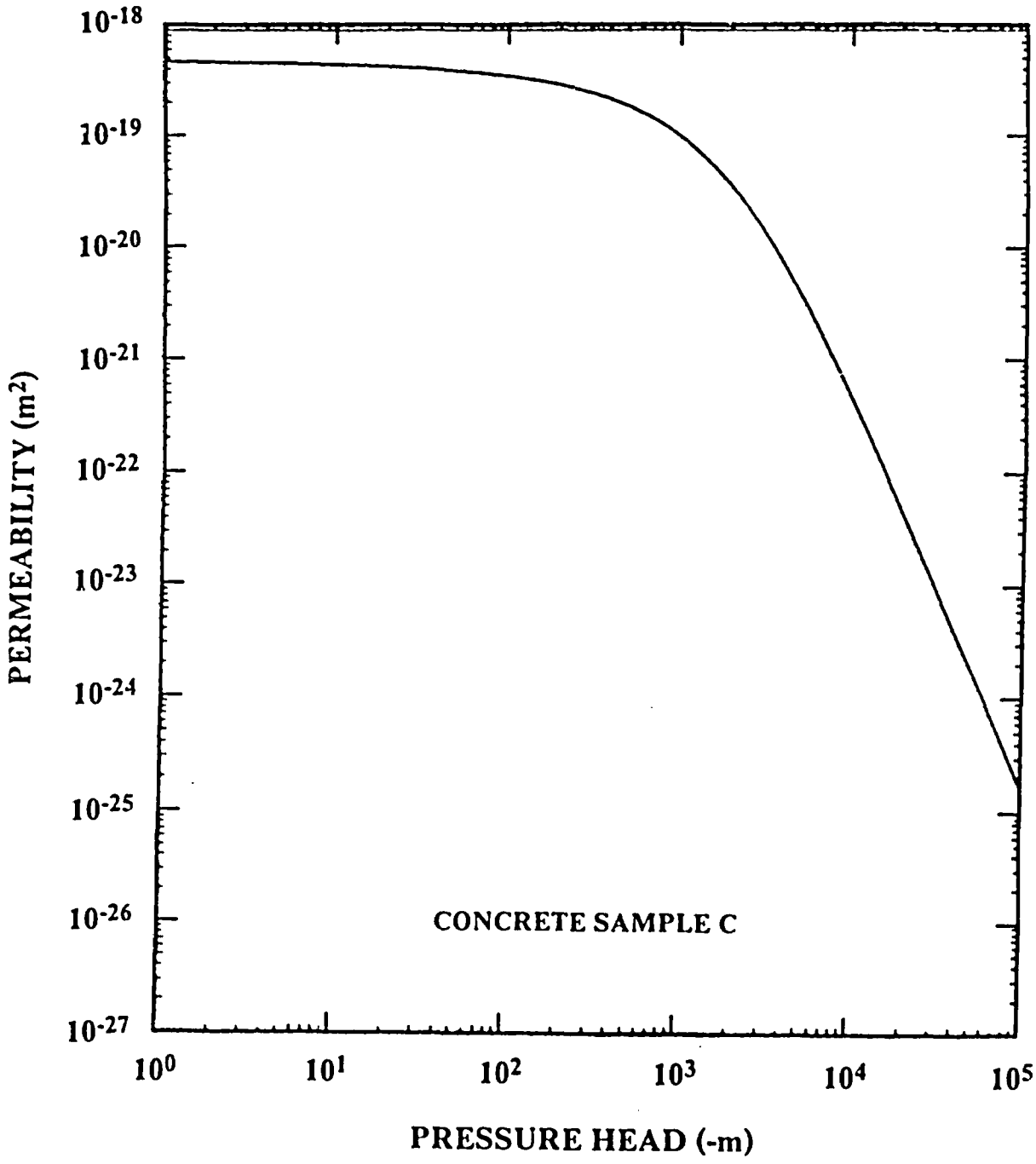


Figure G-19. Permeability Versus Pressure-Head Curve for Concrete Sample C

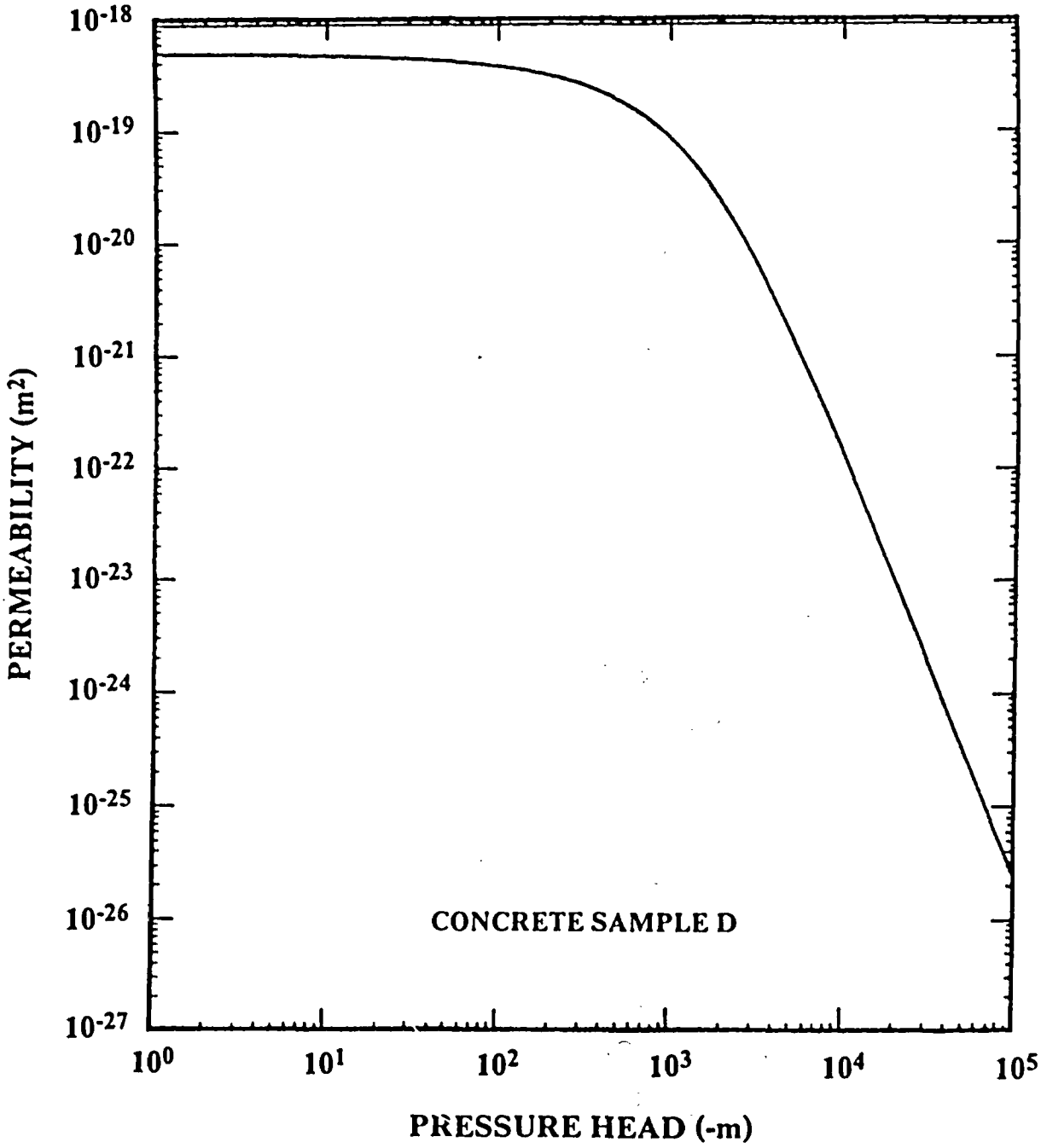


Figure G-20. Permeability Versus Pressure-Head Curve for Concrete Sample D

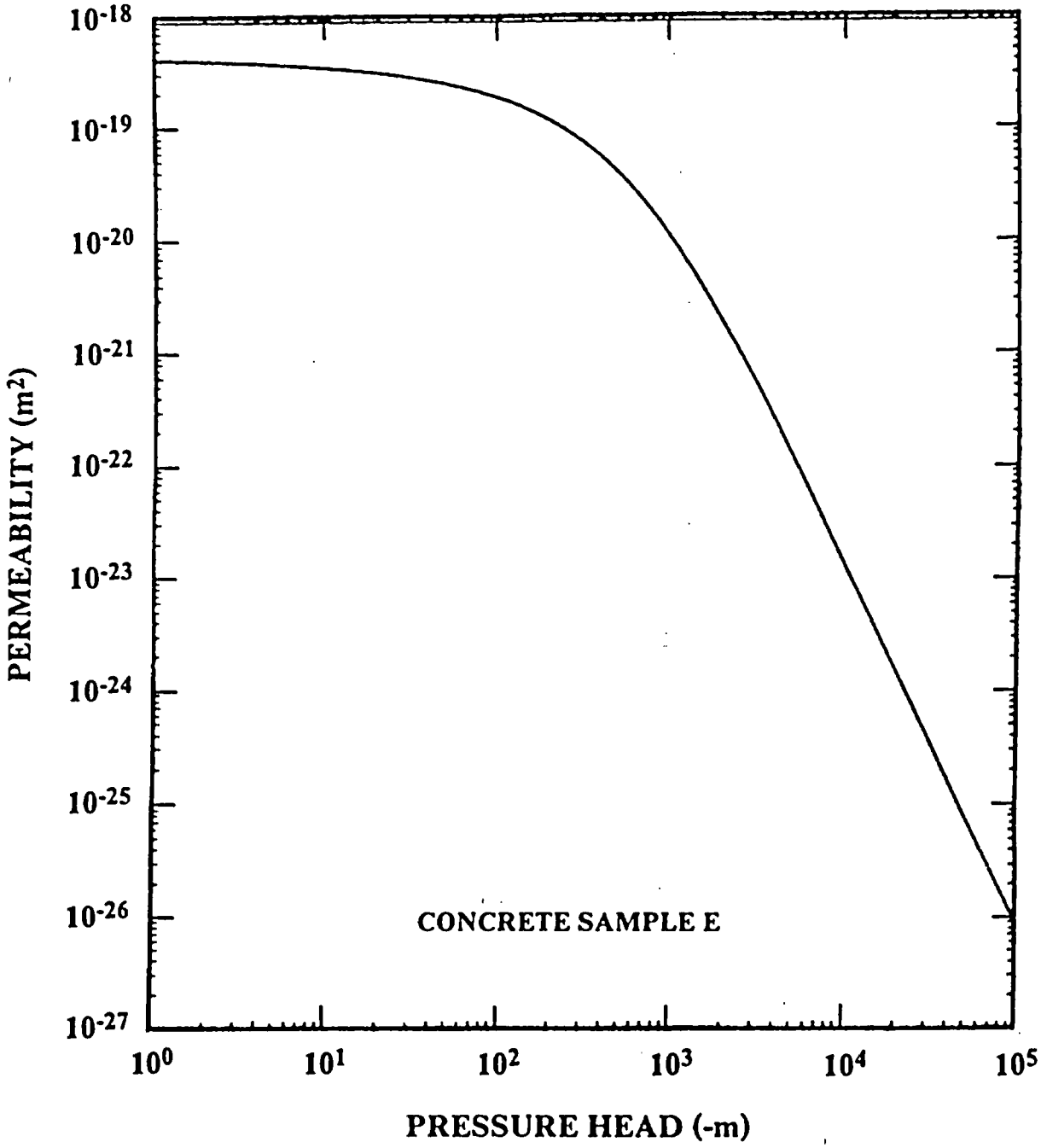


Figure G-21. Permeability Versus Pressure-Head Curve for Concrete Sample E

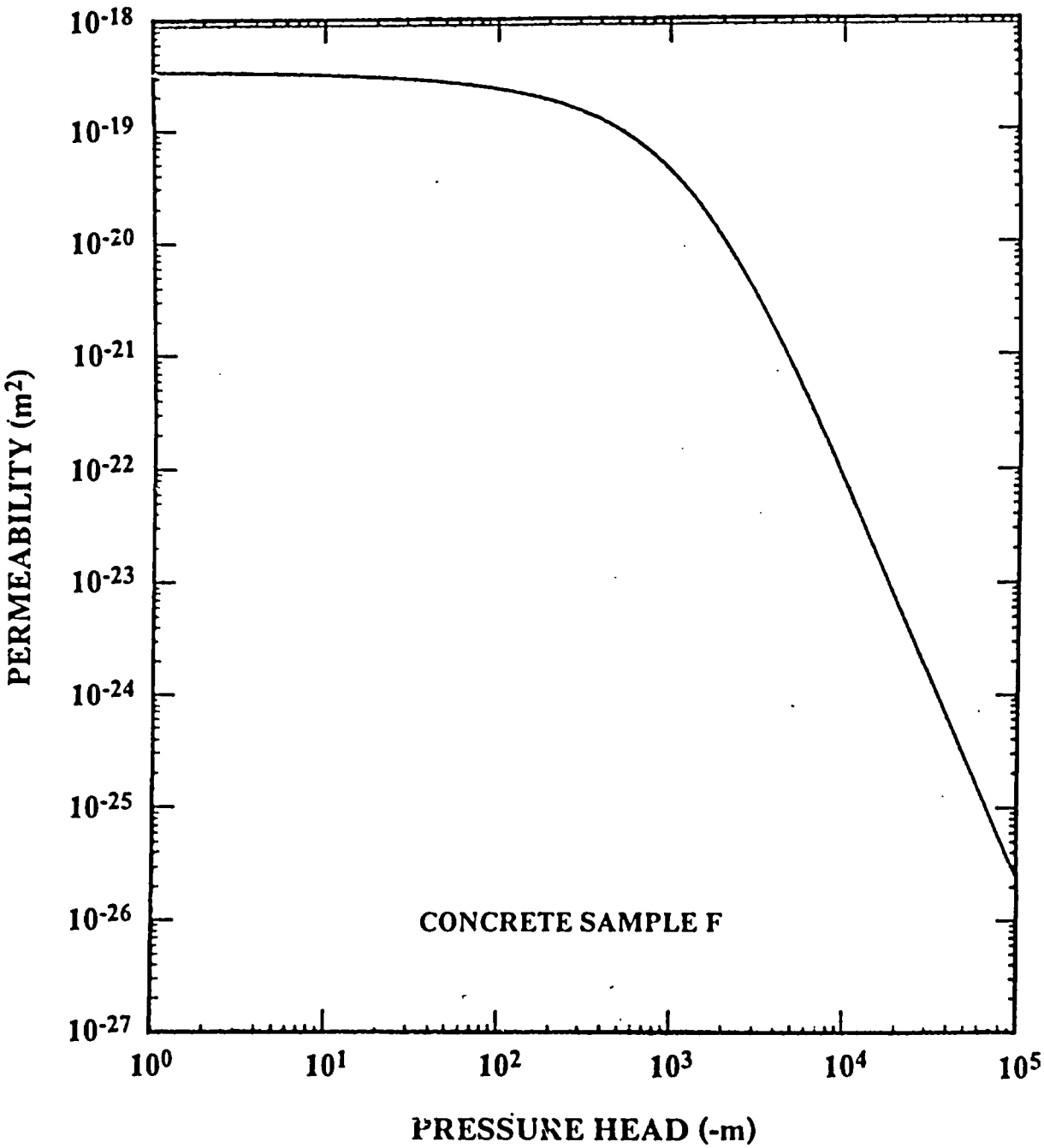


Figure G-22. Permeability Versus Pressure-Head Curve for Concrete Sample F

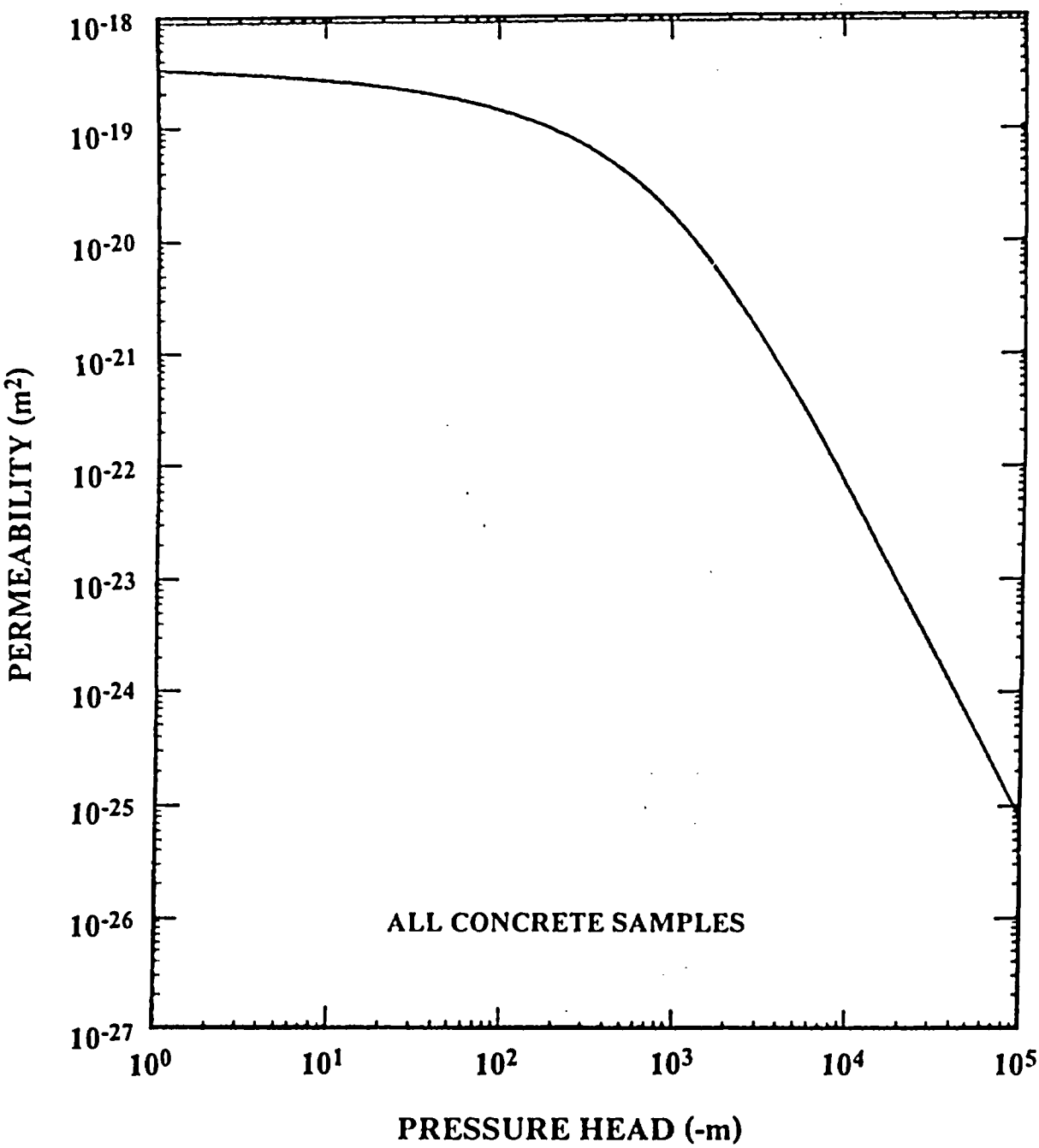


Figure G-23. Permeability Versus Pressure-Head Curve for All Concrete Samples

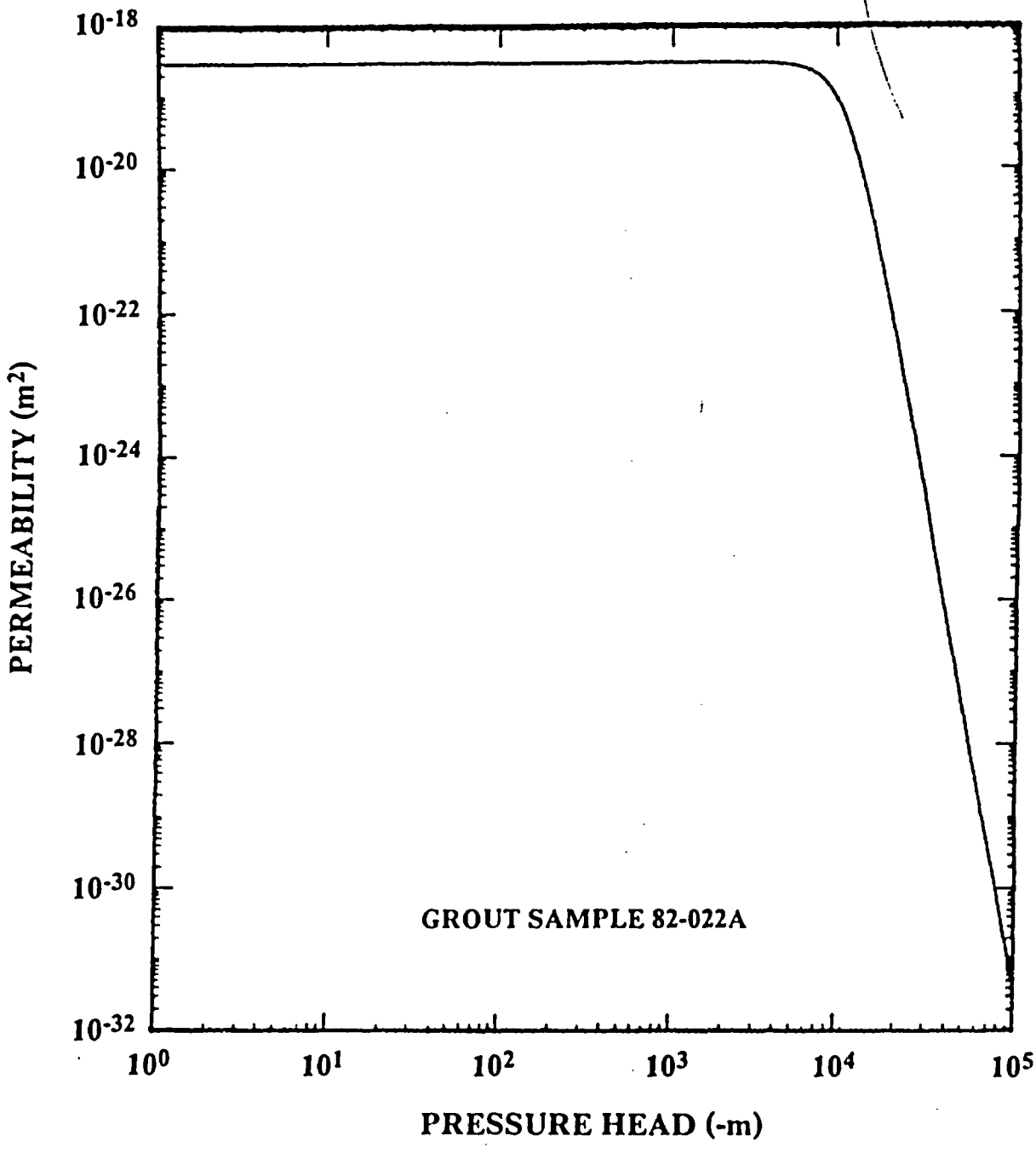


Figure G-24. Permeability Versus Pressure-Head Curve for Grout Sample 82-022A

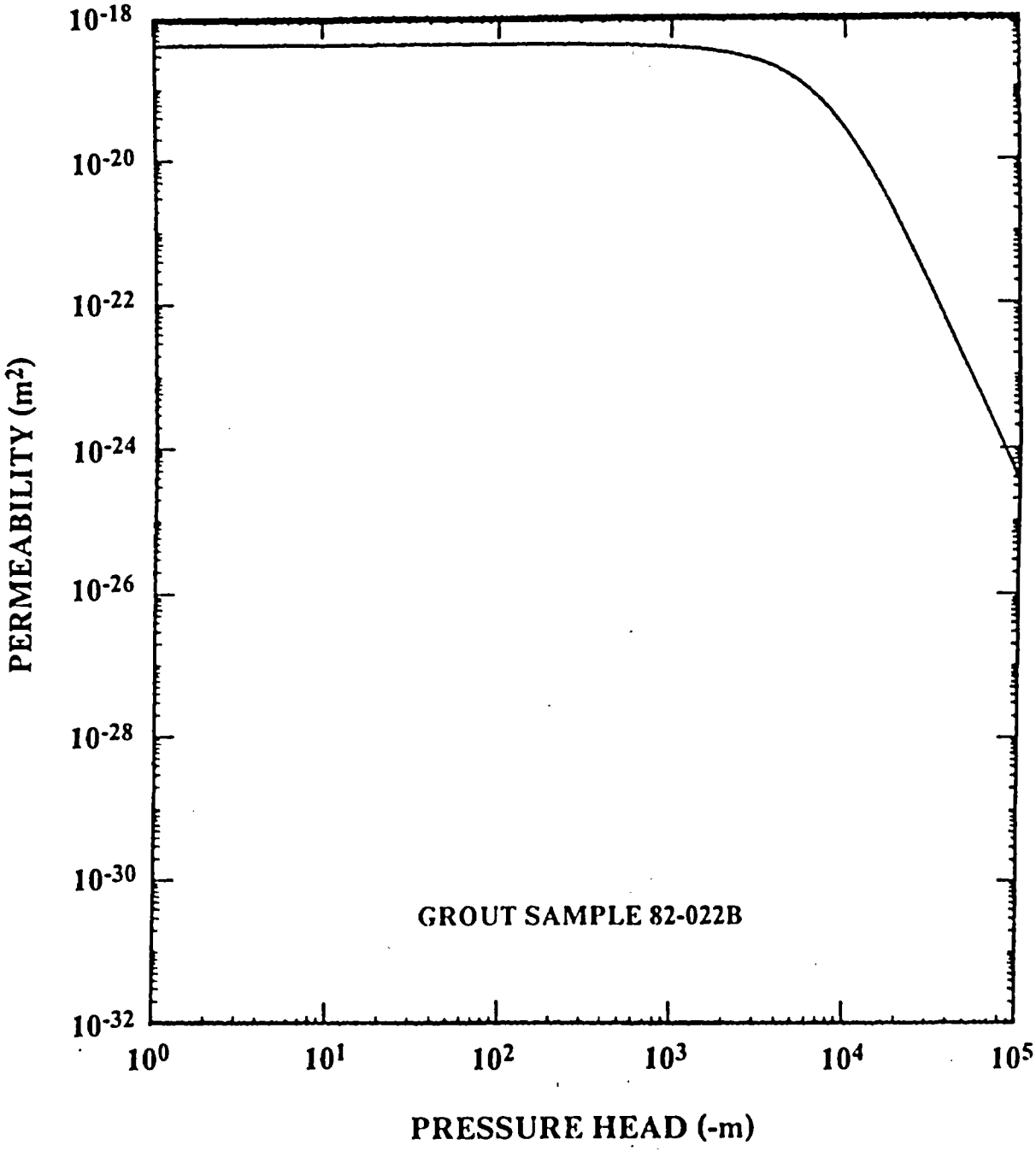


Figure G-25. Permeability Versus Pressure-Head Curve for Grout Sample 82-022B

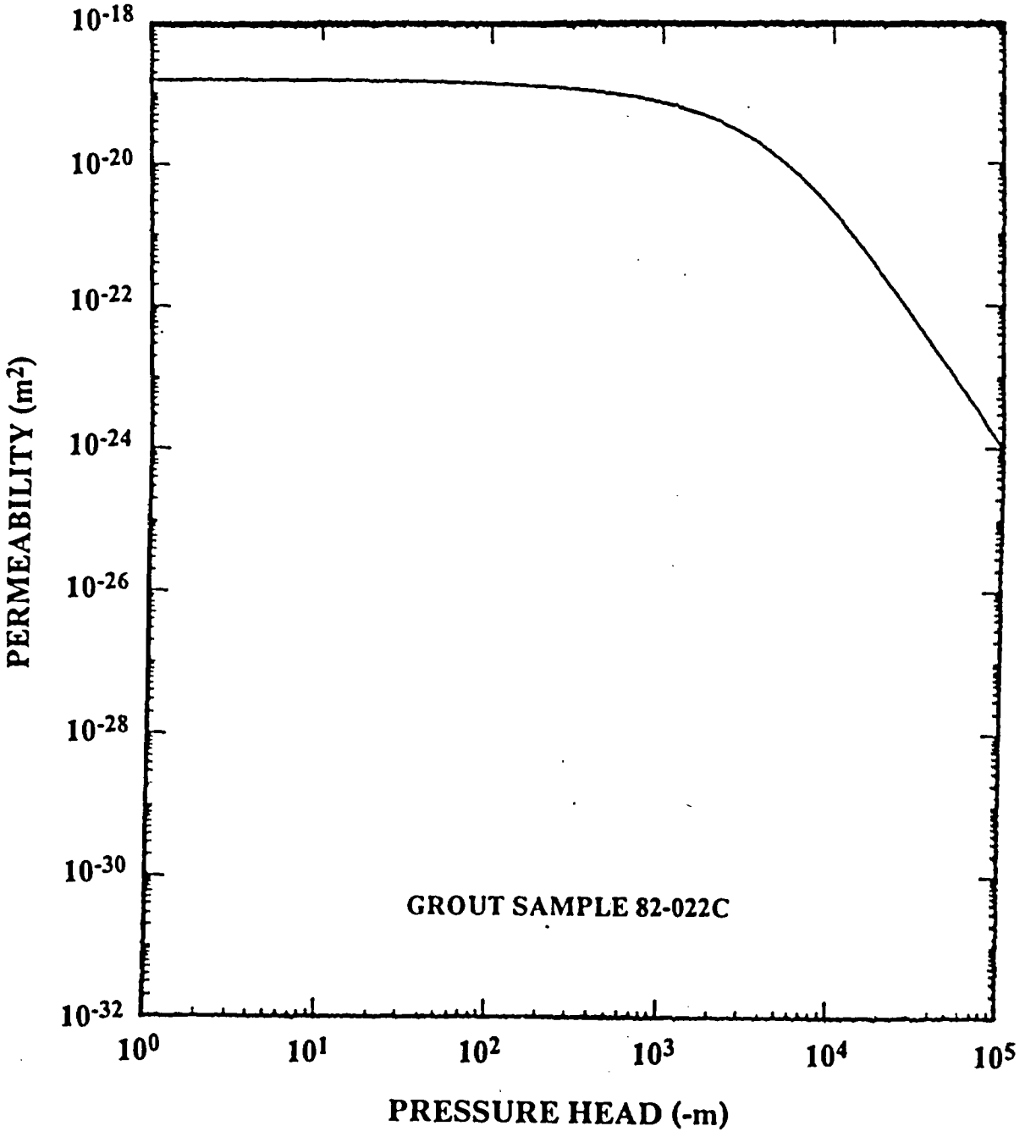


Figure G-26. Permeability Versus Pressure-Head Curve for Grout Sample 82-022C

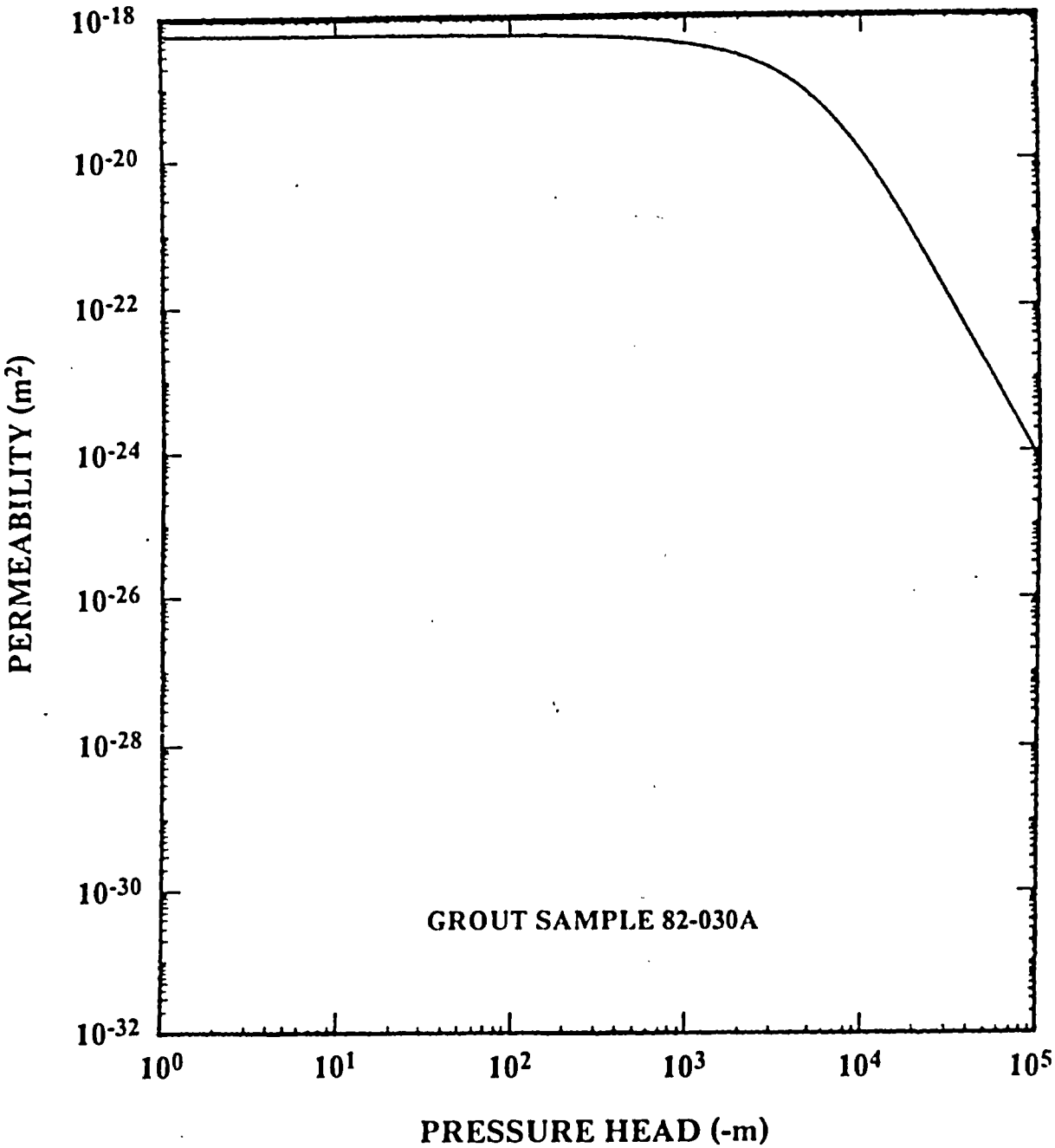


Figure G-27. Permeability Versus Pressure-Head Curve for Grout Sample 82-030A

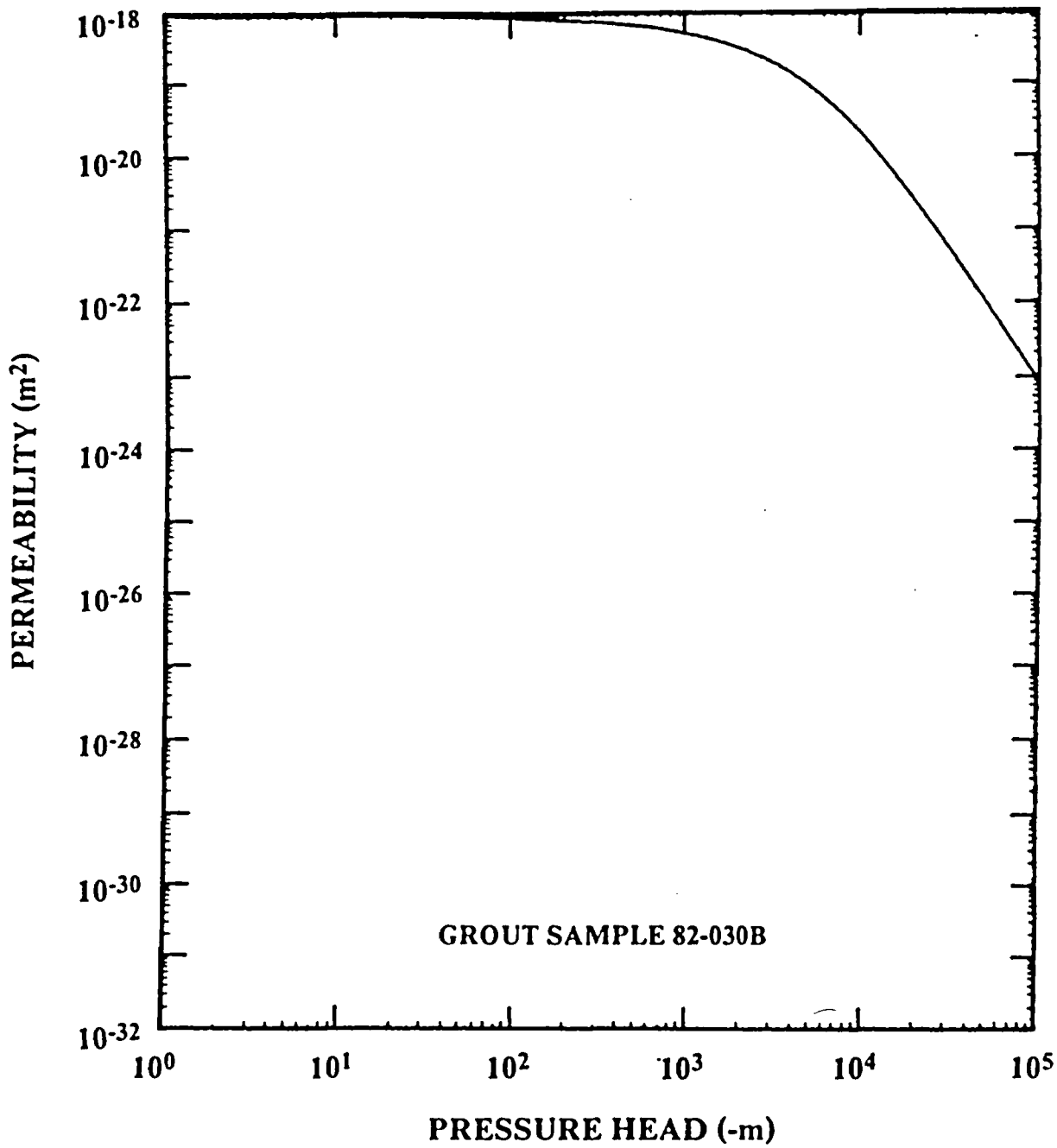


Figure G-28. Permeability Versus Pressure-Head Curve for Grout Sample 82-030B

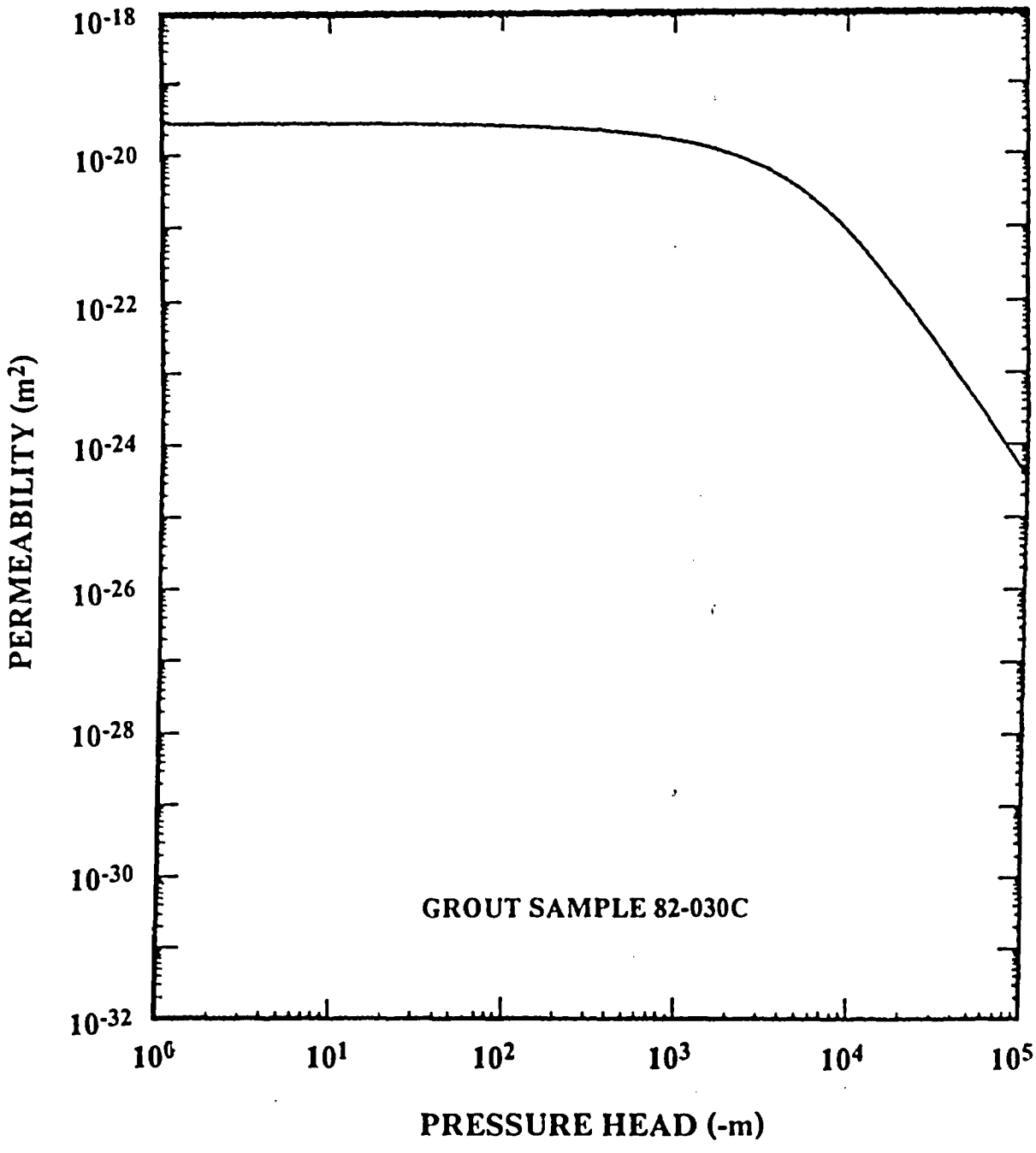


Figure G-29. Permeability Versus Pressure-Head Curve for Grout Sample 82-030C

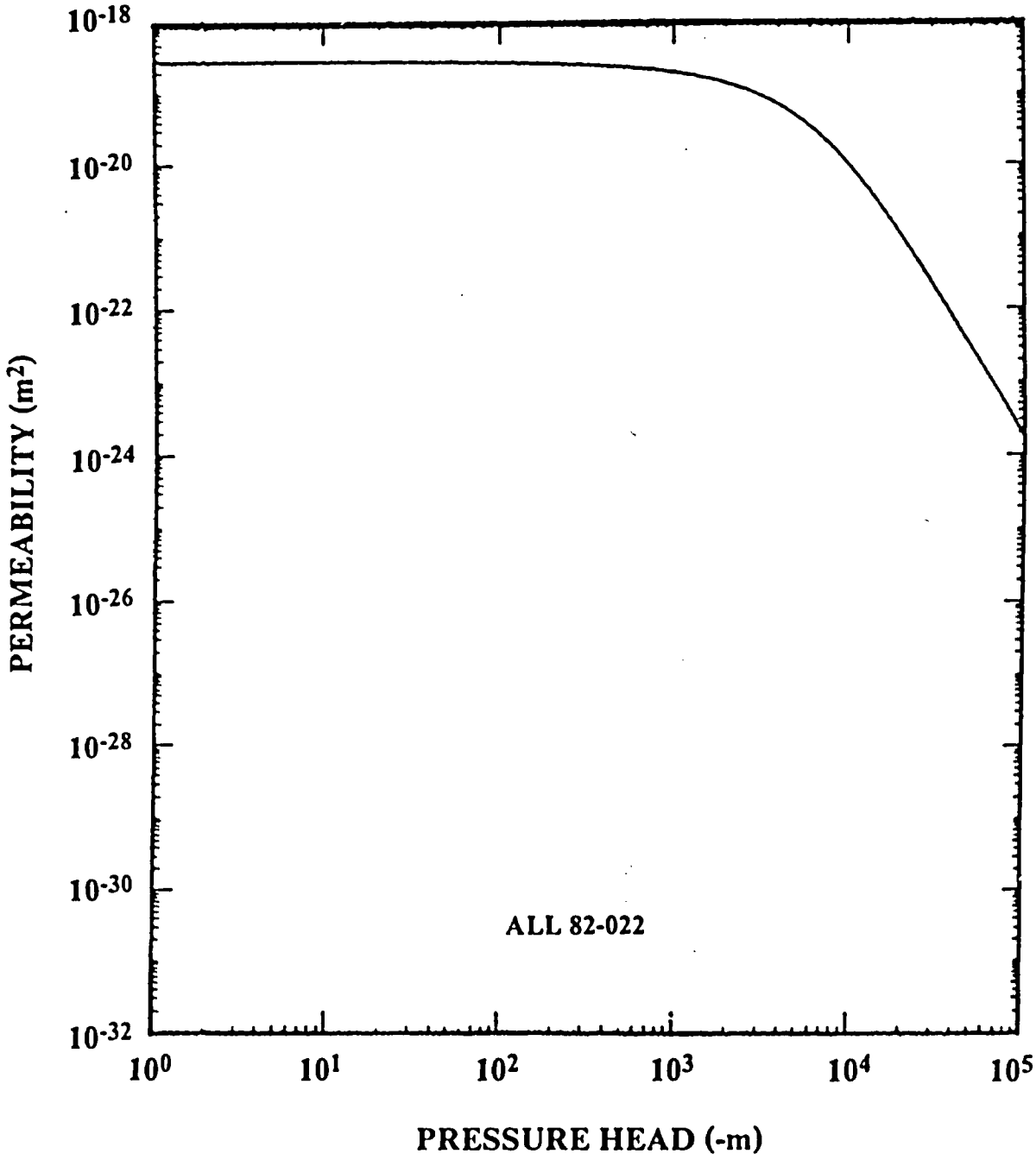


Figure G-30. Permeability Versus Pressure-Head Curve for Grout Samples 82-022A, 82-022B, 82-022C

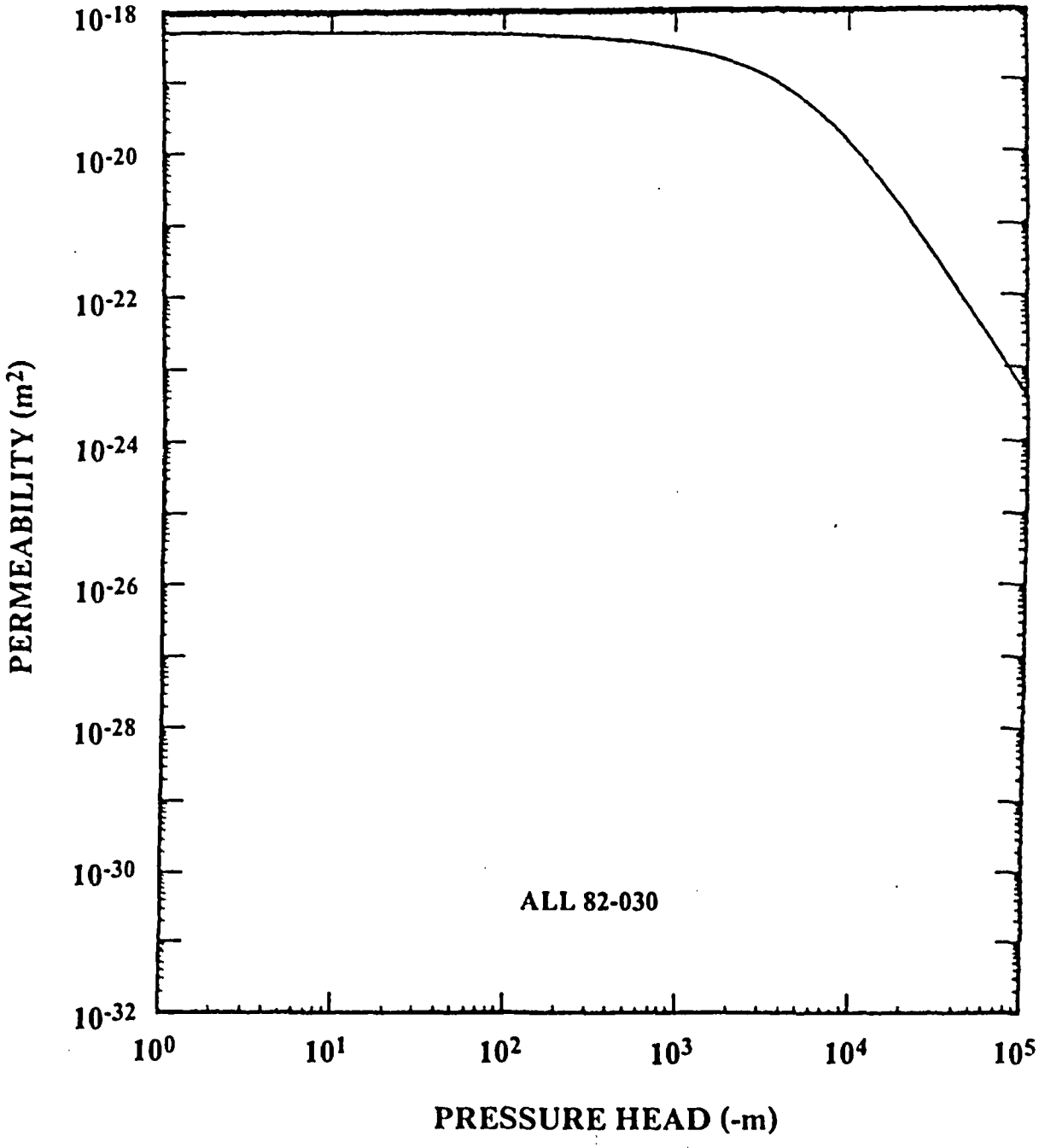


Figure G-31. Permeability Versus Pressure-Head Curve for Grout Samples 82-030A, 82-030B, 82-030C

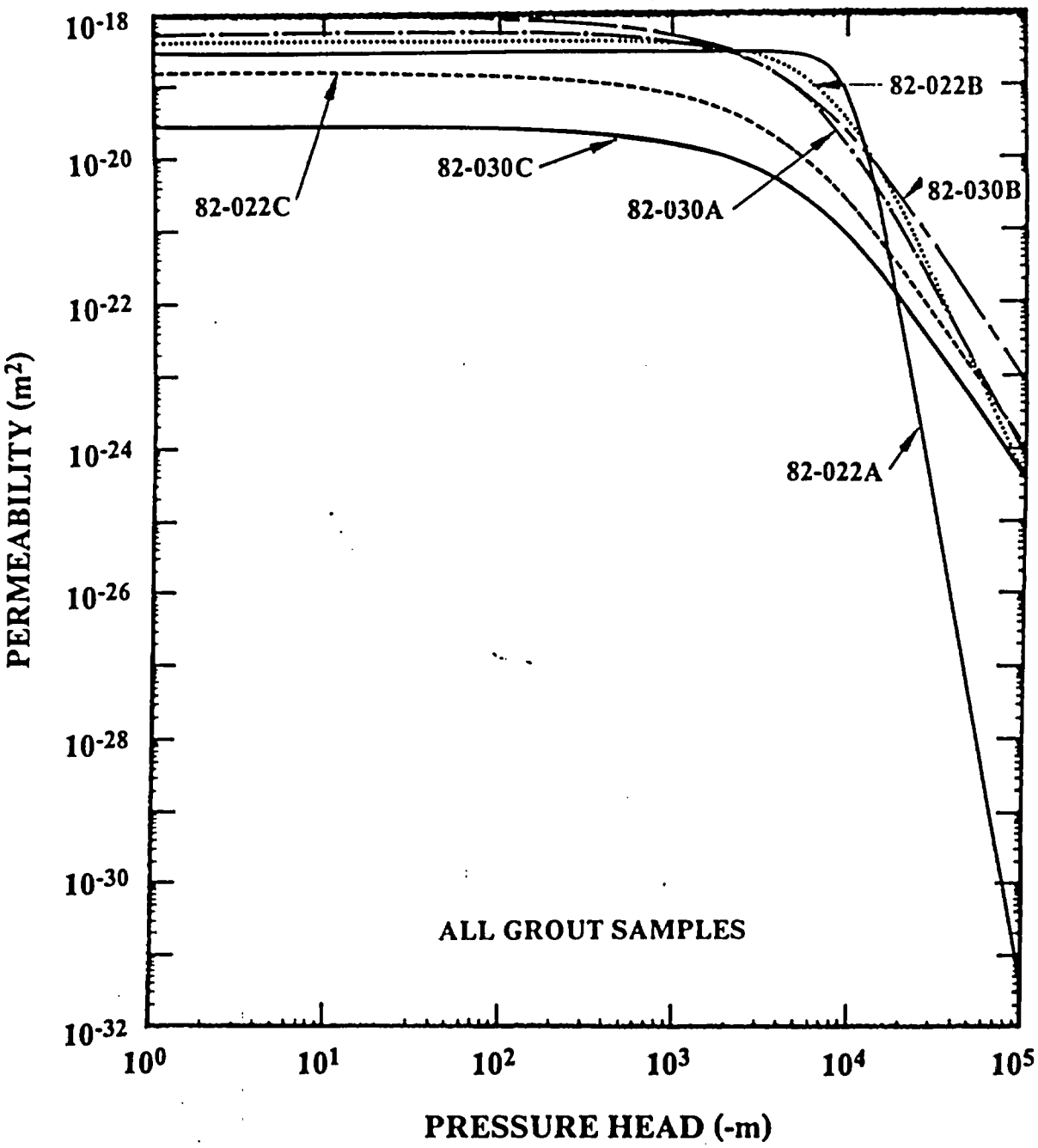


Figure G-32. Permeability Versus Pressure-Head Curve for All Grout Samples

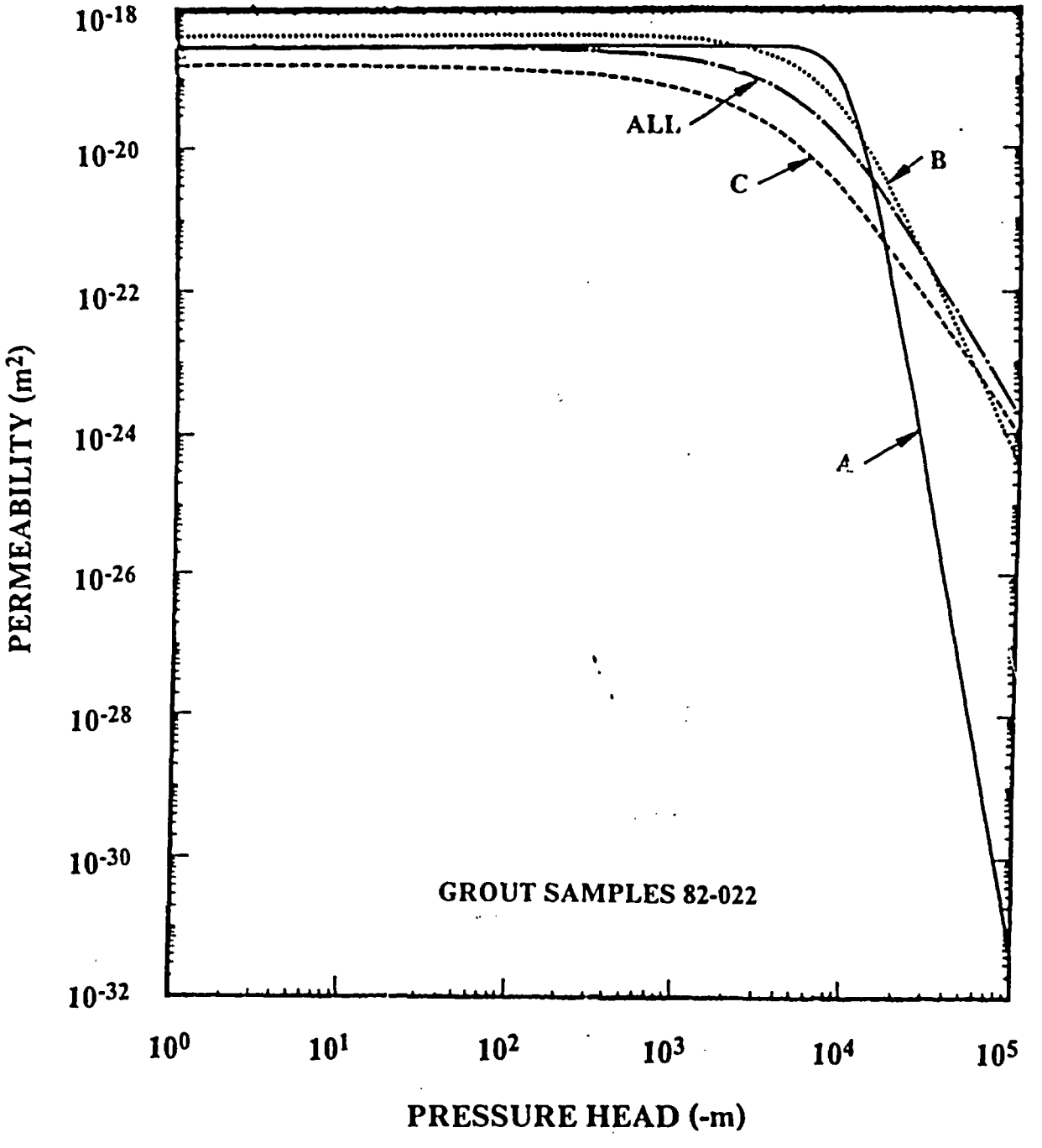


Figure G-33. Permeability Versus Pressure-Head Curve for Grout Samples 82-022

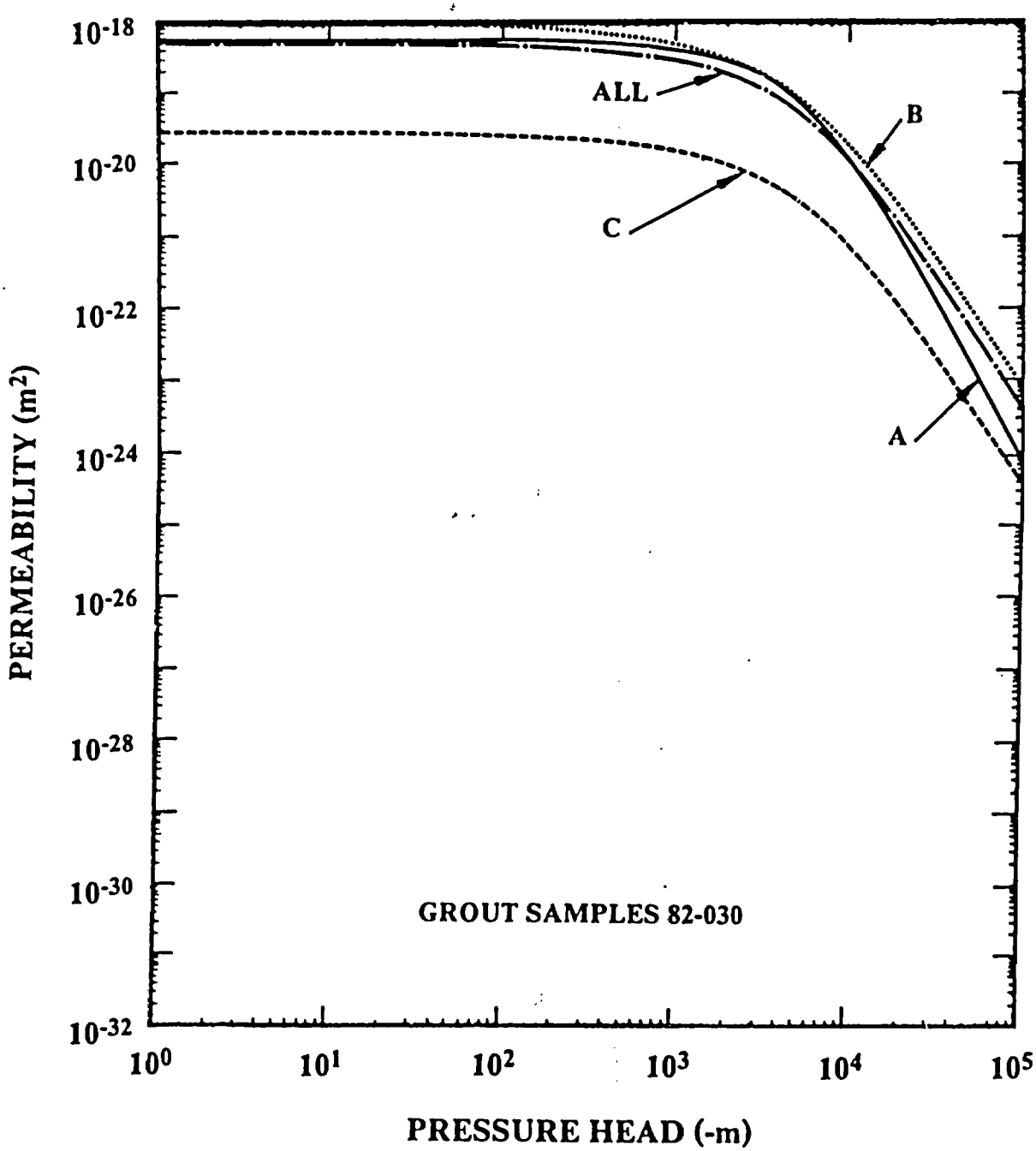


Figure G-34. Permeability Versus Pressure-Head Curve for Grout Samples 82-030

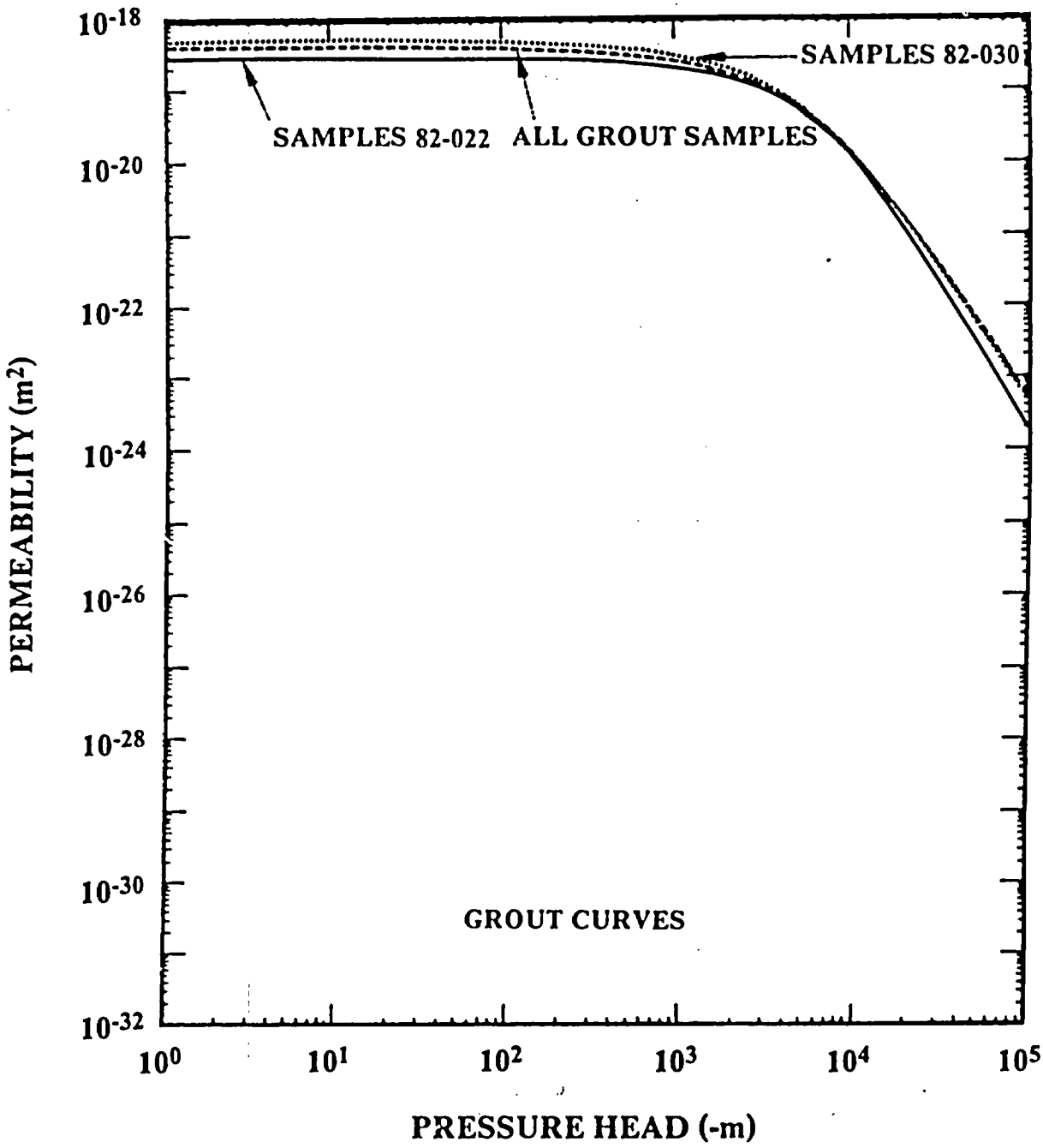


Figure G-35. Permeability Versus Pressure-Head Curve for Grout Curves 82-022, 82-030, All Grout Samples

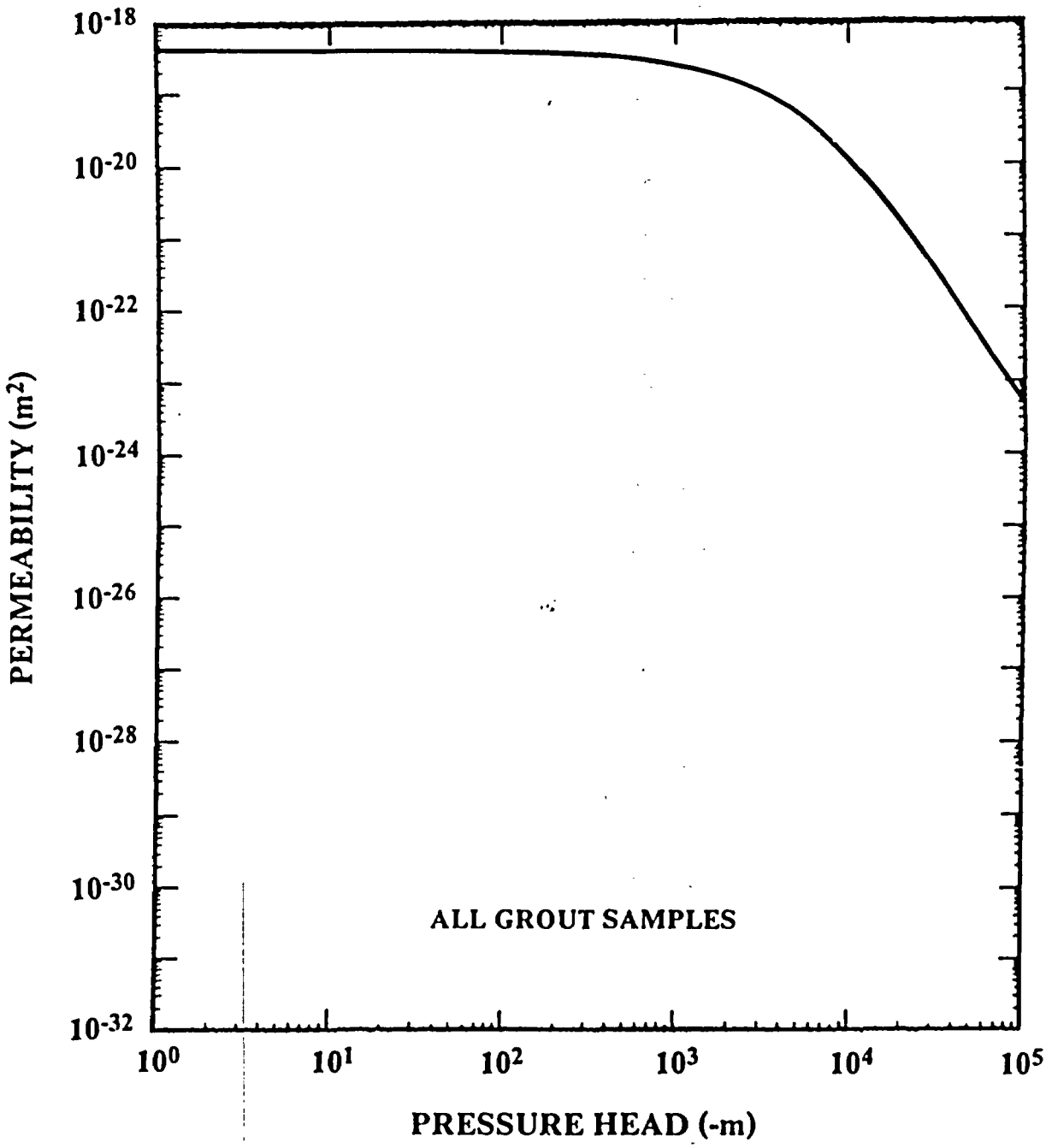


Figure G-36. Permeability Versus Pressure-Head Curve for All Grout Samples 82-022A, B, C, and 82-030A, B, C

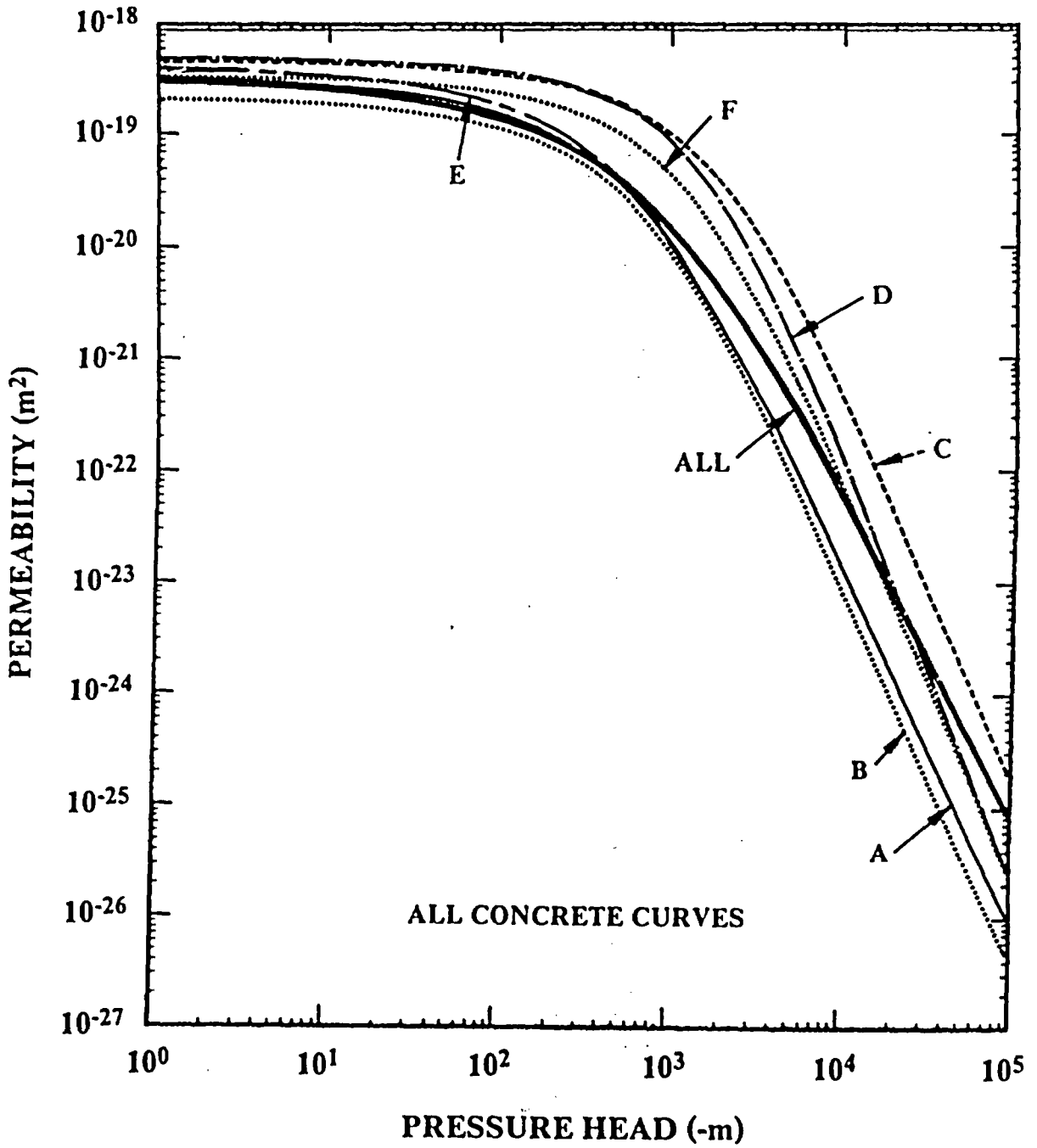


Figure G-37. Permeability Versus Pressure-Head Curve for All Concrete Samples

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PARAMETER	CHAPTER	SECTION	PAGE#	REPORT VALUE	UNITS	RIB VALUE	RIB SECTION	EXPLANATION
GENERAL STRATIGRAPHY	2	TABLE 2-1	3	--		NONE	1.3.2.1	(C)
NORTHWEST FAULT LOCATIONS	2	2.1	5	REQUIRED MAPS		NONE	1.3.2.4.1	(C)
GHOST DANCE FAULT - DIP	2	2.1	5	84° - 89° WEST		NONE	1.3.2.4.1	(C)
GHOST DANCE FAULT - OFFSET	2	2.1	5	25	M	NONE	1.7.2.2	(C)
ANNUAL EVAPOTRANSPIRATION	2	2.2.1	5	630	MM	NONE	(B)	
AVERAGE ANNUAL PRECIPITATION	2	2.2.1	5	150	MM	NONE	1.11.1.2.1	(C)
EVAPOTRANSPIRATION	2	2.2.1	5	0.76 - 10.7	IN	NONE	1.11.1.2.2	(C)
MAXIMUM THICKNESS OF ALLUVIUM	2	2.1	5	45 (153)	M (FT)	NONE	1.3.2.1	(C)
TUFF BEDS ORIENTATION	2	2.1	5	DIP 5° - 8° EAST		NONE	1.3.2.4	(C)
FLOOD LOCATIONS	2	2.2.1	6	REQUIRES MAPS		NONE	1.17.1	(C)
GROUNDWATER TABLE DEPTH	2	2.2.2	6	200-400	M	NONE	(B)	
AVERAGE INFILTRATION	2	2.2.2	10	0.1	MM/YR	NONE	1.1.5	(C)
CONTOURS OF THICKNESS OF UNSATURATED ROCK BENEATH REPOSITORY	2	2.2.2	8	--	M	--	1.3.1.1.2	IN CALMA SYSTEM
BULK POROSITY - CALICO HILLS (ZEOLITIZED)	2	TABLE 2-2	9	30±8.6	%	19.7-36.8	1.3.1.2.1	(A)
BULK POROSITY - CALICO HILLS (VITRIFIED)	2	TABLE 2-2	9	39±7.7	%	33.5±9.7	1.3.1.2.1	(A)
BULK POROSITY - OLDER TUFFS	2	TABLE 2-2	9	23	%	16.4-39.4	1.3.1.2.1	(C)
BULK POROSITY - PAINTBRUSH TUFF	2	TABLE 2-2	9	45±11.8	%	44.8	1.3.1.2.1	(A)
BULK POROSITY - TIVA CANYON	2	TABLE 2-2	9	10±5.3	%	11.4	1.3.1.2.1	(A)
BULK POROSITY - TOPOPAH SPRING	2	TABLE 2-2	9	15±5.1	%	0.8-18.3	1.3.1.2.1	(C)
FRACTURE DENSITY OF ALL UNITS	2	TABLE 2-2	9	5-40	$1/M^3$	NONE	1.3.2.4.2.1	(C)
SATURATED, HYDRAULIC CONDUCTIVITY - CALICO HILLS (VITRIFIED)	2	TABLE 2-2	9	1.3×10^{-6}	CM/S	3.4×10^{-7}	1.1.4.2	(A)
SATURATED, HYDRAULIC CONDUCTIVITY - CALICO HILLS (ZEOLITIZED)	2	TABLE 2-2	9	4.2×10^{-9}	CM/S	1.7×10^{-9}	1.1.4.2	(A)

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SATURATED HYDRAULIC CONDUCTIVITY - PAINTBRUSH TUFF	2	TABLE 2-2	9	2.4×10^{-6}	CM/S	NONE	1.1.4.2	(C)
SATURATED HYDRAULIC CONDUCTIVITY - TIVA CANYON	2	TABLE 2-2	9	2.5×10^{-9}	CM/S	NONE	1.1.4.2	(C)
SATURATED HYDRAULIC CONDUCTIVITY - TOPOPAH SPRING	2	TABLE 2-2	9	3.5×10^{-9}	CM/S	2.3×10^{-9}	1.1.4.2	(A)
SATURATED HYDRAULIC CONDUCTIVITY OF THE OLDER TUFFS	2	TABLE 2-2	9	1.1×10^{-7}	CM/S	6.8 TO 37×10^{-8}	1.1.4.2	(A)
SATURATED ROCK MASS HYDRAULIC CONDUCTIVITY - ALL UNITS	2	TABLE 2-2	9	10^{-2} TO 10^{-6}	CM/S	NONE	1.1.4.3	(C)
MAXIMUM INFILTRATION RATE	2	2.2.2	10	0.5	MM/YR	0.722	1.1.4.2	(A)
AMOUNT OF SPENT FUEL	2	2.3	11	59,350	MTHM	NONE	(B)	
DEFENSE HIGH-LEVEL WASTE	2	2.3	11	10,000	MTU	NONE	(B)	
ESTIMATED NUMBER OF CANISTERS IN REPOSITORY	2	2.3	11	42,200	--	45651	2.2.2.1 & 2.2.4.9 & 10	(A)
MONITORED RETRIEVABLE STORAGE ASSUMED SPENT FUEL	2	2.3	11	62,000	MTHM	NONE	(B)	
MONITORED RETRIEVABLE STORAGE DEFENSE HIGH-LEVEL WASTE	2	2.3	11	8,000	MTHM	NONE	(B)	
MTU'S CONTAINED IN BWR ASSEMBLIES	2	2.3	11	0.1833	MTU	NONE	(B)	
MTU'S CONTAINED IN PWR ASSEMBLIES	2	2.3	11	0.4614	MTU	NONE	(B)	
PERCENT OF SPENT FUEL IN BWR ASSEMBLIES	2	2.3	11	40	%	NONE	(B)	
PERCENT OF SPENT FUEL IN PWR ASSEMBLIES	2	2.3	11	60	%	NONE	(B)	
REFERENCE WASTE AMOUNT OF DEFENSE HIGH-LEVEL WASTE	2	2.3	11	7,350	MTHM	NONE	(B)	

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REFERENCE WASTE AMOUNT OF WEST VALLEY HIGH-LEVEL WASTE	2	2.3	11	650	MTU	NONE	(B)	
WEST VALLEY HIGH-LEVEL WASTE	2	2.3	11	650	MTU	NONE	(B)	
GENERAL LAYOUT OF THE REPOSITORY	2	2.4	12	18	PANEL	NONE	2.2.1.1.6	(C)
NUMBER OF CONSOLIDATED BWR ASSEMBLIES EACH CANISTER HOLDS	2	2.3	11	12	--	NONE	(B)	
NUMBER OF CONSOLIDATED PWR ASSEMBLIES EACH CANISTER HOLDS	2	2.3	11	6	--	NONE	(B)	
NUMBER OF INTACT BWR ASSEMBLIES EACH CANISTER HOLDS	2	2.3	11	6	--	NONE	(B)	
NUMBER OF INTACT PWR ASSEMBLIES EACH CANISTER HOLDS	2	2.3	11	3	--	NONE	(B)	
PERCENT OF CONSOLIDATED FUEL ASSEMBLIES	2	2.3	11	90	%	NONE	(B)	
DEFENSE HIGH-LEVEL WASTE IN EACH WASTE CANISTER	2	2.3	11	0.5	MTU	NONE	(B)	
NUMBER OF CANISTERS CONTAINING WVHLW	2	2.3	11	310	---	NONE	(B)	
PERCENT OF UNCONSOLIDATED FUEL ASSEMBLIES	2	2.3	11	10	%	NONE	(B)	
REPOSITORY DEPTH	2	2.4	12	305 (1000)	M (FT)	NONE	2.2.7.3	(C)
EMPLACEMENT AND EXHAUST SHAFT DEPTH BELOW REPOSITORY	2	2.4	15	10	FT	NONE	(B)	
EXPLORATORY SHAFT DEPTH BELOW REPOSITORY	2	2.4	15	470	FT	NONE	(B)	
MEN AND MATERIALS SHAFT DEPTH BELOW REPOSITORY	2	2.4	15	80	FT	NONE	(B)	
MAXIMUM GRADE FOR DRIFTS	2	2.4	12	8	%	NONE	(B)	

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PARAMETER	CHAPTER	SECTION	PAGE#	REPORT VALUE	UNITS	RIB VALUE	RIB SECTION	EXPLANATION
EMPLACEMENT AND EXHAUST SHAFT DEPTH	2	TABLE 2-3	17	1040	FT	NONE	(B)	
EMPLACEMENT AND EXHAUST SHAFT EXCAVATED DIAMETER	2	TABLE 2-3	17	22.5	FT	22	2.2.5.4	SEE SNL DRWG. NO. R07001
EMPLACEMENT AND EXHAUST SHAFT FINISHED DIAMETER	2	TABLE 2-3	17	20	FT	20	2.2.5.4	
ES-2 DEPTH	2	TABLE 2-3	17	1020	FT	NONE	(B)	
ES-2 EXCAVATED DIAMETER	2	TABLE 2-3	17	8	FT	8	2.2.5.3	
ES-2 FINISHED DIAMETER	2	TABLE 2-3	17	6	FT	6	2.2.5.3	
EXPLORATORY SHAFT DEPTH	2	TABLE 2-3	17	1490	FT	NONE	(B)	
EXPLORATORY SHAFT EXCAVATED DIAMETER	2	TABLE 2-3	17	14.5	FT	14	2.2.5.2	(A)
EXPLORATORY SHAFT FINISHED DIAMETER	2	TABLE 2-3	17	12	FT	12	2.2.5.2	
MEN AND MATERIALS SHAFT DEPTH	2	TABLE 2-3	17	1110	FT	NONE	(B)	
MEN AND MATERIALS SHAFT EXCAVATED DIAMETER	2	TABLE 2-3	17	22.5	FT	22	2.2.5.1	SEE SNL DRWG NO. R07001
MEN AND MATERIALS SHAFT FINISHED DIAMETER	2	TABLE 2-3	17	20	FT	20	2.2.5.1	
TUFF RAMP EXCAVATED WIDTH (HORIZONTAL)	2	TABLE 2-3	17	21	FT	21	2.2.5.6	
TUFF RAMP EXCAVATED WIDTH (VERTICAL)	2	TABLE 2-3	17	25	FT	25	2.2.6.6	
TUFF RAMP FINISHED WIDTH (HORIZONTAL)	2	TABLE 2-3	17	20	FT	20	2.2.5.6	
TUFF RAMP FINISHED WIDTH (VERTICAL)	2	TABLE 2-3	17	24	FT	24	2.2.6.6	
TUFF RAMP LENGTH	2	TABLE 2-3	17	4627	FT	NONE	(B)	
WASTE RAMP EXCAVATED WIDTH (HORIZONTAL)	2	TABLE 2-3	17	21	FT	21	2.2.5.5	

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WASTE RAMP EXCAVATED WIDTH (VERTICAL)	2	TABLE 2-3	17	23	FT	23	2.2.6.5	
WASTE RAMP FINISHED WIDTH (HORIZONTAL)	2	TABLE 2-3	17	19	FT	19	2.2.5.5	
WASTE RAMP FINISHED WIDTH (VERTICAL)	2	TABLE 2-3	17	21	FT	21	2.2.6.5	
WASTE RAMP LENGTH	2	TABLE 2-3	17	6603	FT	NONE	(B)	
DIMENSIONS, AREAS, VOLUMES FOR THE UNDERGROUND EXCAVATED VERTICAL MODE	2	TABLE 2-4	19-20	--	--	NONE	(B)	(D)
DIMENSIONS, AREAS, VOLUMES FOR THE UNDERGROUND EXCAVATED HORIZONTAL MODE	2	TABLE 2-5	21-22	--	--	NONE	(B)	(D)
EXTRACTION RATIO HORIZONTAL EMPLACEMENT	2	2.4	15	8.73	%	NONE	(B)	
EXTRACTION RATIO VERTICAL EMPLACEMENT	2	2.4	15	17.7	%	NONE	(B)	
NONEMPLACEMENT DRIFTS (HORIZONTAL)	2	2.4	15	67	%	NONE	(B)	
NONEMPLACEMENT DRIFTS (VERTICAL)	2	2.4	16	39	%	NONE	(B)	
SUMMARY OF AREAS AND VOLUMES FOR UNDERGROUND EXCAVATIONS	2	TABLE 2-6	23	--	--	NONE	(B)	(D)
PERCENT RADIONUCLIDES WITH THE UO_2 MATRIX	3	3.1	1	90	%	NONE	(B)	
PERCENT CS-135, CS-137, I-129 AND SR90 WITHIN THE GAP	3	3.1.1.1	3	0.5	%	NONE	(B)	
GRAIN BOUNDARY PREFERENTIAL DISSOLUTION FACTORS	3	3.1.1.1	3	3.8-8.7	--	NONE	(B)	

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PERCENT C-14 INVENTORY IN THE STRUCTURAL PARTS - BWR	3	3.1.1.1	4	67	%	NONE	(B)	
PERCENT C-14 INVENTORY IN THE STRUCTURAL PARTS	3	3.1.1.2	4	61	%	NONE	(B)	
PERCENT C-14 WITHIN THE FULL CLADDING	3	3.1.1.2	4	67	%	NONE	(B)	
PERCENT NI-59 WITHIN THE FULL CLADDING	3	3.1.1.2	4	100	%	NONE	(B)	
PERCENT ZR-93 WITHIN THE FULL CLADDING	3	3.1.1.2	4	5	%	NONE	(B)	
DEPTH OF OXIDIZED ZIRCOLOY	3	3.1.1.2	4	17	MM	NONE	(B)	
DEPTH OF OXIDIZED ZIRCOLOY	3	3.1.1.2	4	0.0469	MM	NONE	(B)	
PERCENT GASEOUS C-14	3	3.1.1.2	5	0.3	%	NONE	(B)	
PERCENT OF FAILED FULL RODS	3	3.1.1.2	5	.02	%	NONE	(B)	
SOLUBILITY LIMIT OF THE URANIUM MATRIX IN SPARTAN	3	3.1.2	10	5×10^{-3}	KG/M^3	NONE	1.2.4.5	(C)
PERCENT OF CESIUM AND IODINE FOR FISSION GAS RELEASE	3	3.1.4	12	50	%	NONE	(B)	
PERCENT GASEOUS SPECIES OF THE TOTAL INVENTORY	3	3.2	20	0.5	%	NONE	(B)	
VALLEY WIDTH AT EMPLACEMENT EXHAUST	4	4.1.1	2	107 (350)	M (FT)	NONE	(B)	
VALLEY WIDTH AT ESCAPE SHAFT LOCATION	4	4.1.1	2	107 (350)	M (FT)	NONE	(B)	
VALLEY WIDTH AT MEN AND MATERIALS SHAFT LOCATION	4	4.1.1	2	244 (800)	M (FT)	NONE	(B)	
TOTAL FLOW - PMF 14 HOUR EMPLACEMENT EXHAUST SHAFT	4	TABLE 4-1	3	24	ACRE-FT	NONE	1.17.1.3	(C, D)

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TOTAL FLOW - PMF 14 HOUR EXPLORATORY SHAFT	4	TABLE 4-1	3	75	ACRE-FT	NONE	1.17.1.3	(C, D)
TOTAL FLOW - PMF 14 HOUR MEN AND MATERIALS SHAFT	4	TABLE 4-1	3	107	ACRE-FT	NONE	1.17.1.3	(C, D)
TOTAL FLOW - PMF 6 HOUR EMPLACEMENT EXHAUST SHAFT	4	TABLE 4-1	3	41	ACRE-FT	NONE	1.17.1.3	(C, D)
TOTAL FLOW - PMF 6 HOUR EXPLORATORY SHAFT	4	TABLE 4-1	3	129	ACRE-FT	NONE	1.17.1.3	(C, D)
TOTAL FLOW - PMF 6 HOUR MEN AND MATERIALS SHAFT	4	TABLE 4-1	3	183	ACRE-FT	NONE	1.17.1.3	(C, D)
SHAFT FILL HYDRAULIC CONDUCTIVITY	4	TABLE 4-2	7	10^{-2}	CM/S	NONE	(B)	
MINIMUM PRECIPITATION FOR RUNOFF	4	4.1.4	8	0.5	IN	NONE	(B)	
EXTENSION OF MPZ FROM SHAFT WALL	4	4.1.5	10	ONE	RADIUS	NONE	(B)	
RANGE MPZ VERSUS BULK ROCK HYDRAULIC CONDUCTIVITY TIVA CANYON	4	4.1.5	10	20 TO 60X GREATER	--	NONE	(D)	
SUCTION HEAD OF COHESIONLESS MATERIAL	4	4.1.5	13	-1.0	M	NONE	(B)	
SATURATED HYDRAULIC CONDUCTIVITY OF FILL	4	4.1.5	16	10^{-2}	CM/S	NONE	(B)	
ALLUVIAL GRADE	4	4.1.5	16	16	%	NONE	(B)	
ALLUVIAL THICKNESS IN DRAINAGE MODEL	4	4.1.5	16	9.1 (30)	M (FT)	NONE	(B)	
DRIFT INFLOW - CLAY FILL	4	4.2.1	22	1.3×10^{-12}	M^3/S	NONE	(B)	
DRIFT INFLOW - SAND FILL	4	4.2.1	22	9.7×10^{-15}	M^3/S	NONE	(B)	
SATURATED ROCK MATRIX PERMEABILITY	4	4.2.1	22	1.9×10^{-9}	CM/S	NONE	(B)	
FAULT ZONE HYDRAULIC CONDUCTIVITY	4	4.2.2.2	25	10^{-5} TO 10^{-2}	CM/S	NONE	1.1.4	(C)
GREEN & AMPT PARAMETER - HYDRAULIC CONDUCTIVITY OF FAULT	4	4.2.2.2	26	10^{-2} TO 10^{-5}	CM/S	NONE	(B)	
GREEN & AMPT PARAMETER - POROSITY OF FAULT	4	4.2.2.2	26	0.004 TO 0.4		NONE	(B)	

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TOTAL DISCHARGE TUNNEL U12e	4	4.2.2.4	26	30 TO 50	MILLION GALS	NONE	(B)	
FAULT APERTURE	4	4.2.4	33	0.3	M	NONE	1.3.2.4.2.3	(C)
PERCENT SATURATION IN FAULT	4	4.2.4	33	0-85	%	NONE	(B)	
CALICO HILLS HYDRAULIC CONDUCTIVITY	6	6.7.3	47	10^{-8} TO 10^{-4}	CM/S	NONE	1.1.4.2	(C)
RANGE OF SLOPE GRADES IN MAINS	6	6.5.3.3	27	2.5 TO 75	%	NONE	(B)	
BOREHOLES TO WATER TABLE WITHIN 5 KM OF REPOSITORY	6	6.7.1	42	25	--	--	1.3.1.1.2	IN CALMA SYSTEM
CONTOUR MAPS OF STRATIGRAPHIC CONTACTS	6	6.7.3	42	--	M	--	1.3.1.1.2	IN CALMA SYSTEM
SCHEMATIC CASES ANALYZED FOR BOREHOLE DESIGN REQUIREMENT DEVELOPMENT	6	6.7.4	49	--	M	--	1.3.1.1.2	IN CALMA SYSTEM
HYDRAULIC CONDUCTIVITY OF GROUT AND CONCRETE	7	7.4.1	13	1.0 TO 9.5×10^{-10}	CM/S	NONE	(B)	
RANGE OF TYPICAL CONCRETE HYDRAULIC CONDUCTIVITIES	7	7.4.1	13	$<10^{-5}$ TO 10^{-3}	DARCY	NONE	(B)	
ALLUVIAL THICKNESS NEAR EXPLORATORY AND ESCAPE SHAFT	A-1	A-1.1	1	6.5 TO 10	M (FT)	NONE	1.3.1.1.3	(C)
100-YEAR FLOOD PEAK DISCHARGE	A-1	A-1.2	14	$482\ A^{0.565}$	CFS	NONE	(B)	(D)
500-YEAR FLOOD PEAK DISCHARGE	A-1	A-1.2	14	$2{,}200\ A^{0.571}$		NONE	(B)	(D)
ALLUVIUM/COLLUVIUM THICKNESS NEAR WASTE HANDLING RAMP	A-1	A-1.1.5	7	15 (50)	M (FT)	NONE	1.3.1.1.3	(C)
MAXIMUM EXPECTED RAINFALL IN SOUTHERN NEVADA IN 1 HOUR	A-1	A-1.2	16	18 (7)	CM (IN)	NONE	1.11.2.2	(C)
WATERSHED AREA OF COYOTE WASH	A-1	A-1.2	14	(0.19) 0.48	K^2 (MI^2)	NONE	(B)	
HYDRAULIC CONDUCTIVITY OF BOTH CALICO HILLS AND TOPOPAH SPRING	A-5	A-5.1.1	36	5×10^{-6}	CM/S	NONE	1.1.4.3	(C)

APPENDIX H

COMPARISON OF DATA USED IN THIS REPORT WITH THE REFERENCE INFORMATION BASE (RIB)

THE FOLLOWING NOTES ARE USED THROUGHOUT THIS APPENDIX:

(A) DATA OBTAINED FROM REFERENCE INDICATED IN THE TEXT. THE VALUES USED WERE SELECTED PRIOR TO ISSUANCE OF THE RIB.
(B) NO SECTION FOUND IN RIB APPLICABLE TO THESE PARAMETERS.
(C) SECTION IDENTIFIED IN THE RIB, BUT VALUES NOT FOUND.
(D) RECOMMEND VALUES BE INCORPORATED INTO THE RIB.

PARAMETER	CHAPTER	SECTION	PAGE#	REPORT VALUE	UNITS	RIB VALUE	RIB SECTION	EXPLANATION
AIR CONDUCTIVITIES IN REPOSITORY BACKFILL	C	C.3.2	7	3.0×10^{-6} - 3.0	M/MIN	NONE	(B)	
AIR CONDUCTIVITY IN DAMAGED ZONE	C	C.3.2	7	20 TO 60X GREATER		NONE	(B)	
TOTAL ROOF AREA OF REPOSITORY (VERTICAL)	C	C.3.2	8	983,700	M^2	NONE	(B)	
TOTAL ROOF AREA OF REPOSITORY (HORIZONTAL)	C	C.3.2	8	486,000	M^2	NONE	(B)	
ASSUMED AIR CONDUCTIVITY FOR TIVA CANYON AND TOPOPAH SPRING	C	C.3.2	8	3×10^{-7} TO 3×10^{-4}	M/MIN	NONE	(B)	
ASSUMED AIR CONDUCTIVITY OF NONWELDED PAINTBRUSH	C	C.3.2	8	3×10^{-7} TO 3×10^{-5}	M/MIN	NONE	(B)	
THICKNESS OF NONWELDED UNITS AT PAINTBRUSH	C	C.3.2	8	40	M	1.3.1.1.1	(C)	
TOTAL THICKNESS OF WELDED UNITS AT TIVA CANYON AND TOPOPAH SPRING	C	C.3.2	8	260	M	1.3.1.1.1	(C)	
ESTIMATED RANGE OF POROSITY OF BACKFILL	D	D.1	2	30 TO 35	%	NONE	(B)	

APPENDIX I

DATA RECOMMENDED FOR INCLUSION INTO THE SITE AND ENGINEERING PROPERTIES DATA BASE (SEPDB) AND INFORMATION PROPOSED FOR THE INCLUSION INTO THE REFERENCE INFORMATION BASE (RIB)

The data in Appendix G is recommended for inclusion into the SEPDB.

The information proposed for inclusion into the RIB is the modified permeability zone (MPZ) model that has been used in this report. The following table summarizes the MPZ model. Details of how the model was developed are contained within Case and Kelsall (1987).

Table I.1. Equivalent Permeability of the Modified Permeability Zone[a]

Depth	Stress Redistribution Without Blast Damage		Expected[b] Case	Upper Bound[c] Case
	Elastic	Elastoplastic		
100	15	20	20	40
310	15	40	20	80

[a] Equivalent permeability is averaged over an annulus one-radius wide around the 4.4-m-diameter, exploratory shaft.

[b] This is based on an elastic analysis with expected strength, in situ stress, least sensitivity of permeability to stress, and a 0.5-m-wide, blast-damage zone.

[c] This is based on an elastoplastic analysis with lower-bound strength; upper-bound, in situ stress; greatest sensitivity of permeability to stress; and a 1.0-m-wide, blast-damage zone.

REFERENCES

Abeele, W. V., 1984, Determination of Hydraulic Conductivity in Crushed Bandelier Tuff, LA-8147-MS, Los Alamos National Laboratory, Los Alamos, NM.

Aines, R. D., 1986, Estimates of Radionuclide Release from Glass Waste Forms in a Tuff Repository and the Effects on Regulatory Compliance, UCRL 93735, Lawrence Livermore National Laboratory, Livermore, CA.

ANSI (American National Standards Institute), 1981, American National Standard for Determining Design Basis Flooding at Power Reactor Sites, ANSI/ ANS-2.8-1981, American Nuclear Society, La Grange Park, IL.

Apted, M. J., A. M. Liebetrau, and D. W. Engel, 1986, Analysis of Spent Fuel Performance in a Geologic Repository, PNL-SA-13928, Pacific Northwest Laboratories, Richland, WA.

Bentley, C. B., 1984, Geohydrologic Data for Test Well USW G-4 Yucca Mountain Area, Nye County, Nevada, Open-File Report 84-063, prepared for U.S. Department of the Interior Geological Survey, Denver, CO.

Bullard, K. L, 1986, PMF (Probable Maximum Flood) Study for Nevada Nuclear Waste Storage Investigations Project, GR-87-8, United States Department of the Interior Bureau of Reclamation, Denver, CO.

Case, J. B. and P. C. Kelsall, 1987, Modification of Rock Mass Permeability in the Zone Surrounding a Shaft in Fractured, Welded Tuff, SAND86-7001, Sandia National Laboratories, Albuquerque, NM.

Coons, W. E., D. Meyer, and P. C. Kelsall, 1982, Evaluation of Polymer Concrete for Application to Repository Sealing, ONWI-410, Office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH.

Crippen, J. R. and C. D. Bue, 1977, Maximum Floodflows in the Counterminous United States, Geological Survey Water-Supply Paper No. 1887, U.S. Geological Survey, Denver, CO.

Daugherty, R. L. and J. B. Franzini, 1965, Fluid Mechanics with Engineering Applications, McGraw-Hill Book Company, New York, NY.

Davis, S. N. and J. M. DeWeist, 1966, Hydrogeology, John Wiley & Sons, Inc., New York, NY.

Dixon, D. A. and M. N. Gray, 1985, "Engineering Properties of Buffer Material," AECL TR-350, Proceedings of the 19th Information Meeting of the Nuclear Waste Management Program, Vol. III, Pinawa, Manitoba, September 1985.

DOE (U.S. Department of Energy), 1979, Technology for Commercial Radioactive Waste Management, DOE/ET-0028, Vol. 1, U.S. Department of Energy, Washington, D.C.

DOE (U.S. Department of Energy), 1986, Environmental Assessment, Yucca Mountain Site, Nevada Research and Development Area, Nevada, DOE/RW-0073, U.S. Department of Energy, Office of Civilian Radioactive Waste Management, Washington, D.C.

DOE (U.S. Department of Energy), in preparation, Site Characterization Plan -- Yucca Mountain Site, Nevada Research and Development Area, Nevada, U.S. Department of Energy, Office of Civilian Radioactive Waste Management, Washington, D.C. (August 17, 1987 version).

Eglinton, T. W. and R. J. Dreicer, 1984, Meteorological Design Parameters for the Candidate Site of a Radioactive Waste Repository at Yucca Mountain, Nevada, SAND84-0440/2, Sandia National Laboratories, Albuquerque, NM.

EPA (U.S. Environmental Protection Agency), 1986, "Environmental Radiation Protection Standards for the Management and Disposal of Spent Nuclear Fuel, High-Level and Transuranic Radioactive Wastes," Code of Federal Regulations, Protection of Environment, Title 40, Part 191, U.S. Government Printing Office, Washington, D.C.

Evans, D. D. and C. Huang, 1983, "Role of Desaturation on Transport Through Fractured Rock," Role of the Unsaturated Zone in Radioactive and Hazardous Waste Disposal, J. Mercer, ed., Ann Arbor Science, Ann Arbor, MI.

Fagerstrom, H. and B. Lundahl, 1977, Handling of Buffer Material Quartz-Bentonite, KBS-37, Karn-Bransle-Sakarhet (KBS), Stockholm, Sweden.

Fernandez, J. A. and M. D. Freshley, 1984, Repository Sealing Concepts for the Nevada Nuclear Waste Storage Investigations Project, SAND83-1778, Sandia National Laboratories, Albuquerque, NM.

Fernandez, J. A., 1985, Repository Sealing Plan Fiscal Years 1984 through 1990 for the Nevada Nuclear Waste Storage Investigations Project, SAND84-0910, Sandia National Laboratories, Albuquerque, NM.

Fernandez, J. A. and G. S. Wong, in preparation, Evaluation of Concrete Using Tuff as a Coarse Aggregate for the NNWSI Repository Sealing Program, SAND87-1374, Sandia National Laboratories, Albuquerque, NM.

Freeze, R. A. and J. A. Cherry, 1979, Groundwater, Prentice-Hall, Inc., Englewood Cliffs, NJ.

Freshley, M.D., F. H. Dove, and J. A. Fernandez, 1985, Hydrologic Calculations to Evaluate Backfilling Shafts and Drifts for a Prospective Nuclear Waste Repository in Unsaturated Tuff, SAND83-2465, Sandia National Laboratories, Albuquerque, NM.

Glancy, P. A. and L. Harmsen, 1975, A Hydrologic Assessment of the September 14, 1974, Flood in Eldorado Canyon, Nevada, U.S. Geological Survey, Professional Paper 930, Washington, D.C.

Grim, R. E., 1962, Clay Mineralogy, McGraw-Hill Book Company, New York, NY.

Hall, C. J., 1981, Mine Ventilation Engineering, Society of Mining Engineers of the AIME, New York, NY.

Hartman, H. L., 1982, Mine Ventilation and Air Conditioning, John Wiley & Sons, NY.

Hillel, D., 1971, Soil and Water, Physical Principles and Processes, Academic Press, Inc., New York, NY.

Howard, James J. and D. M. Roy, 1985, "Development of Layer Charge and Kinetics of Experimental Smectite Alteration," Journal of Clays and Clay Minerals, Vol. 33, No. 2.

IT Corporation, 1984, Assessment of Crushed Salt Consolidation and Fracture Healing Processes in a Nuclear Waste Repository in Salt, BMI/ONWI-546, Office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH.

Johnson, L. H., K. I. Burns, H. H. Joling, and C. J. Moore, 1983, "Leaching of ^{137}Cs, ^{134}Cs, and ^{129}I from Irradiated UO_2 Fuel," Nuclear Technology, Vol. 63.

Karol, R. H., 1982, "Chemical Grouts and Their Properties," Proceedings of the Conference on Grouting in Geotechnical Engineering, American Society of Civil Engineers, New York, NY.

Keely, J. F. and C. F. Tsang, 1983, "Velocity Plots and Capture Zones of Pumping Centers for Ground-Water Investigations," Ground Water, Vol. 21, No. 6.

Kelsall, P. C., J. B. Case, and C. R. Chabannes, 1982, A Preliminary Evaluation of the Rock Mass Disturbance Resulting from Shaft, Tunnel, or Borehole Excavation, ONWI-411, Office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH.

Licastro, P. H., D. M. Roy, and J. A. Fernandez, in preparation, Mechanical Compatibility Between Selected Cementitious Materials and Topopah Spring Member Tuff, SAND86-0558, Sandia National Laboratories, Albuquerque, NM.

Lin, Y. T., 1985, SPARTAN--A Simple Performance Assessment Code for the Nevada Nuclear Waste Storage Investigations Project, SAND85-0602, Sandia National Laboratories, Albuquerque, NM.

Lingle, R. and D. D. Bush, 1982. Draft Report: Full Scale Borehole Sealing Test in Basalt Under Simulated Downhole Conditions, TRE-82-09, Terra Tek Engineering for Office of Nuclear Waste Isolation, Battelle Memorial Institute, Columbus, OH.

Lundstrom, R. A., J. B. Case, P. C. Kelsall, and S. R. Cullinan, 1986, "The Influence of the Damaged Zone, Interface, and Various Sealing Components on Shaft Seal Performance for a Repository in Basalt," High-Level Nuclear Waste Disposal, H. C. Burkholder, ed., Battelle Press, Columbus, OH.

Mather, B., 1966, Stronger Concrete, US-CE-C No. 6-851, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, MS.

Mather, B., 1967, Cement Performance in Concrete, US-CE-C No. 6-787, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, MS.

Mather, K., 1965, High-Strength, High-Density Concrete, US-CE-C No. 6-698, U.S. Army Engineer Waterways Experiment Station, Corps of Engineers, Vicksburg, MS.

Mondy, L. A., B. L. Baker, and R. R. Eaton, 1985, Vadose Water Flow Around a Backfilled Drift Located in Tuff, SAND84-0369, Sandia National Laboratories, Albuquerque, NM.

Montazer, P. and W. E. Wilson, 1984, Conceptual Hydrologic Model of Flow in the Unsaturated Zone, Yucca Mountain, Nevada, Water-Resources Investigations Report 84-4345, U.S. Geological Survey, Lakewood, CO.

Neal, J. T., 1986, Preliminary Validation of Geology at Site for Repository Surface Facilities, Yucca Mountain, Nevada, SAND85-0815, Sandia National Laboratories, Albuquerque, NM.

NRC (U.S. Nuclear Regulatory Commission), 1986, "Disposal of High-Level Radioactive Wastes in Geologic Repositories," Code of Federal Regulations, Energy, Title 10, Part 60, U.S. Government Printing Office, Washington, D.C.

NWPA (Nuclear Waste Policy Act of 1982), 1983, Public Law 97-425, 96 Stat 2201, 42 USC 10101-10126, Washington, D.C.

O'Brien, P. D., 1985, Reference Nuclear Waste Descriptions for a Geologic Repository at Yucca Mountain, Nevada, SAND84-1848, Sandia National Laboratories, Albuquerque, NM.

ONWI (Office of Nuclear Waste Isolation), 1984, Reducing the Likelihood of Future Human Activities that Could Affect Geologic High-Level Waste Repositories, BMI/ONWI-537, Battelle Memorial Institute, Columbus, OH.

Ortiz, T. S., R. L. Williams, F. B. Nimick, B. C. Whittet. and D. L. South. 1985, A Three-Dimensional Model of Reference Thermal/Mechanical and Hydrological Stratigraphy at Yucca Mountain, Southern Nevada, SAND84-1076, Sandia National Laboratories, Albuquerque, NM.

Oversby, V. M., 1984, Reaction of the Topopah Spring Tuff with J-13 Well Water at 90°C and 150°C, UCRL-53552, Lawrence Livermore National Laboratory, Berkeley, CA.

Oversby, V. M. and R. D. McCright, 1984, Laboratory Experiments Designed to Provide Limits on the Radionuclide Source Term for the NNWSI Project, UCRL-91257, Lawrence Livermore Laboratory, Livermore, CA.

Oversby, V. M. and C. N. Wilson, 1985, "Derivation of a Waste Package Source Term for NNWSI from the Results of Laboratory Experiments, terials Research Society Symposia Proceedings on the Scientific Basis for Nuclear Waste Management, Materials Research Society, Pittsburgh, PA.

Peters, R. R., E. A. Klavetter, I. J. Hall, S. C. Blair, P. R. Heller, and G. W. Gee, 1984, Fracture and Matrix Hydrologic Characteristics of Tuffaceous Materials from Yucca Mountain, Nye County, Nevada, SAND84-1471, Sandia National Laboratories, Albuquerque, NM.

Peterson, E. W. and C. L. Christensen, 1980, Analysis of Bell Canyon Test Results, SAND80-7044C, Sandia National Laboratories, Albuquerque, NM.

Pruess, K., J. S. Y. Wang, and Y. W. Tsang, 1986, Effective Continuum Approximation for Modeling Fluid and Heat Flow in Fractured Porous Tuff, SAND86-7000, Sandia National Laboratories, Albuquerque, NM.

Pusch, R., 1978, Self-Injection of Highly Compacted Bentonite into Rock Joints, KBS-73, Karn-Bransle-Sakarhet (KBS), Stockholm, Sweden.

Pusch, R., 1979, Water Percolation Effects on Clay-Poor Bentonite/Quartz Buffer Material at High Hydraulic Gradients, KBS-19, Karn-Bransle-Sakarhet (KBS), Stockholm, Sweden.

Pusch, R., 1980, Swelling Pressure of Highly Compacted Bentonite, TR-80-13, Karn-Bransle-Sakarhet (KBS), Stockholm, Sweden.

Radhakrishna, H. S. and H. T. Chan, 1982, "Laboratory Study of Physical Properties of Clay Buffers for a Nuclear Fuel Waste Vault," Proceedings from the International Conference of Radioactive Waste Management, Toronto, Ontario, Canada.

Reynolds, W. D., D. E. Elrick, and G. C. Topp, 1983, "A Reexamination of the Constant Head Well Permeameter Method for Measuring Saturated Hydraulic Conductivity Above the Water Table," Soil Science, Vol. 136, No. 4.

Rothman, A. J., 1984, Potential Corrosion and Degradation Mechanisms of Zircaloy Cladding on Spent Nuclear Fuel in a Tuff Repository, UCID-20172, Lawrence Livermore National Laboratory, Livermore, CA.

Scheetz, B. E. and D. M. Roy, in preparation, Preliminary Survey of the Stability of Silica-Rich Cementitious Mortars (82-22 and 84-12) with Tuff, Los Alamos National Laboratories, Los Alamos, NM.

Scott, R. B., R. W. Spengler, S. Diehl, A. R. Lappin, and M. P. Chornack, 1983, "Geologic Character of Tuffs in the Unsaturated Zone at Yucca Mountain, Southern Nevada," Role of the Unsaturated Zone in Radioactive and Hazardous Waste Disposal, J. Mercer, ed., Ann Arbor Sciences, Ann Arbor, MI.

Scott, R. B., G. D. Bath, V. J. Flanigan, D. B. Hoover, J. G. Rosenbaum, and R. W. Spengler, 1984, Geological and Geophysical Evidence of Structures in Northwest-Trending Washes, Yucca Mountain, Southern Nevada, and Their Possible Significance to a Nuclear Waste Repository in the Unsaturated Zone, Open-File Report 84-567, U.S. Geological Survey, Denver, CO.

Scott, R. B. and J. Bonk, 1984, Preliminary Geologic Map of Yucca Mountain, Nye County, Nevada with Geologic Sections, Open-File Report 84-494, U.S. Geological Survey, Denver, CO.

Seed, H. B., R. J. Woodward, Jr., and R. Lundgren, 1962, "Prediction of Swelling Potential for Compacted Clays," Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 88, No. SM3.

Sherif, M. A., I. Ishibashi, and B. W. Medhin, 1982, "Swell of Wyoming Montmorillonite and Sand Mixtures," Journal of Geotechnical Engineering, Proceedings of the American Society of Civil Engineers, Vol. 108, No. GT1.

Sinnock, S., Y. T. Lin, and J. P. Brannen, 1984, Preliminary Bounds on the Expected Postclosure Performance of the Yucca Mountain Repository Site, Southern Nevada, SAND84-1492, Sandia National Laboratories, Albuquerque, NM.

SNL (Sandia National Laboratories), 1987a, An Engineering Study of the Impact on Costs and Schedules of Using a Monitored Retrievable Storage Facility in Conjunction with a Repository in Tuff at Yucca Mountain, SAND85-7112, prepared by Bechtel National, Inc., for Sandia National Laboratories, Albuquerque, NM.

SNL (Sandia National Laboratories), 1987b, Site Characterization Plan - Repository Conceptual Design, SAND84-2641, compiled by H. R. MacDougall, L. W. Scully and J. R. Tillerson, Albuquerque, NM.

Squires, R. R. and R. L. Young, 1984, Flood Potential of Fortymile Wash and its Principal Southwestern Tributaries, Nevada Test Site, Southern Nevada, Water-Resources Investigations Report 83-4001, U.S. Geological Survey, Denver, CO.

Stephens, D. B. and S. P. Neuman, 1982a, "Vadose Zone Permeability Tests: Summary," Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 108, No. HY5.

Stephens, D. B. and S. P. Neuman, 1982b, "Vadose Zone Permeability Tests: Steady State Results," Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 108, No. HY5.

Taylor, C. L., G. J. Antonnen, J. E. O'Rourke, and D. Allirot, 1980, Preliminary Geochemical and Physical Testing of Materials for Plugging of Man-Made Accesses to a Repository in Basalt, RHO-BWI-C-66, Rockwell Hanford Operations, Richland, WA.

Terzaghi, K. and R. Peck, 1967, Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc., New York, NY.

Thordarson, W., 1965, Perched Ground Water in Zeolitized-Bedded Tuff, Rainier Mesa and Vicinity, Nevada Test Site, Nevada, Report No. TEI-862, U.S. Geological Survey, Denver, CO.

Todd, D. K., 1980, Groundwater Hydrology, John Wiley & Sons, Inc., New York, NY.

Uhlig, H. H., 1967, Corrosion and Corrosion Control - An Introduction to Corrosion Science and Engineering, John Wiley & Sons, Inc., New York, NY.

U.S.B.R. (U.S. Department of the Interior Bureau of Reclamation), 1974, Design of Small Dams, U.S. Government Printing Office, Washington, D.C.

U.S.G.S. (U.S. Department of the Interior Geological Survey), 1984, A Summary of Geologic Studies through January 1, 1983, of a Potential High-Level Radioactive Waste Repository Site at Yucca Mountain, Southern Nye County, Nevada, Open File Report 84-0792, U.S. Geological Survey, Menlo Park, CA.

Van Koynenburg, R. A., C. F. Smith, H. W. Culham, and C. H. Otto, Jr., 1984, "Behavior of Carbon-14 in Waste Packages for Spent Fuel in a Repository in Tuff," UCRL-90855, Materials Research Society Symposia Proceedings on the Scientific Basis for Nuclear Waste Management VIII (Boston, MA), Vol. 44, Materials Research Society, Pittsburgh, PA.

Wakeley, L. D. and D. M. Roy, 1982, "A Method for Testing the Permeability Between Grout and Rock," Cement and Concrete Research, Vol. 12, Pergamon Press, Ltd., United Kingdom.

Wang, J. S. Y. and T. N. Narasimhan, 1985, Hydrologic Mechanisms Governing Fluid Flow in Partially Saturated, Fractured, Porous Tuff at Yucca Mountain, LBL-18473, Lawrence Berkeley Laboratory, Berkeley, CA.

Westik, J. H., F. N. Hodges, W. L. Kuhn, and T. R. Myers, 1983, "Water Migration through Compacted Bentonite Backfills for Containment of High Level Nuclear Waste," Nuclear and Chemical Waste Management, Vol. 4., No. , pp. 291-299.

Wheelwright, E. J., F. N. Hodges, L. A. Bray, J. H. Westic, and D. H. Lester, 1981, Development of Backfill Material as an Engineered Barrier in the Waste Package System - Interim Topical Report, PNL-3873, Pacific Northwest Laboratory, Richland, WA.

Wilson, C. N., 1985, Results from NNWSI Series 1 Spent Fuel Leach Tests, HEDL-TME-84-3, Westinghouse Hanford Company, Richland, WA.

Wilson, C. N. and V. M. Oversby, 1985, Radionuclide Release from PWR Fuels in a Reference Tuff Repository Groundwater, UCRL-91464 Preprint, Lawrence Livermore Laboratory, Berkeley, CA.

Winterkorn, H. F. and H. Y. Fang, 1979, Foundation Engineering Handbook, Van Nostrand Reinhold Company, New York, NY.

Woodley, R. E., 1983, The Characteristics of Spent LWR Fuel Relevant to its Storage in Geologic Repositories, HEDL-TME-83-28, Hanford Engineering Development Laboratory, Richland, WA.

Yong, R. N., P. Boonsinsuk, and G. Wong, 1986, "Formulation of Backfill Material for a Nuclear Fuel Waste Disposal Vault," Canadian Geotechnical Journal, Vol. 23, Montreal, Canada.

Zeuch D. H., and M. J. Eatough, 1986, Draft Reference Information Base for the Nevada Nuclear Waste Storage Investigations Project, SLTR 86-5005, Sandia National Laboratories, Albuquerque, NM.

Zienkiewicz, O. C., 1977, The Finite Element Method, McGraw-Hill Book Company, New York, NY.

DISTRIBUTION LIST

B. C. Rusche (RW-1), Director
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

Ralph Stein (RW-23)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

J. J. Fiore (RW-221)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

M. W. Frei (RW-23)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

C. P. Gertz, Program Manager (5)
Waste Management Project Office
U.S. Department of Energy
Post Office Box 98518
Las Vegas, NV 89193-8518

Chief, Repository Projects Branch
Division of Waste Management
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

NTS Section Leader
Repository Project Branch
Division of Waste Management
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

B. G. Gale (RW-223)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

R. W. Gale (RW-40)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

J. O. Neff, Manager
Salt Repository Project Office
U.S. Department of Energy
110 North 25 Mile Avenue
Hereford, TX 79045

V. J. Cassella (RW-222)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

S. A. Mann, Manager
Crystalline Rock Project Office
U.S. Department of Energy
9800 South Cass Avenue
Argonne, IL 60439

K. Street, Jr.
Lawrence Livermore National Laboratory
P.O. Box 808
Mail Stop L-209
Livermore, CA 94550

L. D. Ramspott (3)
Technical Project Officer for NNWSI
Lawrence Livermore National Laboratory
P.O. Box 808
Mail Stop L-204
Livermore, CA 94550

Document Control Center
Division of Waste Management
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

V. M. Glanzman
U.S. Geological Survey
P.O. Box 25046
913 Federal Center
Denver, CO 80225

P. T. Prestholt
NRC Site Representative
1050 East Flamingo Road
Suite 319
Las Vegas, NV 89109

M. E. Spaeth
Technical Project Officer for NNWSI
Science Applications International
Corporation
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101 Convention Center Drive
Las Vegas, NV 89109

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A. E. Gurrola
General Manager
Energy Support Division
Holmes & Narver, Inc.
Mail Stop 580
P.O. Box 14340
Las Vegas, NV 89193-3838

D. T. Oakley (4)
Technical Project Officer for NNWSI
Los Alamos National Laboratory
N-S, Mail Stop F-619
P.O. Box 1663
Los Alamos, NM 87545

L. R. Hayes (6)
Technical Project Officer for NNWSI
U.S. Geological Survey
Post Office Box 25046
421 Federal Center
Denver, CO 80225

C. H. Johnson, Technical Program Manager
Nuclear Waste Project Office
State of Nevada
Evergreen Center, Suite 252
1802 North Carson Street
Carson City, NV 89701

ONWI Library
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Roy F. Weston, Inc.
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H. D. Cunningham, General Manager
Reynolds Electrical & Engineering
Co., Inc.
P.O. Box 14400
Mail Stop 555
Las Vegas, NV 89114

T. Hay, Executive Assistant
Office of the Governor
State of Nevada
Capitol Complex
Carson City, NV 89710

J. A. Cross, Manager
Las Vegas Branch
Fenix & Scisson, Inc.
Mail Stop 514
P.O. Box 93265
Las Vegas, NV 89193-3265

John Fordham
Desert Research Institute
Water Resources Center
P.O. Box 60220
Reno, NV 89506

Department of Comprehensive Planning
Clark County
225 Bridger Avenue, 7th Floor
Las Vegas, NV 89155

Lincoln County Commission
Lincoln County
P.O. Box 90
Pioche, NV 89043

Community Planning and Development
City of North Las Vegas
P.O. Box 4086
North Las Vegas, NV 89030

City Manager
City of Henderson
Henderson, NV 89015

Timothy G. Barbour
Science Applications International Corporation
1626 Cole Boulevard, Suite 270
Golden CO 80401

E. P. Binnall
Field Systems Group Leader
Building 50B/4235
Lawrence Berkeley Laboratory
Berkeley, CA 94720

R. R. Loux, Jr., Executive Director (3)
Nuclear Waste Project Office
State of Nevada
Evergreen Center, Suite 252
1802 North Carson Street
Carson City, NV 89701

Dr. Martin Mifflin
Desert Research Institute
Water Resources Center
Suite 1
2505 Chandler Avenue
Las Vegas, NV 89120

Planning Department
Nye County
P.O. Box 153
Tonopah, NV 89049

Economic Development Department
City of Las Vegas
400 East Stewart Avenue
Las Vegas, NV 89101

Director of Community Planning
City of Boulder City
P.O. Box 367
Boulder City, NV 89005

Commission of the European Communities
200 Rue de la Loi
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Technical Information Center
Roy F. Weston, Inc.
955 L'Enfant Plaza, Southwest, Suite 800
Washington, D.C. 20024

Gerald Parker (RW-241)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

T. H. Isaacs (RW-20)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

D. H. Alexander (RW-232)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

B. J. King, Librarian (2)
Basalt Waste Isolation Project Library
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Richland, WA 99352

D. L. Fraser, General Manager
Reynolds Electrical & Engineering Co., Inc.
Mail Stop 555
P.O. Box 98521
Las Vegas, NV 89193-8521

J. P. Pedalino
Technical Project Officer for NNWSI
Holmes & Narver, Inc.
101 Convention Center Drive, Suite 860
Las Vegas, NV 89109

Eric Anderson
Mountain West Research-Southwest, Inc.
Phoenix Gateway Center
432 North 44 Street
Suite #400
Phoenix, AZ 85008-6572

Judy Foremaster (5)
City of Caliente
P.O. Box 158
Caliente, NV 89008

J. P. Knight (RW-24)
Office of Civilain Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

Allen Jelacic (RW-233)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

J. R. Rollo
Deputy Assistant Director for Engineering Geology
U.S. Geological Survey
106 National Center
12201 Sunrise Valley Drive
Reston, VA 22092

Vincent Gong
Technical Project Officer for NNWSI
Reynolds Electrical & Engineering Co., Inc.
Mail Stop 615
P.O. Box 98521
Las Vegas, NV 89193-8521

S. H. Kale (RW-20)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, D.C. 20585

J. H. Anttonen, Deputy Assistant Manager for Commercial Nuclear Waste
Basalt Waste Isolation Project Office
U.S. Department of Energy
P.O. Box 550
Richland, WA 99352

J. C. Bresee (RW-22)
Office of Civilian Radioactive Waste Management
U.S. Department of Energy
Forrestal Building
Washington, DC 20585

J. L. Fogg (12)
Technical Information Office
Nevada Operations Office
U.S. Department of Energy
P.O. Box 98518
Las Vegas, NV 89193-8518

R. L. Bullock
Technical Project Officer for NNWSI
Fenix & Scisson, Inc.
Mail Stop 514
P.O. Box 93265
Las Vegas, NV 89193-3265

Robert A. Bellman, Jr.
Science Applications International Corporation
3349 South Highland Drive
Suite 403
Las Vegas, NV 89109

C. L. West, Director
Office of External Affairs
Nevada Operations Office
U.S. Department of Energy
P.O. Box 98518
Las Vegas, NV 89193-8518

P. K. Fitzsommons, Director
Health Physics and Environmental Division
Nevada Operations Office
U.S. Department of Energy
P.O. Box 98518
Las Vegas, NV 89193-8518

Prof. S. W. Dickson
Department of Geological Sciences
Mackay School of Mines
University of Nevada, Reno
Reno, NV 89557

Mark P. Board
ITASCA Consulting Group
1313 Fifth Street, SE
Minneapolis, MN 55414

Elaine Ezra, NNWSI GSI Project Manager
EG&G Energy Measurements, Inc.
Mail Stop H-02
P.O. Box 1912
Las Vegas, NV 89125

A. T. Tamura
Science and Technology Division
Office of Scientific and Technical
Information
U.S. Department of Energy
P.O. Box 62
Oak Ridge, TN 37831

John B. Case (5)
International Technology Corporation
2340 Alamo St. SE, Suite 306
Albuquerque, NM 87106

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